ICE manual of geotechnical engineering

Institution of Civil Engineers

ICE manual of geotechnical engineering

Volume I
Geotechnical Engineering Principles, Problematic Soils and Site Investigation

Edited by

John Burland
Imperial College London, UK
Tim Chapman
Arup Geotechnics, UK
Hilary Skinner
Donaldson Associates Ltd, UK
Michael Brown
University of Dundee, UK

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Foreword and endorsement

The civil engineering sector represents some of the best professionalism, foresight and talent of any profession and this hard won reputation has been built up over decades. This ICE Manual series helps the profession maintain this position through the provision of coherent and authoritative frameworks for the modern civil engineer.

The importance of geotechnical engineering cannot be underestimated. It has a critical role to play in almost all major infrastructure projects being carried out across the world today.

The challenges are significant to deliver projects with the lowest carbon output and value for money. Promoting and developing our understanding of the impact of earth materials on engineering schemes will be crucial if we are to deliver safe, sustainable and economic answers to these challenges.

In bringing together often fragmented sources in a single document for civil engineers ICE Publishing is providing an excellent service for the professional and their projects in the interest of society. I commend the publication of this work and look forward to future editions.

Richard Coackley BSc CEng FICE CWEM FCIWEM
ICE President 2011-2012

It is several centuries since the *Magistri Ludi* were respected for understanding known science. It is several decades since individuals could be compared to the *Magistri* in the broad field of geotechnical engineering. So the *ICE Manual of Geotechnical Engineering*, as conceived here, has a dual function: to aid the specialist practitioners in areas where they are less experienced and to guide the non-specialists in their approach to problems. It fulfils this role commendably.

Rab Fernie Eur Ing BSc CEng FICE FIHT FGS
Chairman, British Geotechnical Association

Preface

We began to formulate the initial ideas for this Manual as early as 2006. It had become apparent to us that civil and structural engineers not specialising in geotechnics face a daunting knowledge gap when they come up against a geotechnical problem. Most civil engineers leave university with very little grounding in geotechnical engineering. They will have a fair grasp of applied mechanics (mainly aimed at structural engineering). They will have had a basic introduction to geology and they will have studied the elements of soil mechanics and rock mechanics. But a recent graduate usually lacks a coherent understanding of the approach to, and methods of, geotechnical engineering and how these differ from other more widely practised branches of engineering. A survey carried out by ICE Publishing showed that information tends to be obtained from a wide range of sources through word of mouth, the internet and various publications. For the young practitioner this leads to a fragmented approach. Much of the geotechnical material is written by specialists for specialists and its *ad hoc* application by a general practitioner is often inappropriate and can be extremely dangerous. We felt that it would be of great benefit to our profession to provide a single first-port-of-call authoritative reference source aimed at informing the less experienced engineer. To our delight this concept was endorsed by the ICE Best Practice Panel and the British Geotechnical Association and has offered a unique opportunity to provide authoritative guidance within a coherent framework of good geotechnical engineering.

This *ICE Manual of Geotechnical Engineering* has been a labour of love! The contribution of 99 contributors and 10 section editors has made it possible to distil a great deal of experience from the profession into the books you see here. Don't imagine this will cover everything that a geotechnical engineer will face in their career – but it provides a "starting point" from which to build experience whilst remaining grounded in robust fundamentals.

As mentioned previously, the Manual is aimed at people in the early stage of their careers who need a readily accessible source of information when working in new aspects of geotechnical engineering. However it is expected that it also should prove valuable to all geotechnical engineering professionals. The aim has been to produce a manual that addresses the practice of geotechnical engineering in the 21st century including contemporary procurement, process and design standards and procedures. The grouping of chapters has been carefully chosen to facilitate a multi-disciplinary and holistic approach to the solution of construction challenges. A key message is the importance of drawing on "well-winnowed experience" for the smooth and reliable execution of projects. Such experience is best gained by working closely with a suitably experienced design or construction team.

It is hoped that this Manual will help in the training and development of the next generation of geotechnical engineers and will act as a useful source of reference to those with more experience.

The Editors are grateful to all those contributors and section editors who have generously given so much of their time and knowledge in producing such a comprehensive book.

John Burland, Tim Chapman, Hilary Skinner and Michael Brown

List of contributors

GENERAL EDITORS:
M. Brown University of Dundee, UK
J. B. Burland Imperial College London, UK
T. Chapman Arup, London, UK
H. D. Skinner Donaldson Associates Ltd, London, UK

SECTION EDITORS:
A. Bracegirdle Geotechnical Consulting Group, London, UK
M. Brown University of Dundee, UK
J. B. Burland Imperial College London, UK
M. Devriendt Arup, London, UK
A. Gaba Arup Geotechnics, London, UK
I. Jefferson University of Birmingham, UK
P. A. Nowak Atkins Ltd, Epsom, UK
A. S. O'Brien Mott MacDonald, Croydon, UK
W. Powrie University of Southampton, UK
T. P. Suckling Balfour Beatty Ground Engineering, Basingstoke, UK

CONTRIBUTORS:
S. Anderson Arup, London, UK
P. Ball Keller Geotechnique, St Helens, UK
F. G. Bell British Geological Survey, UK
A. Bell Cementation Skanska Ltd, Doncaster, UK
A. L. Bell Keller Group plc, London, UK
E. N. Bromhead Kingston University, London, UK
M. Brown University of Dundee, UK
J. B. Burland Imperial College London, UK
T. Chapman Arup, London, UK
J. Chew Arup London, UK
B. Clarke University of Leeds, UK
C. R. I. Clayton University of Southampton, UK
P. Coney Atkins, Warrington, UK
J. Cook Buro Happold Ltd, London, UK
D. Corke DCProjectSolutions, Northwich, UK
A. Courts Volker Steel Foundations Ltd, Preston, UK
J. C. Cripps University of Sheffield, UK
M. G. Culshaw University of Birmingham and British Geological Survey, UK
M. A. Czerewko URS (formerly Scott Wilson Ltd), Chesterfield, UK
J. Davis Geotechnical Consulting Group, London, UK
M. H. de Freitas Imperial College London and Director of First Steps Ltd, UK
M. Devriendt Arup, London, UK
P. G. Dumelow Balfour Beatty, London, UK
J. Dunnicliff Geotechnical Instrumentation Consultant, Devon, UK
C. Edmonds Peter Brett Associates LLP, Reading, UK
E. Ellis University of Plymouth, UK
R. Essler RD Geotech, Skipton, UK
I. Farooq Mott MacDonald, Croydon, UK
E. R. Farrell AGL Consulting, and Department of Civil, Structural and Environmental Engineering, Trinity College, Dublin, Republic of Ireland
R. Fernie Skanska UK Plc, Ricksmanworth, UK
S. French Testconsult Limited, Warrington, UK
A. Gaba Arup Geotechnics, London, UK
M. R. Gavins Keller Geotechnique, St Helens, UK
P. Gilbert Atkins, Birmingham, UK
S. Glover Arup London, UK
R. Handley Aarsleff Piling, Newark, UK
A. Harwood Balfour Beatty Major Civil Engineering, Redhill,UK
J. Hislam Applied Geotechnical Engineering, Berkhamsted, UK
V. Hope Arup Geotechnics, London, UK
G. Horgan Huesker, Warrington, UK
P. Ingram Arup, London, UK
I. Jefferson School of Civil Engineering, University of Birmingham, UK
C. Jenner Tensar International Ltd, Blackburn, UK
T. Jolley Geostructural Solutions Ltd, Old Hatfield, UK
L. D. Jones British Geological Survey, Nottingham, UK
J. Judge Tata Steel Projects, York, UK
M. Kemp Atkins, Epsom, UK
N. Langdon Card Geotechnics Ltd, Aldershot, UK
C. Lee (nee Swords) Card Geotechnics Ltd, Aldershot, UK
R. Lindsay Atkins, Epsom, UK
C. Macdiarmid SSE Renewables, Glasgow, UK
S. Manceau Atkins, Glasgow, UK
W. A. Marr Geocomp Corporation, Acton, MA, USA
J. Martin Byland Engineering, York, UK
B. T. McGinnity London Underground, London, UK
P. Morrison Arup, London, UK
D. Nicholson Arup Geotechnics, London, UK
R. Nicholson CAN Geotechnical Ltd, Chesterfield, UK
P. A. Nowak Atkins Ltd, Epsom, UK
A. S. O'Brien Mott MacDonald, Croydon, UK
T. Orr Trinity College, Dublin, Republic of Ireland
H. Pantelidou Arup Geotechnics, London, UK
D. Patel Arup, London, UK
S. Pennington Arup, London, UK

M. Pennington Balfour Beatty Ground Engineering, Basingstoke, UK
A. Pickles Arup, London, UK
D. Potts Imperial College London, UK
J. J. M. Powell BRE, Watford, UK
W. Powrie University of Southampton, UK
M. Preene Golder Associates (UK) Ltd, Tadcaster, UK
J. Priest Geomechanics Research Group, University of Southampton, UK
D. Puller Bachy Soletanche, Alton, UK
D. Ranner Balfour Beatty Ground Engineering, Basingstoke, UK
J. M. Reid TRL, Wokingham, UK
J. M. Reynolds Reynolds International Ltd, Mold, UK
C. Robinson Cementation Skanska Ltd, Doncaster, UK
C. D. F. Rogers University of Birmingham, UK
A. C. D. Royal University of Birmingham, UK
C. S. Russell Russell Geotechnical Innovations Limited, Chobham, UK
N. Saffari Atkins, London, UK
D. J. Sanderson University of Southampton, UK
H. Scholes Geotechnical Consulting Group (GCG), London, UK
C. J. Serridge Balfour Beatty Ground Engineering Ltd, Manchester, UK
H. D. Skinner Donaldson Associates Ltd, London, UK
J. A. Skipper Geotechnical Consulting Group, London, UK
B. Slocombe Keller Limited, Coventry, UK
P. Smith Geotechnical Consulting Group (GCG), London, UK
M. Srbulov Mott MacDonald, Croydon, UK
J. Standing Imperial College London, UK
J. Strange Card Geotechnics Ltd, Aldershot, UK
T. P. Suckling Balfour Beatty Ground Engineering, Basingstoke, UK
D. G. Toll Durham University, UK
V. Troughton Arup, London, UK
M. Turner Applied Geotechnical Engineering Limited, Steeple Claydon, UK
S. Wade Skanska UK Plc, Rickmansworth, UK
T. Waltham Engineering geologist, Nottingham, UK
M. J. Whitbread Atkins, Epsom, UK
C. Wren Independent Geotechnical Engineer
L. Zdravkovic Imperial College London, UK

Section 1: Context

Section editor: **John B. Burland and William Powrie**

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Chapter 1

Introduction to Section 1

John B. Burland Imperial College London, UK
William Powrie Imperial College London, UK

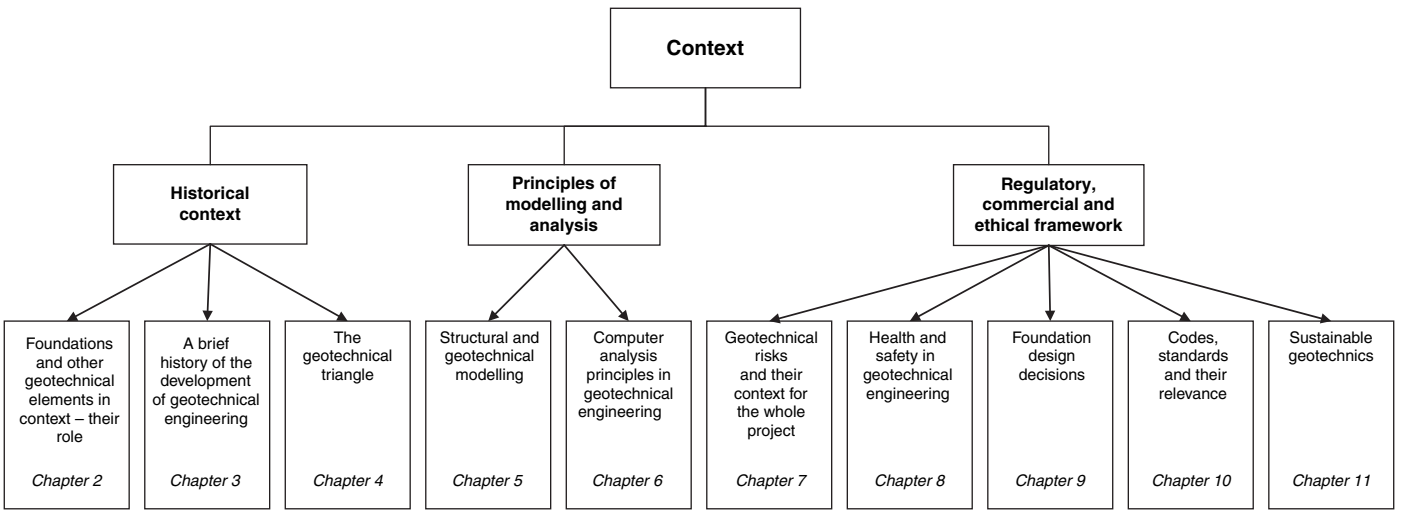


Figure 1.1 Layout of chapters in Section 1

Figure 1.1 outlines the layout and contents of Section 1 *Context*.

The first three parts of the section address the development of soil mechanics and geotechnical engineering as a distinct discipline over the past 80 years or so. Chapter 2 *Foundations and other geotechnical elements in context - their role* explains the importance of foundations and structures built in or of the ground within civil engineering and construction, and the need for a formal and holistic geotechnical engineering design process. Chapter 3 *A brief history of the development of geotechnical engineering* gives a history of the development of geotechnical engineering at the borderline between science and art, with the latter defined by Terzaghi in 1957 as a 'mental processes leading to satisfactory results without the assistance of step-for-step logical reasoning'. This is reflected in the 'geotechnical triangle', described in Chapter 4 *The geotechnical triangle*, which emphasises the essential elements of successful geotechnical engineering as understanding the ground, material properties and relevant precedence (well-winnowed experience), connected by an appropriate model for analysis.

Chapters 5 *Structural and geotechnical modelling* and 6 *Computer analysis principles in geotechnical engineering* address some important principles of modelling and analysis in geotechnical engineering. Particular attention is paid in Chapter 6 *Computer analysis principles in geotechnical engineering* to computer methods, for which it is essential that an engineer has a sound understanding of the basis of the method of analysis, the influence of the material properties used, and the shortcomings and limitations of the approach.

Finally, Chapters 7 *Geotechnical risks and their context for the whole project* to 11 *Sustainable geotechnics* discuss key aspects of the regulatory, commercial and ethical framework with which geotechnical engineering practice must comply, now and into the future. Chapter 9 *Foundation design decisions* emphasises that foundation engineering is a 'process' involving a number of interlinked operations, and design decisions need to take account of this. The other chapters address understanding and apportioning geotechnical risk within a whole project context (Chapter 7 *Geotechnical risks and their context for the whole project*); health and safety (Chapter 8 *Health and safety*

in geotechnical engineering); current standards and codes of practice (Chapter 10 *Codes, standards and their relevance*); and sustainability (Chapter 11 *Sustainable geotechnics*). These issues are especially important, given that the place occupied by geotechnical engineering at the borderline between science and art means that risks are often more difficult to foresee and quantify than other areas of civil engineering, and the discipline does not lend itself to highly detailed codes of practice.

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Chapter 2

Foundations and other geotechnical elements in context – their role

John B. Burland Imperial College London, UK
Tim Chapman Arup, London, UK

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The purpose of this chapter is to describe the basic principles of geotechnical design and construction in the context of the whole project. At a given site the ground conditions have resulted from millions of years of natural geological processes (which are seldom simple) and have sometimes been modified by humans, e.g. by mining or many other processes. As a consequence there are always inherent uncertainties and risks, and the art of geotechnical engineering is to make informed allowance for these, in both design and construction. The key requirements for all geotechnical elements of a project are described. Emphasis is placed on the importance of constructive and positive interaction with professionals engaged in the many other contributing disciplines. The design life of the geotechnical elements is considered and the important concept of the geotechnical design and construction cycle is introduced. Various managerial approaches to identifying the key elements of the complete design and construction process are described. The chapter concludes with a summary of the factors common to most geotechnical design and construction projects that are necessary for a successful outcome.

2.1 Geotechnical elements in the context of the rest of the whole structure

All built structures touch the ground in some way and hence all need some form of foundation. Other geotechnical elements include retaining walls and ground anchors. Sometimes they can be shallow, e.g. pad footings or gravity retaining walls; other times they are deep, such as piles or embedded retaining walls. Often they rely on geotechnical processes such as ground improvement to produce a geotechnical element.

All foundations and other geotechnical elements have a number of characteristics that distinguish them from other parts of the structures that they support:

- they tend to be amongst the most heavily loaded elements in any structure;
- their installation process is less amenable to factory-style production;
- their capacity is very dependent on the ground of the site, which is always characterised by few observations and tests, and is normally very heterogeneous and may contain hazards that are difficult to foresee;
- their capacity is strongly influenced by the method of construction and how well it is controlled.

Hence, the risk of failure tends to be significantly higher than that for other parts of the structure. The management of ground uncertainty is an important part of the design and construction process in order to produce elements that have the required degree of reliability.

A vital consideration in geotechnical design is the interaction of the structural element that is inserted into the ground with the ground itself – so-called 'soil–structure interaction'. Structural loads are applied to the element and the ground resists – generally either by friction along the element, or by bearing of the element against the ground. Both these resistances can occur vertically or horizontally, as shown in **Figure 2.1**. Normally ground stresses are maintained within failure limits, so the resulting displacements depend on the stiffness of both the element and the ground.

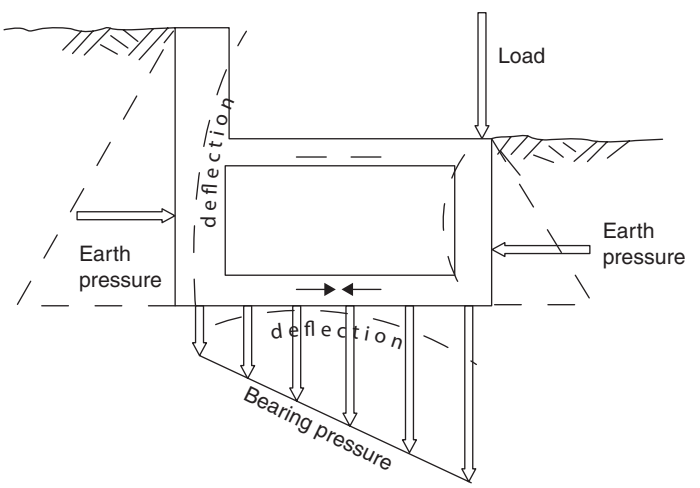


Figure 2.1 Soil–structure interaction

Loads can be imparted into foundations directly, as vertical or horizontal loads, or as imposed bending moments. They can also be imposed indirectly as displacements, which induce loads into the structural parts of the foundation. Eurocode 7 (BS EN 1997) (British Standards Institution, 2004, 2007b) treats these in a similar way by introducing the Newtonian concept of an 'action'.

Geotechnical elements are characterised by much higher degrees of uncertainty than other structural elements. The uncertainty is derived from the following:

- The inevitable significant assumptions and idealisations that underlie the overall geotechnical design.
- Natural variability in the ground being modelled, often manifested by vast scatter in data points on graphs.
- Inherent variability in the construction processes, and their control, that are used to install the geotechnical elements and foundations, which may also have a profound effect on their eventual performance in service.
- Usual variability in the applied loads transmitted through a normally highly structurally indeterminate construction meaning that the actual loads on each foundation may be very uncertain.
- There is always the possibility of the construction and performance being heavily influenced by an unexpected feature of a site, for example a major geological discontinuity such as a sinkhole or a fault. It is not unusual for major construction problems to arise owing to adverse groundwater conditions.

All of these uncertainties mean that engineers who presume that their calculations are precise and predictable are deluded and may expose themselves to much more onerous load combinations than they could imagine. The wise geotechnical engineer should be humble and make prudent allowances for uncertainty. This is explained in more detail in Chapter 7 *Geotechnical risks and their context for the whole project*.

2.2 Key requirements for all geotechnical elements

2.2.1 General

All foundations or other geotechnical elements must fulfil a number of essential criteria as listed below.

- They must not fail, or else the structure they support will also fail. In terms of limit state design, failure by any mode is termed reaching or exceeding an 'ultimate' limit state, and may involve failure of a structural element or rupture along a soil–structure or a soil–soil interface.
- They must not move excessively or else the structure they support may become impaired or fail to operate as intended. In terms of limit state design, excessive deflection involves breaching a 'serviceability' limit state.
- They must last for as long as intended. Unlike many other building elements, foundations are hugely difficult to upgrade or repair and so their longevity will often dictate the life of the structure that they support.

2.2.2 Ultimate limit state modes of failure

There is a range of ways in which geotechnical structures can fail an ultimate limit state, and these are shown in **Figure 2.2**.

2.2.3 Serviceability limit state and displacements

Failure of a serviceability limit state is usually less serious than failure of an ultimate limit state and is often repairable. It usually occurs when excessive displacements have taken place that impair the function of the structure. In addition to excessive movement, it also includes other forms of unacceptable tolerances, such as moisture penetration into basements. Examples are illustrated in **Figure 2.2**.

2.2.4 Design life and modes of deterioration

Design lives are covered in more detail in section 2.4. Other than failures of a limit state, the life of a structure can be reached when:

- it has become affected by material deterioration processes, such as corrosion of steel, carbonation of concrete or rotting or insect infestation of timber;
- it has been subjected to physical processes, such as repeated loading cycles causing fatigue, or excessive damage from impacts;
- it fails to meet new design or material standards and so offers a less than acceptable level of resistance against load, corrosion, etc.

2.3 Interaction with other professionals

2.3.1 General

The geotechnical engineer is seldom the professional solely in charge of a complete project – almost invariably the function sought of a new structure or facility goes beyond merely geotechnical considerations. Therefore, the role of a geotechnical engineer should be to support the wider design and construction process. He or she is likely to have most influence if the

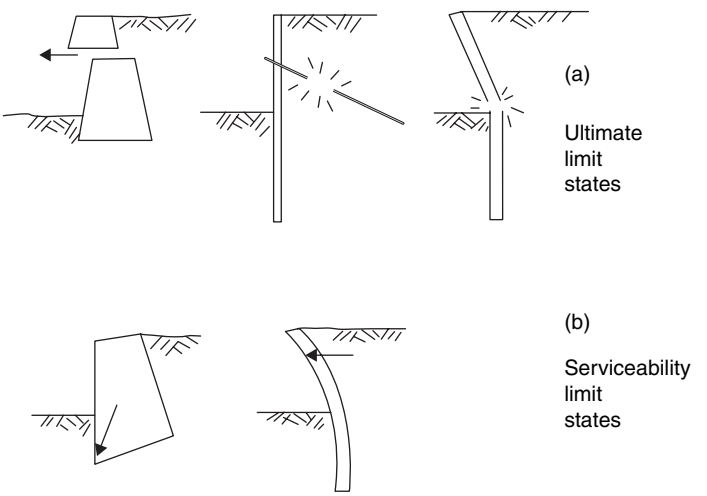


Figure 2.2 Limit state failures (BS EN 1997) (British Standards Institution, 2004, 2007b)

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input complements the work of other professionals rather than seeking to dominate or failing to engage.

2.3.2 General construction process

All project programmes involve controlled phases of input so that the optimum solution is devised with mutually consistent input from all members of the project team, through design procurement, implementation and commissioning. Once commissioned there is a need for maintenance, upgrades and repairs through the working life of the structure and finally decommissioning and sometimes re-use of the original elements in a fresh structure.

Client Best Practice Guide (Institution of Civil Engineers, 2009) provides a generalised approach for such inputs, based on the systems used for buildings (Royal Institute of British Architects, 2009) and the rail industry (Network Rail, 2010; previously known as *Guide to Railway Investment Projects*), as well as guidance from the UK's former Office of Government Commerce (OGC) (now part of the Cabinet Office Efficiency and Reform Group). More detail on these processes is given in section 2.5.

In the early stages, there will be little certainty about the optimum solution and many competing options will be considered and dismissed, often for reasons not obvious to individual members of the technical team. The design team can contribute best to this process if they understand the required function and client's business drivers for the new structure or facility.

The early stages are also the period when it is easiest for the project to take on board geotechnical constraints and design and construction requirements. Hence, the geotechnical engineer is likely to have most success in integrating these requirements into the rest of the structure if they are identified early on, although this will often be in advance of the acquisition of data and completion of analyses. Therefore, experienced geotechnical input is valuable in these early stages and will often lead to a better solution, simpler design and construction processes, and less costly subsequent geotechnical involvement.

The work of the geotechnical engineer is seldom complete in isolation. Normally there is much interaction with structural engineers. Sometimes the work of the geotechnical engineer is subsumed beneath that of the structural engineer. For simple structures, this may be appropriate and sufficient, but for more complex geotechnical structures, or where the ground conditions are complex, it is likely that the geotechnical engineer will be better able to explain the particular issues directly to other members of the design team.

2.4 Design lives for geotechnical elements

Relevant design lives for structures are defined in Eurocode 0 (BS EN 1990) *Basis of Structural Design* (British Standards Institution, 2005) and are generally distinguished according to whether the structure is a building or a piece of infrastructure. In its Table 2.1, it defines design life categories as follows:

- Category 1 – Temporary structures, not including structures or parts of structures that can be dismantled with a view to being re-used – 10 years.
- Category 2 – Replaceable structural parts, e.g. gantry girders, bearings – 10–25 years (The UK National Annex to BS EN 1990:2002 modifies this to 10–30 years).
- Category 3 – Agricultural and similar buildings – 15–30 years (modified in the UK to 15–25 years).
- Category 4 – Building structures and other common structures – 50 years.
- Category 5 – Monumental building structures, bridges and other civil engineering structures – 100 years (currently modified in the UK to 120 years to bring it into line with traditional Highways Agency bridge design life; it is possible that the UK may revert back to 100 years if the Highways Agency changes their guidance).

As foundations are difficult to repair or upgrade and as most structures last for longer than the period for which they were designed, allowing for a longer life is prudent.

Foundations are difficult and expensive to remove and so some consideration should be given to what will occur after the life expiry of the structure they support. This was addressed by the RuFUS (Reuse of Foundations on Urban Sites) project (see Butcher *et al*. (2006) and Chapman *et al*. (2007)). Where it is likely that a new structure with new foundations could become a major obstruction, the potential for future development of the site may be inhibited.

To prevent abandoned foundations from becoming an insidious form of ground contamination, the RuFUS project advocated that all foundations should be designed with the intention of allowing subsequent re-use. This mainly relates to the recording and saving of records so that the old foundations can be assessed by the future design team.

While most foundation and geotechnical elements are provided for 'permanent' elements that are required to persist for a normal structural life, sometimes foundations are required for shorter periods. These include:

- Interim structures, required for a significant life, perhaps 10 years.
- Contractor's 'temporary' works – structures that fulfil a temporary function during construction of a more significant structure, e.g. thrust blocks, crane bases. Sometimes a life of 1–2 years is specified. For these, Eurocode 7 Part 1 (BS EN 1997-1:2004; British Standards Institution, 2004) Clause 2.4.7.1 (5) states 'Less severe values than those recommended in Annex A [for partial factors] may be used for temporary structures or transient design situations, where the likely consequences justify it'.
- Demountable structures, such as scaffolding or temporary stands as may be required at sport or music venues, where a life of only perhaps weeks is required. The Institution of Structural Engineers (2007) guide, *Temporary Demountable Structures*, defines typical foundation concerns for such structures.

Design life can influence choice of factor of safety. However, it needs to be considered with the following points.

- The frequency with which the most onerous design load combination occurs – if it is very infrequent compared with the design life, some reduction in margin against failure may be possible.

This may occur with 1-in-100-year wind or flood events or with 1-in-475-year seismic events.

- The consequences of failure – where the consequences are mild and do not threaten safety, then a lower factor may be permissible provided the economic consequences are judged and agreed as acceptable.

The design life also to some extent governs the measures required to limit deterioration of the foundation materials. Conventional structural design codes such as Eurocodes 2 (BS EN 1992; British Standards Institution, 2006) and 3 (BS EN 1993; British Standards Institution, 2007a) contain implicit requirements to ensure longevity of foundations; principally, crack width criteria for reinforced concrete, intended to limit the intrusion of air and water which then can come into contact with the reinforcing steel, and whose effect can be exacerbated by the presence of chloride ions from salt. Where potential corrosion processes will be slow compared with the foundation design life, it may be possible to relax these structural requirements for structures intended to have short lives, provided that code non-compliance is acceptable to the owner and any approver.

Temporary structures may sometimes have less onerous movement limits. Their nature may mean that larger movements are easy to accommodate; for instance, in temporary stands where shims and jacks can be used to compensate for differential foundation movements or where there are no rigid finishes that make obvious the effects of differential settlements.

2.5 The geotechnical design and construction cycle

Geotechnical design should be carried out in conjunction with the design of the whole structure, as explained in section 2.1. Geotechnical design must always be carried out considering the source of the data and how the design will be implemented, with all three parts of this process being inextricably linked as illustrated in **Figure 2.3**.

2.5.1 Project phases

All construction projects go through a number of distinct phases:

- planning;
- development;
- implementation;
- operation;
- decommissioning.

A number of organisations have devised 'stages' (e.g. RIBA, 2009) or 'gateways' (Office of Government Commerce, UK) and have further sub-divided projects to suit these. Those work stages identified by RIBA (A to L, see **Figure 2.4**) as most commonly used in general design and construction are used here. Various stages/divisions are shown in **Figure 2.4**. The types of input needed at each stage are shown in **Figure 2.5**.

The modes of input needed at each stage are different. During the *planning* phase, input tends to be strategic and predominantly consultative – usually senior experienced input is needed to guide the project away from the more common pitfalls or significant hazards. By the end of this phase, the design should have progressed close to the stage of a single preferred option having been identified.

In the *development* phase, the design issues are resolved and all disciplines should produce a single coherent and coordinated output. In the latter stages, tender documents are produced which clearly describe the works to be undertaken on site. A design freeze is needed relatively early in this phase

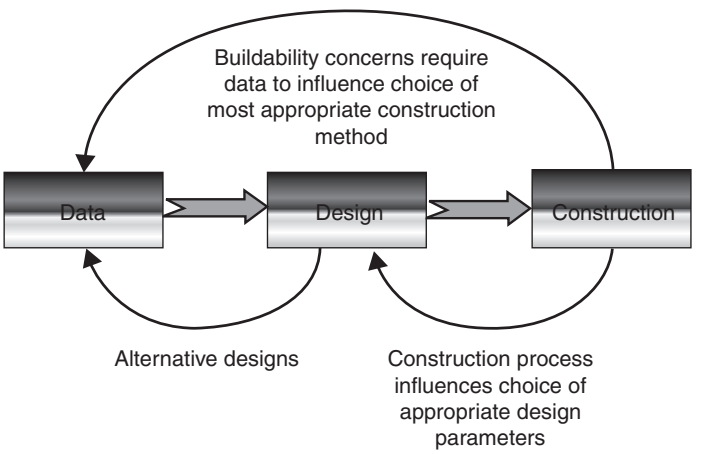


Figure 2.3 The geotechnical design and construction cycle

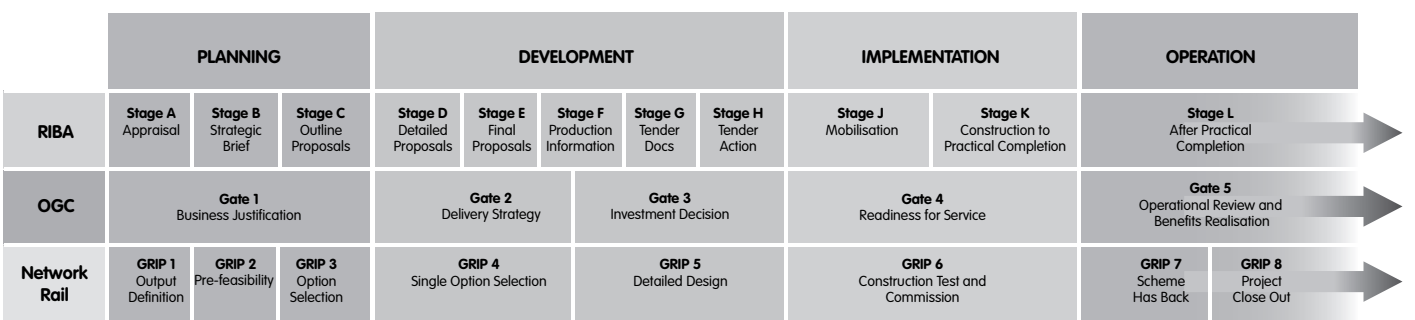


Figure 2.4 The work stages, *RIBA, OGC and NWR*
Reproduced from Institution of Civil Engineers (2009)

(end of RIBA Stage E 'final proposals') so that in subsequent stages coherent documents for tender and construction can be produced. The importance of a proper design freeze cannot be underestimated, as continued changes to the design jeopardise the production of an integrated and mutually consistent set of contract documents.

2.5.2 Importance of clarity in tender process

In RIBA stages F 'product information' to H 'tender action', the tender documents are produced and the contract is let. In these stages, the client becomes committed to investing in the whole of the project; therefore, at this stage clients need to be confident that:

- the investment decision criteria remain valid;
- they have funding;
- they are content that the project has been organised to run smoothly.

This is a key time for geotechnical input; many well-designed projects rush production of the tender documents and do not adequately communicate:

- the design intent and constraints;
- the inherent risks and means for allocating responsibility for those (including those under Construction (Design and Management) Regulations 2007).

2.5.3 Construction control

Unusual geotechnical construction processes need knowledgeable control, both for the contractor and for the client, the latter sometimes provided by a resident engineer or individual with similar technical understanding.

The observational method (see Nicholson *et al.*, 1999 (CIRIA R185) and Chapter 100 *Observational method*) can be an excellent way to build efficiently. Use of the observational method can be used to create something that conventional factors of safety may rule out. However, there must be well-founded evidence and experience that those factors would be inappropriately conservative. The observational method has been used most successfully for deep excavation in stiff clays where stability is controlled by slow dissipation of pore pressures, something that is difficult to rely on, but which can be monitored and managed by a well-trained and dedicated team.

2.6 Common factors associated with geotechnical success

The following factors are associated with successful projects and, conversely, absence of these factors is correlated with an increased frequency of failures.

- **Planning**
- good desk study to identify key hazards appropriate for the planned development – no skimping, e.g. historical map coverage extends back to first significant site occupation;
- gather constraint/hazard data early and present them clearly;
- address the likely construction issues in the ground investigation.
- **Development**
- beware trimming of important parts of the design service to save money, particularly the omission of knowledgeable input;
- choose foundation type to explicitly reduce risks, e.g. avoid bored piles in silty soil;

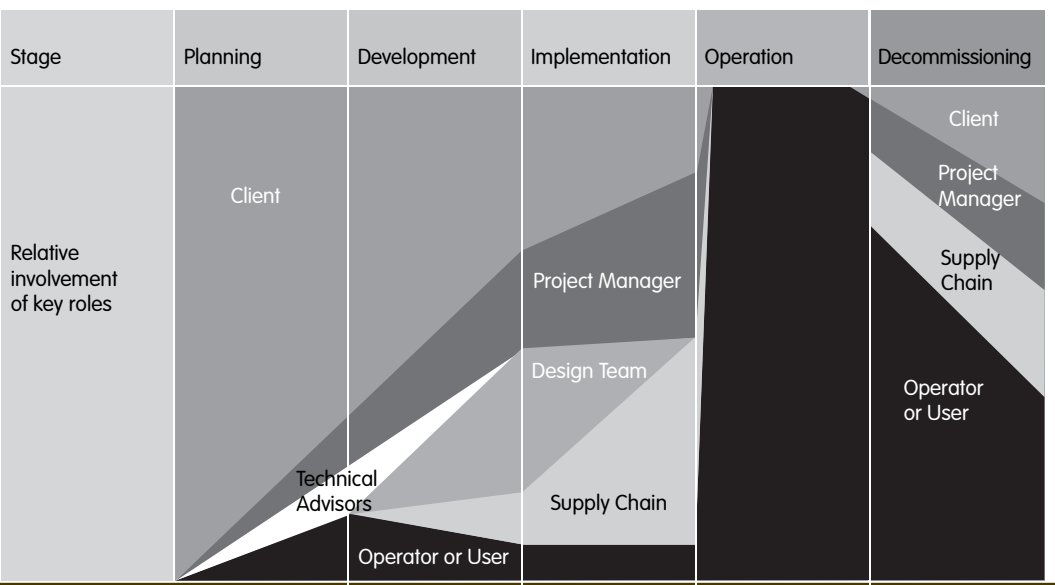


Figure 2.5 Input required at each stage of a project's lifecycle
Reproduced from Institution of Civil Engineers (2009) (after Martin Barnes)

- consider buildability as a critical element of the design process;
- ensure continuity of process ideally by the same capable individuals;
- be clear on responsibilities, especially at gaps where responsibilities are divided or can be confused;
- beware of designs based on optimistic assessment of parameters based on inappropriate extrapolation of test results;
- bring experience to bear early to identify the likely key design issues that affect other members of the design team, for instance space-proofing with the architect and structural engineer;
- watch water-proofing measures for basements, a frequent source of client disappointment;
- ensure designers understand key assumptions from the analysis process and that contractors understand key assumptions from the design process.
- **Implementation**
- produce clear specifications highlighting important issues and communicating the design intent.

In general, beware overconfidence and adopt humility – consider the occurrence of the unexpected and don't rely on everything happening as planned. There is the well known story told of the famous tunnelling engineer Sir Harold Harding who, on being informed of an overconfident young graduate in the design office, growled 'Bring him to me and I will put him down a big hole in the ground and teach him humility'.

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Royal Institute of British Architects (2009). *RIBA Outline Plan of Work 2007* [(Updated): including corrigenda issued January 2009]. London: RIBA Publishing.

2.7.1 Useful websites

Efficiency and Reform Group, Cabinet Office, UK; www.cabinetoffice.gov.uk/government-efficiency

Network Rail, *The GRIP Process*; www.networkrail.co.uk/aspx/4171.aspx

Royal Institute of British Architects (RIBA); www.architecture.com

All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

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Chapter 3

A brief history of the development of geotechnical engineering

John B. Burland Imperial College London, UK

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3.3	Terzaghi, father of geotechnical engineering	12
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This chapter traces the development of the craft and science of foundation engineering from early history to the recent past. The story is told of how Terzaghi struggled to couple engineering geology with the science of soil mechanics so as to provide the necessary rigour for modern geotechnical modelling and analysis. These lessons from the past are very important and a major aim of this manual is to provide the framework and knowledge necessary for sound geotechnical design and construction.

3.1 Introduction

Foundation engineering is as old as the art of building and, like building, it developed largely on the basis of accumulated experience and empirical procedures. Because ground conditions vary so much from one locality to another, foundation practice varied widely. Moreover, the extrapolation of experience from one locality to another was fraught with uncertainty.

Parry (2004) describes how the ancient Egyptians learned from the foundation failure of the South Dahshur Pyramid. This was built during the reign of Pharaoh Snofru (2575–2551 BC) using a central core supported by a series of inclined buttress walls, as had become the tradition. The pyramid was founded on a clay layer. Large differential foundation movements resulted in significant structural distress in the tomb chambers and their access passages. As a result the pyramid was finished off so that the upper portions have a considerably reduced slope, giving rise to the structure being named the 'Bent Pyramid'. Abandoning the concept of buttress walls, future pyramids were built tier by tier, each tier consisting of a single layer of blocks of uniform thickness across the full width of the structure. Greater care was also taken in dressing and placing the blocks.

Kerisel (1987) describes how in Mesopotamia, over a period of three millennia, the art of building ziggurats was developed, in most cases on very weak alluvial soils. Because of the scarcity of stone these high massive structures were made of sun-baked bricks laid out in successive courses. As construction proceeded the underlying alluvium soon yielded under the weight causing the base to spread laterally. Work progressed very slowly with long pauses in between so that little by little the rate of settlement and spreading diminished. Eventually it was possible to build a small temple at the top (**Figure 3.1**). Around 2100 BC the Sumerians began to place thick layers of woven reeds between every six to eight courses of sun-dried brickwork. In this way the horizontal tensions caused by the tendency of the foundations to spread were resisted. As a result ziggurats could be built with nearly sheer sides and massive temples on top. This innovation is often cited as the earliest example of reinforced earth.

Two early examples of successful foundation engineering in China are given by Kerisel (1987). The elegant early 7th century Zhaozhou (otherwise known as Anji) Bridge (**Figures 3.2** and **3.3**) is founded on clay, which was treated by digging it out beneath the abutments and recompacting it in layers interspersed with compacted layers of broken bricks. The late 10th century 44 m high Pagoda of Longhua is founded on a thick layer of soft clay extending to a depth of about 30 m. The foundations are of brick laid on a wooden raft, which in turn rests on wooden piles driven at very close spacing – perhaps one of the earliest examples of a piled raft. The foundations remain unchanged since they were constructed over 1000 years ago.

3.2 Geotechnical engineering in the early 20th century

It is not widely appreciated what a parlous state ground engineering was in, prior to Terzaghi's contributions. Recently, as part of its centenary celebrations, the author was given the interesting task of tracing the development of foundation engineering over the last 100 years through the papers published in *The Structural Engineer* (Burland, 2008). Many of the early papers describe various techniques of foundation construction such as piles, sheet pile wall sections, coffer dams and caissons. But these papers make little reference to the mechanical properties of the ground and how its response can be assessed. For example Brooke-Bradley (1932–34) states that:

> If the bearing power of sub-soil should prove to be inadequate to carry the proposed loads, it must be artificially strengthened.

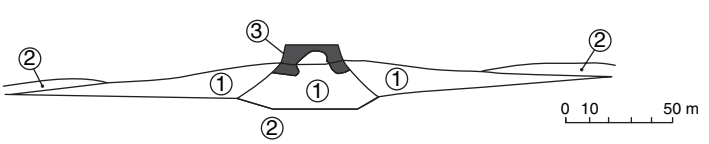


Figure 3.1 Construction of an early ziggurat. (1) Fill, (2) soft alluvium, (3) temenos – sacred enclosure
Reproduced from Kerisel (1987); Taylor & Francis Group

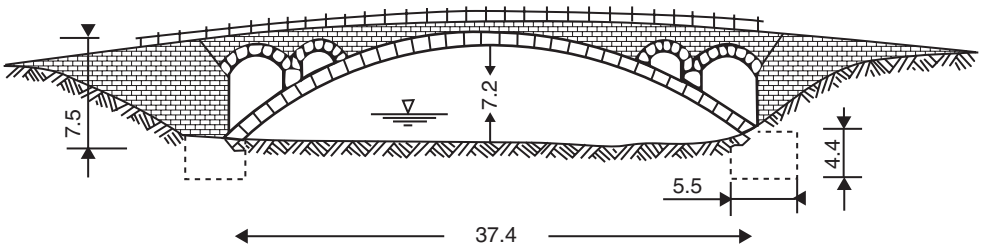


Figure 3.2 Zhaozhou Bridge – early 7th century
Reproduced from Kerisel (1987); Taylor & Francis Group



Figure 3.3 Zhaozhou Bridge

Methods of doing this are then described together with the various types of piles available for this purpose. Nowhere does one find how the 'bearing power' of the ground can be assessed in the first place. It is also stated that 'all settlement should be avoided if possible'; examples are given of damaging settlement but no guidance is given on how it could be estimated.

In the early issues of *The Structural Engineer* some space is given to the design and construction of retaining walls. In 1915 Wentworth-Shields wrote a paper on 'The stability of quay walls on earth foundations' (Wentworth-Shields, 1915). He opens with the following memorable statement:

> In spite of the large amount of experience which has been gained in the construction of quay walls, it is still one of the most difficult problems in engineering to design a wall on an earth foundation with confidence that it will be stable when completed. … Even if the designer of such a wall is assured that it will stand, he cannot with any confidence tell you what factor of safety it possesses.

In 1928 Moncrieff published a major paper in *The Structural Engineer* (Moncrieff, 1928) on earth pressure theories in relation to engineering practice. He summarises the various approaches to calculating earth pressures from Coulomb (1773) through to Bell (1915). At that time the angle of friction was generally equated with the angle of repose and Moncrieff refers to the difficulty of determining this angle for clayey soils. He cites a cutting in clay in which the side slopes varied from vertical to 1 vertical in 1½ horizontal while in places the clay was 'running down like porridge'.

It is all too clear from these early papers that, in spite of significant, even heroic, engineering achievements in the construction of major foundations, retaining structures, tunnels and dams, there was little understanding of the factors that control the mechanical behaviour of soil in terms of its strength and stiffness. Moreover, there is almost no reference to the influence of ground water on strength, stability or earth pressures. It is hardly surprising that there were frequent failures, particularly of slopes and retaining walls. This was the muddle that Terzaghi found when he first began to practise as a civil engineer.

3.3 Terzaghi, father of geotechnical engineering

Because of his work in developing the scientific and theoretical framework of soil mechanics and foundation engineering, Terzaghi is often regarded as essentially a theoretician. Nothing could be further from the truth. It is, therefore, worth reflecting on Terzaghi's struggles to develop the craft and the science of ground engineering for they have relevance in both the teaching and the practice of the discipline.

Goodman (1999) has written a most illuminating and thoroughly researched narrative of Terzaghi's life, *Engineer as Artist*. Terzaghi was born in Prague in 1883. He showed an early interest in geography, especially field exploration, and later astronomy. which evolved into a passion for mathematics. Later at school he was inspired by the natural sciences and performed brilliantly.

3.3.1 Terzaghi's education

He went on to read mechanical engineering at the Technical University of Graz. For a time he lost his way, engaging in drinking and duelling. He found the lectures were simply a set of prescriptions, which he claimed he could read for himself. Ferdinand Wittenbauer, a wise teacher, challenged Terzaghi to do better and go back to the original sources – in particular Lagrange's *Analytical Mechanics* (Lagrange, 2001).

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Wittenbauer led Terzaghi gently on, guiding him not only into the excitement of scientific creativity but also in the very real social and cultural issues of the day. It was Wittenbauer who saved Terzaghi from being expelled after an over-exuberant student prank. Wittenbauer pointed out to the authorities that in the history of the university there had been only three expulsions: Tesla, who went on to revolutionise electrical technology, Riegler, who created the steam turbine, and a third who developed into a leading church architect. He went on to point out that the university was not good at choosing candidates for expulsion. Terzaghi was reprieved!

Though reading mechanical engineering, Terzaghi attended courses in geology. He was keen on climbing and it is related that he made every climbing expedition into a joyous adventure in field geology. During his compulsory year of military service he translated the *Outlines of Field-Geology* by Archibald Geikie (director of the British Geological Survey) into German. In a second edition, he actually extended it to a fuller coverage of karst features and the geomorphology of glaciated country, replacing the English examples with Austrian ones.

3.3.2 The switch to civil engineering

Terzaghi's interest in geology persuaded him that mechanical engineering was not for him. Switching to civil engineering, he returned to Graz for an extra year. He went to work for a firm specialising in hydroelectric power generation. Although his main activity was in the design of reinforced concrete, the planning of the structures was, of course, intimately involved with geology. But frequently he found the guidance of expert geologists unhelpful. He encountered many cases of failure. Significantly these were mainly due to the lack of ability to predict and control groundwater – piping failures were abundant. He also encountered many slope failures, bearing-capacity failures and structures undergoing excessive settlement.

3.3.3 Geology on its own

Recognising the difficulties that civil engineers experienced in dealing with the ground and also the obvious influence of geological factors, he concluded that it was necessary to collect as many case records as possible so as to correlate failures with geological conditions. It is well known that he then spent two intense years (1912–1914) in the western United States observing and recording. Two years that ended in disillusion and depression. The following quote from his presidential address to the 4th International Conference on Soil Mechanics and Foundation Engineering sums up his mood at that time (Terzaghi, 1957):

> At the end of the two years I took my bulky collection of data back to Europe, but when I started separating the wheat from the chaff I realised with dismay that there was practically no wheat. The net result of two years of hard labour was so disappointing that it was not even worth publishing it.

So much for geology on its own! So much for precedent and case histories on their own!

To quote Goodman (1999), the problem lay in the fact that:

> … the names geologists give to different rocks and sediments have developed mainly from a scientific curiosity about the geologic origin of these materials, whereas Terzaghi was aiming towards discerning the differences in their engineering properties.

This is still true today – the engineer needs to understand the key geotechnical properties that affect the response of the ground.

3.3.4 The birth of the science of soil mechanics

Shortly after his appointment to the Royal Ottoman Engineering University in Constantinople in 1916, Terzaghi began to search the literature for insights into the mechanical behaviour of the ground. He became increasingly frustrated. What he witnessed was a steady decline from 1880 in recorded observations and descriptions of behaviour. This was replaced by myriads of theories postulated and published without adequate supporting evidence. This experience must have been uppermost in his mind when, in his presidential address to the 1st International Conference on Soil Mechanics and Foundation Engineering, he stated the following (Terzaghi, 1936):

> In pure science a very sharp distinction is made between hypothesis, theories, and laws. The difference between these three categories resides exclusively in the weight of sustaining evidence. On the other hand, in foundation and earthwork engineering, everything is called a theory after it appears in print, and if the theory finds its way into a text book, many readers are inclined to consider it a law.

Thus, Terzaghi was emphasising the enormous importance of assembling and examining factual evidence to support empirical procedures. He also brings out the importance of instilling rigour. This is often equated with mathematics but there is at least as much rigour in observing and recording physical phenomena, developing logical argument and setting these out on paper clearly and precisely.

In 1918 Terzaghi began to carry out experiments on forces against retaining walls. He then moved on to piping phenomena and the flow beneath embankment dams. He used Forchheimer's flownet construction to analyse his observations and apply them in practice – methods that were themselves adapted from the flow of electricity. We see here the interplay between experiment and analytical modelling.

Over this period Terzaghi came to realise that geology could not become a reliable and helpful tool for engineers unless and until the mechanical behaviour of the ground could be quantified – this required systematic experimentation. On a day in March 1919, and on a single sheet of paper, he wrote down a list of experiments that would have to be performed.

Terzaghi then entered an intense period of experimental work in which he carried out oedometer (confined compression) tests and shear tests on clays and sands, thereby developing his physical understanding of the principle of effective stress

(the cornerstone of soil mechanics), excess pore water pressures and the time-rate of consolidation – this was the birth of soil mechanics. To make headway with modelling the consolidation phenomenon analytically he turned to the mathematics of heat conduction. Again we see here the interplay between experiment and analytical modelling. This intense period of experimental work and theoretical modelling culminated in the publication of his seminal book *Erdbaumechanik* (Terzaghi, 1925).

3.4 The impact of soil mechanics on structural and civil engineering

In 1933, a Soil Physics Section was established at the Building Research Station (BRS) in the UK, and Dr Leonard Cooling was put in charge of it. He set up the first proper soil mechanics laboratory in Britain, equipped with the apparatus necessary to classify soils, measure their basic mechanical properties and carry out sampling. By 1935 the first investigations of civil engineering problems had begun and the group moved to the Engineering Division of BRS and was renamed the Soil Mechanics Section. It was in August 1937 that the well-known Chingford embankment dam failure occurred and the team from BRS carried out the investigation. Terzaghi was called in to redesign the embankment, and the necessary testing and analysis was carried out at BRS. This gave great impetus to the acceptance of soil mechanics as a key discipline in civil engineering in the UK.

On 6 December 1934, Terzaghi delivered a lecture before the Institution of Structural Engineers in London with the title 'The actual factor of safety in foundations' (Terzaghi, 1935). He illustrated his lecture with a large number of case histories of measured distributions of settlement across buildings and their variation with time. He was able to explain the broad features of behaviour using the basic principles of soil mechanics and foundation analysis, demonstrating how vital it is to establish the soil profile with depth and across the plan area of the building. Even so, he showed that local variations in soil properties and stratification make it impossible to predict the settlement patterns with any precision. Without actually using the term, he drew attention to the important concept of ground–structure interaction, pointing out that the structure of a building should not be treated in isolation from its foundations. He even drew attention to the fact that reinforced concrete beams can yield plastically without impairing the stability or appearance of a frame building, provided the cracking is not excessive. It is of interest to note that, in their seminal paper on the allowable settlement of buildings, Skempton and MacDonald (1956) drew extensively on the case histories provided by Terzaghi in this lecture.

Towards the end of his lecture he made the following important assertion:

> Experience alone leads to a mass of incoherent facts. But theory alone is equally worthless in the field of foundation engineering, because there are too many factors whose relative importance can be learned only from experience.

On 2 May 1939, Terzaghi delivered the 45th James Forrest Lecture at the Institution of Civil Engineers, London with the title 'Soil mechanics – a new chapter in engineering science' (Terzaghi, 1939). The lecture summarised in simple terms the basic elements of the discipline of soil mechanics and its application to a number of engineering problems ranging from earth pressure against retaining walls and the failure of earth dams due to piping through to the phenomenon of consolidation and the settlement of foundations. Early in the lecture Terzaghi made the memorable statement that:

> … in engineering practice difficulties with soils are almost exclusively due, not to the soils themselves, but to the water contained in their voids. On a planet without any water there would be no need for soil mechanics.

He was a forceful and charismatic figure and this lecture made a very profound impact on the structural and civil engineers in the UK. The late Peter Dunican, past president of the Institution of Structural Engineers, attended as a young man and told the author of how Terzaghi had electrified the audience. Many leading geotechnical engineers, including the late Sir Alec Skempton, stress what a pivotal role this lecture played in the development of soil mechanics in the UK. As with his earlier lecture to the Institution of Structural Engineers, Terzaghi emphasised very strongly the importance of retaining a balance between theory and practice in soil mechanics. He stressed most strongly that precision of prediction was not possible due to the inherent variability of the ground and construction processes.

It is clear that Terzaghi is very much more than the father of the science of soil mechanics. His contribution was to place ground engineering on a rational basis, with geology as a key supporting discipline and soil mechanics providing the scientific framework for understanding the mechanical response of the ground. He is indeed the father of geotechnical engineering, which embraces engineering geology, soil mechanics and arguably rock mechanics as well.

3.5 Conclusions

Terzaghi's development of the science and art of geotechnical engineering grew out of his experiences as a civil engineer and his gradual realisation that the underlying principles governing the mechanical properties of soil were not understood. Although his contributions are often regarded as primarily theoretical, in reality this is anything but the case. A close study of his work reveals a brilliant and passionate engineer who at all times tried to maintain a balance between underlying theoretical principles, practical experience and the handling of the uncertainties that are always present when dealing with the ground in its natural state.

It is hoped that this chapter will provide a helpful summary that puts into context Terzaghi's struggles to provide a scientific and rigorous basis for geotechnical engineering. It demonstrates his grounding in geology; the importance of gaining an understanding of the mechanical behaviour of the ground and

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Figure 3.4 Karl von Terzaghi
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groundwater by means of experiment and testing; the need to develop an analytical framework for predictive purposes and, very importantly, the key role that experience plays and the importance of case histories. Time and time again he insisted that soil mechanics is not a precise science because of the inherent variability of the ground and the uncertainty of many factors associated with construction.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

Chapter 4

The geotechnical triangle

John B. Burland Imperial College London, UK

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This chapter outlines a coherent approach to the key aspects of geotechnical engineering by making use of a simple aide-memoire termed the *geotechnical triangle.* The distinct and rigorous activities undertaken to obtain (1) the *ground profile*, (2) the *measured behaviour* of the ground and (3) an *appropriate model* are represented as the apexes of an equilateral triangle with empirical procedures and '*well-winnowed experience*' linked to all three at the centre of the triangle. The *ground profile* is placed at the top apex of the triangle because of its crucial importance and *experience* is located at the centre of the triangle because it is an essential aspect of all geotechnical engineering and relates to the other three activities. Each of the above activities has a distinct methodology, each has its own rigour and each is interlinked with the other. Successful ground engineering requires that each activity is properly considered so that a coherent approach is adopted and the triangle is 'kept in balance'. The application of the *geotechnical triangle* is illustrated by re-visiting the well-known case history of the underground car park at the Palace of Westminster in London.

4.1 Introduction

Geotechnics is a difficult subject and is regarded by many engineers as a kind of black art. The author used to think that this was due to the nature of the ground, which is a two- or even three-phase material. It is true that it is much more complex than the more classical structural materials of steel, concrete and even timber with which most engineers are familiar. As explained in Chapter 14 *Soils as particulate materials*, soil is a particulate material with little or no bonding between the particles. As a consequence:

- The stiffness and strength of a given soil is not fixed but depends on the confining pressure.
- When it is deformed, soil will tend to contract or dilate depending on how dense it is, and this can profoundly influence its properties.
- During shearing, the particles tend to change their orientation, which, in turn, has a big influence on the shearing resistance of the material.
- Most important of all – water pressures acting within the pores of the soil are just as important as the applied boundary stresses.

This complex particulate material has to be modelled as if it were a continuum but it is essential never to forget that in reality it is particulate.

Difficult as soil is as an engineering material, the problems confronted by the engineer in tackling a ground engineering problem are more subtle. Chapter 3 *History of geotechnical engineering* describes the struggles experienced by Terzaghi in establishing geotechnical engineering as a valid engineering discipline. After a careful study of the opinions expressed by Terzaghi and others, and from his own experience, the author came to the view that the main problem confronting the engineer is not so much the complex nature of soil as a material (difficult as that is) but a lack of appreciation of the number of aspects that have to be considered in tackling a ground engineering problem (Burland, 1987).

A close study of Terzaghi's struggles towards establishing the subject reveals that there are three distinct but interlinked aspects that have to be considered in tackling any ground engineering problem:

- the ground profile including groundwater conditions;
- the measured behaviour of the ground;
- the appropriate model for assessing and predicting performance.

All three of these aspects are supported by empirical procedures, judgement based on case histories and what the author has termed 'well-winnowed experience'.

The boundaries between these aspects often become confused. For example, it is frequently not clear whether a particular design approach is based on analysis or is mainly empirical. Moreover, one or more of the aspects is frequently completely neglected. The first three aspects may be depicted as forming the apexes of an equilateral triangle, with empiricism and experience occupying the centre and linked to the apexes as shown in **Figure 4.1**. This representation was originally developed as a teaching aid but it has since become apparent that it also provides a valuable aide-memoire for geotechnical practice.

Associated with each of the above aspects are distinct and rigorous activities and procedures, which are shown in **Figure 4.1** by the broken circles. Each activity is a major discipline in its own right. Successful ground engineering requires that all aspects of the geotechnical triangle should be considered. Excessive reliance on one aspect (e.g. computer modelling or empirical procedures) can be disastrous. The four aspects depicted in the geotechnical triangle and their associated

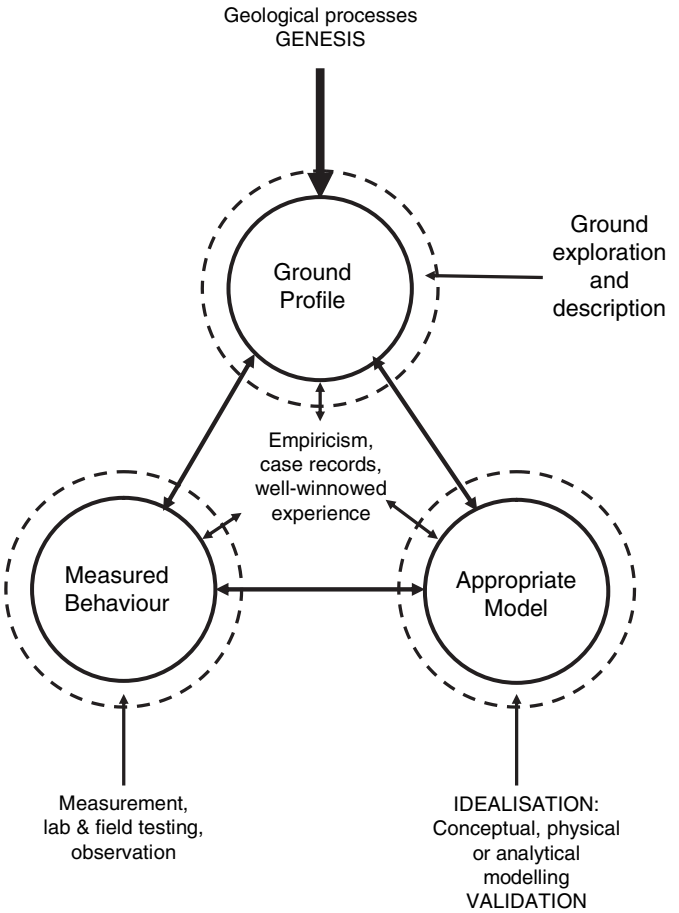


Figure 4.1 The geotechnical triangle. Each activity has its own distinct methodology and its own rigour

activities will now be briefly described. The order in which the activities are carried out will depend on the problem and may involve an iterative approach. For most problems the establishment of the ground profile is an early priority. It is also very important to establish at an early stage what experience is available for the problem being tackled and the ground conditions to be encountered.

4.2 The ground profile

Establishing the ground profile is a key outcome of the ground investigation as described in Chapter 13 *The ground profile and its genesis*. The ground profile is a description of the successive strata in simple engineering terms together with the groundwater conditions and the variations across the site. The importance of handling and describing the soil so as to establish the ground profile cannot be over-emphasised. Also it is vital to understand the geological processes and man-made activities that formed the ground profile, i.e. its genesis. The author believes that nine times out of ten, the key conceptual design decisions for a project can be made on the basis of a good description of the ground profile. Similarly, nine failures out of ten result from a lack of knowledge about the ground profile – often the groundwater conditions. The importance of the ground profile is emphasised by placing it at the top apex of the triangle.

Peck (1962) argued that the methodology of the engineering geologist in establishing the geological model for a site consists in making observations, organising and assembling these, formulating a hypothesis and then critically testing the hypothesis. However, the civil engineer, and in particular the structural engineer, is not usually trained in this methodology, which is at the heart of much geotechnical engineering.

4.3 The measured or observed behaviour of the ground

The activity associated with this aspect involves making measurements of the mechanical behaviour and interpreting the measurements. It includes laboratory and field testing, carrying out field observations of the ground and structural movements (e.g. back analysis of a landslip or of settlement observations) and measurements of groundwater pressures and flow. Rigorous methodologies and advanced instrumentation are often required for this work. The measurements require interpretation, which needs an appropriate analytical framework. A good basic understanding of the mechanical behaviour of soils and rocks is essential to these activities. Section 2 *Fundamental principles* of this manual provides a practical introduction to the fundamentals of soil and rock behaviour.

4.4 Appropriate model

When the author first put forward the original *soil mechanics triangle* in 1987 (now referred to as the geotechnical triangle), the term ‘applied mechanics’ was used for the bottom right-hand corner of the triangle. However, the term ‘appropriate model’ is a much better description of this aspect and its associated activities. The procedures involved in geotechnical and structural modelling are described in Chapter 5 *Structural and geotechnical modelling*.

Modelling is the process of:

- identifying what is to be modelled, e.g. the part of the structure, the particular type of behaviour or limit state;
- idealising or simplifying the geometry, material properties and loading;
- assembling these idealisations appropriately into a model, which is then amenable to analysis and, hence, can be used to predict responses.

The modelling process is not complete until the responses have been validated and assessed. The procedure may involve a number of iterations.

It is important to note that the process of modelling is very much more than simply carrying out an analysis. A model can be a very simple conceptual one, it can be a physical 1g model or a centrifuge model, or it can be a very sophisticated numerical model. By using the term ‘model’ we are emphasising the

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idealisation process and demystifying the analytical process. The geotechnical triangle helps in this. It is also important to appreciate that a variety of models and idealisations will usually be needed for different aspects of a given project or structure.

4.5 Empirical procedures and experience

With materials as complex and varied as the ground, empiricism is inevitable and it is (and will always remain) an essential aspect of geotechnical engineering. That is why this aspect is placed at the centre of the triangle and is linked to the other three aspects. Many of our design and construction procedures are the product of what may be termed 'well-winnowed experience', that is, experience which results from a rigorous sifting of all the facts that relate to a particular empirical procedure or case history. The author chose the term having read Terzaghi's description of his attempts to 'separate the wheat from the chaff'' following his two years in the USA collecting case records (see Chapter 3 *History of geotechnical engineering*).

Empirical procedures are often regarded as somehow inferior to analytical procedures. The fact is that both have their essential place in geotechnical engineering and both need to be carried out with care and with understanding of their limitations.

4.6 Summary of the geotechnical triangle

In summary, depicted within the *geotechnical triangle* illustrated in **Figure 4.1** there are four key aspects of geotechnical engineering, each associated with distinct types of activity with different outputs. Each activity has a distinct methodology, each has its own rigour, and each is interlinked with the other. Terzaghi's approach to ground engineering reveals a coherence and integration, which is reflected in a 'balanced' geotechnical triangle. The geotechnical triangle provides a useful aide-memoire when tackling any ground engineering problem, whether it is in feasibility, design, construction, forensic investigation or teaching. It is possible to adapt the geotechnical triangle for particular applications. For example, in Chapter 9 *Different types of foundation applications* an adaptation is described for the vital task of verifying design assumptions when construction work has commenced and the influence of construction processes on the ground needs to be assessed.

4.7 Re-visiting the underground car park at the Palace of Westminster

Many of the above aspects can be illustrated by means of the well-known case history of the underground car park at the Palace of Westminster, London.

4.7.1 Background

This case history is described in detail by Burland and Hancock (1977). The site finally chosen for the 18.5-m-deep car park for the Members of Parliament was in New Palace Yard,



Figure 4.2 The site of the underground car park in New Palace Yard, Westminster

presenting major engineering challenges due to the proximity of the Big Ben Clock Tower, the Palace of Westminster and the 14th century Westminster Hall, as shown in **Figure 4.2**. The sensitive, load-bearing masonry construction of the buildings and their priceless historic value demanded a structural form and construction procedure for the car park which would result in minimum ground movements, both during construction and in the long term. The structural form chosen consisted of a reinforced-concrete diaphragm wall, strutted at all stages of excavation and construction by the permanent reinforced-concrete floors as shown in **Figure 4.3**. The columns are founded on under-reamed bored piles.

4.7.2 The importance of the ground profile

A preliminary site investigation had been carried out prior to the author's involvement in the project. The results showed that the top 10 m consisted of fill and soft alluvium, overlying medium-dense sand and gravel. London Clay extended from a depth of 10 m to 44 m, at which depth the Woolwich and Reading beds were encountered (now known as the Lambeth Group). A new site investigation was planned, aimed at ascertaining the detailed soil profile, its variation across the site and the groundwater conditions. Altogether 14 boreholes were sunk on the site. The author personally carried out a detailed visual examination on open drive samples from a number of the boreholes by splitting the samples longitudinally with a knife so as to overcome the smearing on the outside of the samples. Visual and tactile descriptions were carried out using a simple scheme described by Burland (1987), which is consistent with BS5930:1999, Code of Practice for Site Investigations. See Chapter 13 *The ground profile and*

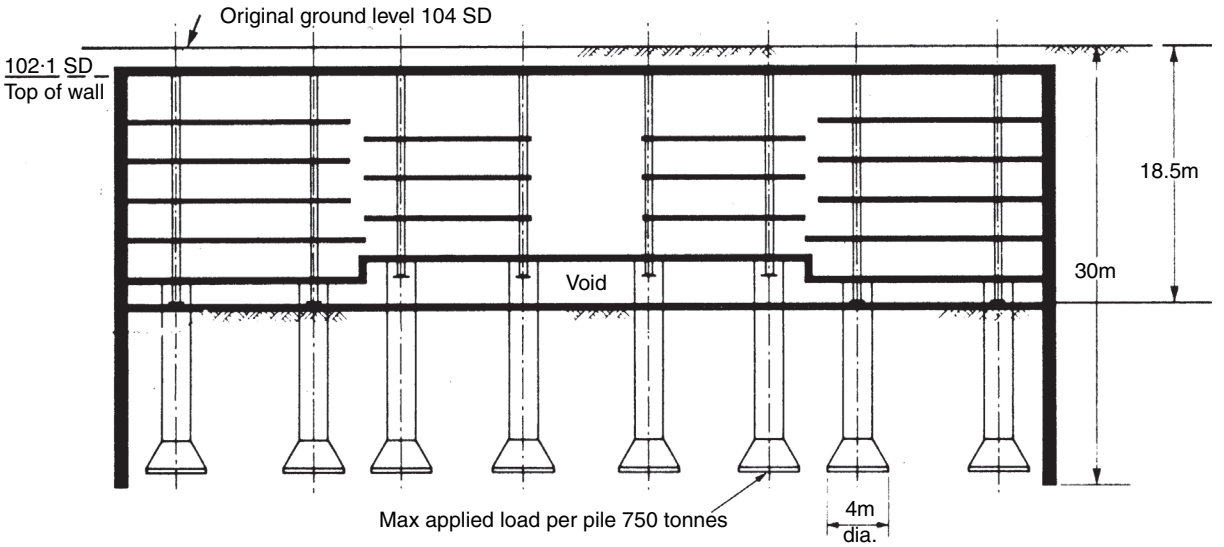


Figure 4.3 Section through underground car park in New Palace Yard

its genesis for more details of the information required in a description of the soil profile.

Good correlation of the various strata was found between the boreholes and the ground profile, as shown in **Figure 4.4**. A visual inspection revealed that immediately beneath the lowest level of the proposed car park, and extending from a depth of 19 m to 30 m, there exists a layer of London Clay containing partings of fine sand and silt up to 10 mm thick and at 50 mm spacing near the top of the layer. The frequency of the partings reduces with depth. This layer is immediately underlain by a 4 m thick layer of very stiff intact clay. Casagrande standpipes installed in most of the soil exploration boreholes showed that the groundwater pressures down to a depth of about 35 m were hydrostatic and in equilibrium with the water table in the overlying alluvium and gravel.

At the time, the finding of the layer containing silt and sand partings came as a surprise. Subsequently it has come to light that it had been encountered during tunnelling elsewhere in London and had caused problems of face instability. Recently, Standing and Burland (2006) encountered the same layer in nearby St James's Park in connection with the Jubilee Line Extension underground railway, where excessive volume losses were encountered during tunnelling. The discovery of this layer containing silt and sand partings had very important implications for the conceptual design of the retaining walls and the foundations. The relatively high horizontal permeability of this layer, coupled with hydrostatic water pressures in the surrounding ground, meant that high water pressures could develop beneath the excavation level, leading to hydraulic uplift and possible base failure. Moreover, seepage from the sand layers could have caused difficulties in the formation of the underreams of the bored piles. The development of high water pressures beneath the bottom of excavations due to the presence of water-bearing permeable strata is a well-known geotechnical hazard. Ward (1957) describes a case in which high water pressures in a stratum of laminated sand caused uplift at the bottom of a trench excavation in stiff clay. The problem was solved by the use of simple gravel-filled relief boreholes.

Various methods were considered for relieving the water pressures immediately beneath the final basement level of the car park, including the use of relief wells. Also detailed flownet calculations were carried out to assess the upward seepage gradients. However, there were considerable doubts about the short- and long-term effectiveness of relief wells. Moreover, the author was very unwilling to rely on a seepage analysis, knowing that even a small undetected permeable region could completely nullify the analysis. A robust solution was adopted, which involved taking the diaphragm retaining walls down into the intact clay layer at a depth of 30 m, thereby cutting off all horizontal seepage along the sand partings.

The choice and depth of the foundations was also profoundly influenced by the soil profile. The indications were that below a depth of 34 m (i.e. below the 4 m thick intact clay layer) there was the possibility of significant erosive seepage during excavation of the piles due to the silty nature of the clay at that depth. At a much shallower depth the significant reductions in effective stress immediately beneath the excavation would give rise to long-term reductions in stiffness and strength. With these considerations in mind, it was decided to use under-reamed bored piles founded on the intact clay layer at a depth of about 30 m, thereby making use of the higher shear strength beneath that level but avoiding difficulties due to seepage. A trial pile was constructed ahead of the main job in order to check on the groundwater and soil conditions and to evaluate the proposed foundation construction procedure. *In situ* inspection of the soil in the trial pile confirmed all the earlier deductions.

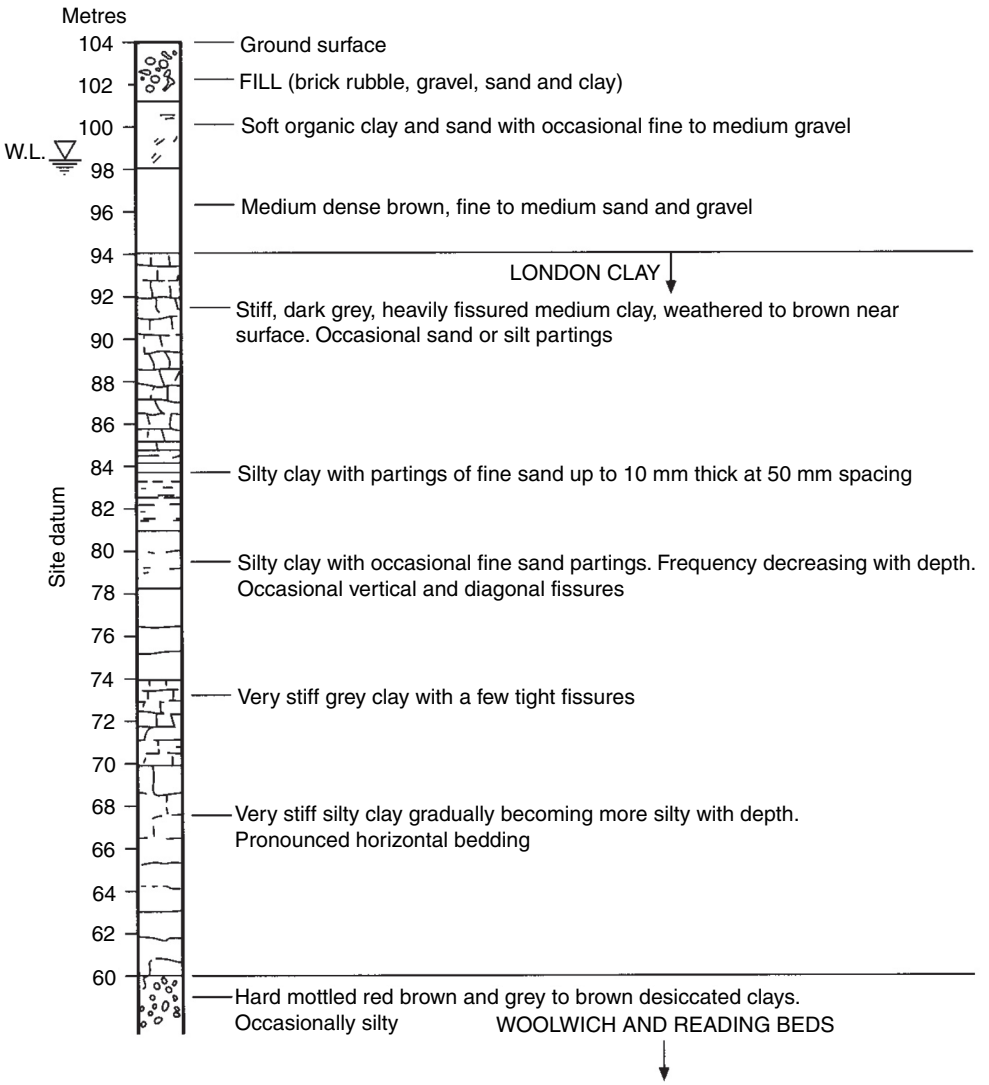


Figure 4.4 The ground profile at the Palace of Westminster

4.7.3 Modelling the ground movements

A major task in the design of the car park was to estimate the effects of the excavation on the surrounding buildings and on the structure itself. Accordingly a detailed finite element analysis was undertaken. The analysis was based on a number of idealisations and it is important to be absolutely clear about these.

Regarding the mechanical properties of the ground, the London Clay was assumed to behave as a linear elastic isotropic porous material having a stiffness that varies with depth. (See Chapter 17 *Strength and deformation behaviour of soils* for a discussion on the strength and deformation behaviour of soils.) This assumption was felt to be reasonable as the best research data available at the time suggested that at small strains, samples of London Clay tested in the laboratory exhibited linear behaviour. The variation of the undrained Young's modulus, E_u with depth used in the analysis is given by the line in **Figure 4.5**. The stiffness values for the London Clay and Woolwich and Reading beds were based on values obtained by back analysis of measurements of the movements of the retaining walls for the deep basement excavation for Britannic House in the City of London (Cole and Burland, 1972). Thus, the choice of stiffness values was based on previous relevant case histories and observations in the London area – a clear example of the application of 'well-winnowed experience'. The adopted values of E_u for the basement beds of the London Clay and the Woolwich and Reading beds are somewhat lower than those obtained at Britannic House. It was felt necessary to be conservative in view of the lack of knowledge about the Woolwich and Reading beds in the Westminster area. Nevertheless, it is worth noting that the values of E_u used in the analysis were three to five times larger than the values deduced from careful laboratory tests. The author would never have chosen such high values without the benefit of experience from the analysis of previous relevant case histories.

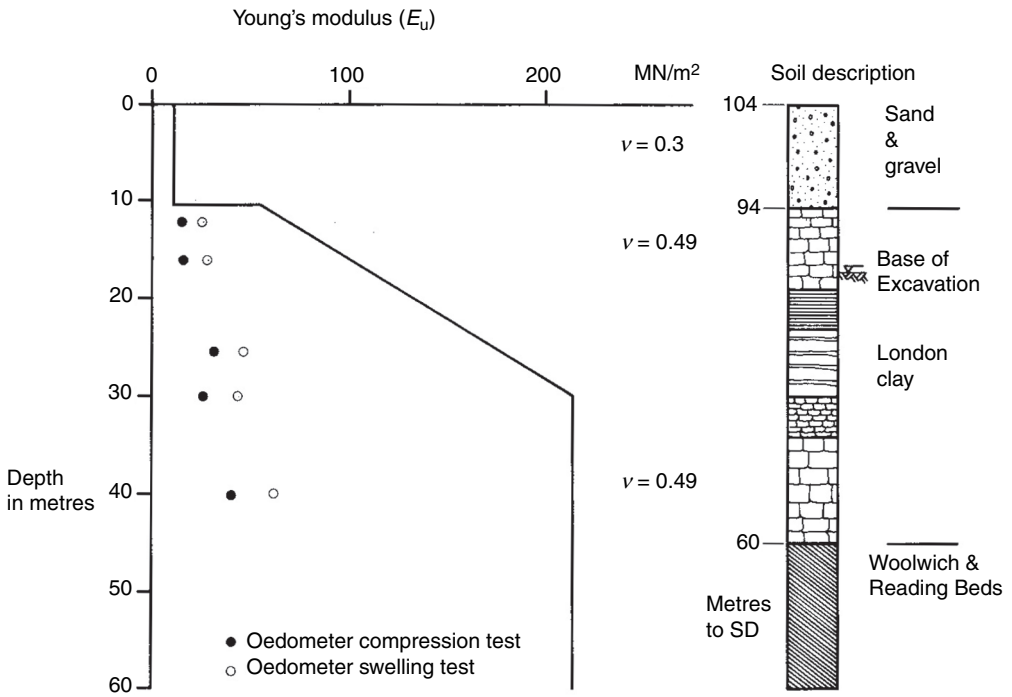


Figure 4.5 Values of undrained Young's modulus used in the finite element analysis

A key aspect of the modelling involved assessing the magnitude of the initial horizontal stresses in the clay, since these had to be progressively released during excavation. Once again the author made use of values deduced by others for various sites in London Clay. An assessment had to be made for the effects of erosion of the surface of the London Clay and the subsequent deposition of the gravel. This illustrates the importance of understanding the genesis of the ground profile – see section 4.2 above.

Even though the problem is highly three-dimensional, a plane strain analysis was adopted as being approximately representative of the centre of the north and south walls, respectively. The finite element mesh extended to the base of the Woolwich and Reading beds (60 m depth) and laterally to a distance of 80 m from the edge of the excavation.

Numerical analysis was carried out in a step-by-step process simulating the excavation and installation of the floor props at each stage, as described by Burland and Hancock (1977). **Figure 4.6** shows the predicted wall displacements at the various stages of excavation, the maximum inward displacement being about 22 mm. The predicted short-term horizontal and vertical movements of the ground surface outside the excavation are given in **Figure 4.7**. These predicted ground surface movements gave a maximum change of gradient of 1/800 and a maximum extension strain of 0.02%. These strains were felt to be unlikely to cause any significant damage to the surrounding buildings. Estimates were also made of the movements of the Clock Tower using an axisymmetric analysis, which was felt to be more appropriate for conditions close to the north-east corner of the car park. The Clock Tower was predicted to rotate away from the excavation by about 1/6000, while its foundations were predicted to move in towards the excavation by about 3 mm. The finite element model was also used to estimate the long-term movements and the swelling of the clay at the base of the excavation. On the basis of these calculations it was decided to use a suspended basement floor with a drained void beneath, rather than a solid slab, so that the clay could swell without interacting with the rest of the structure. It is very important to note that the predicted movements were published prior to work commencing on the project (Ward and Burland, 1973).

The numerical model was also used to estimate the vertical movement of the under-reamed piles during and subsequent to excavation. This information was used to assess the approximate magnitudes of the differential movements across the floor slabs and the associated bending and shear forces within them due to ground movements.

4.7.4 Field monitoring

In view of the depth of excavation, the close proximity of such priceless historic buildings and the sensitivity of the project, a very comprehensive programme of monitoring was undertaken during all phases of excavation and construction. Briefly, the monitoring can be considered under three broad headings: precision surface surveying, ground movements at depth and pore water pressures. Approximately 60 movement points were established by grouting Building Research Station (BRS)

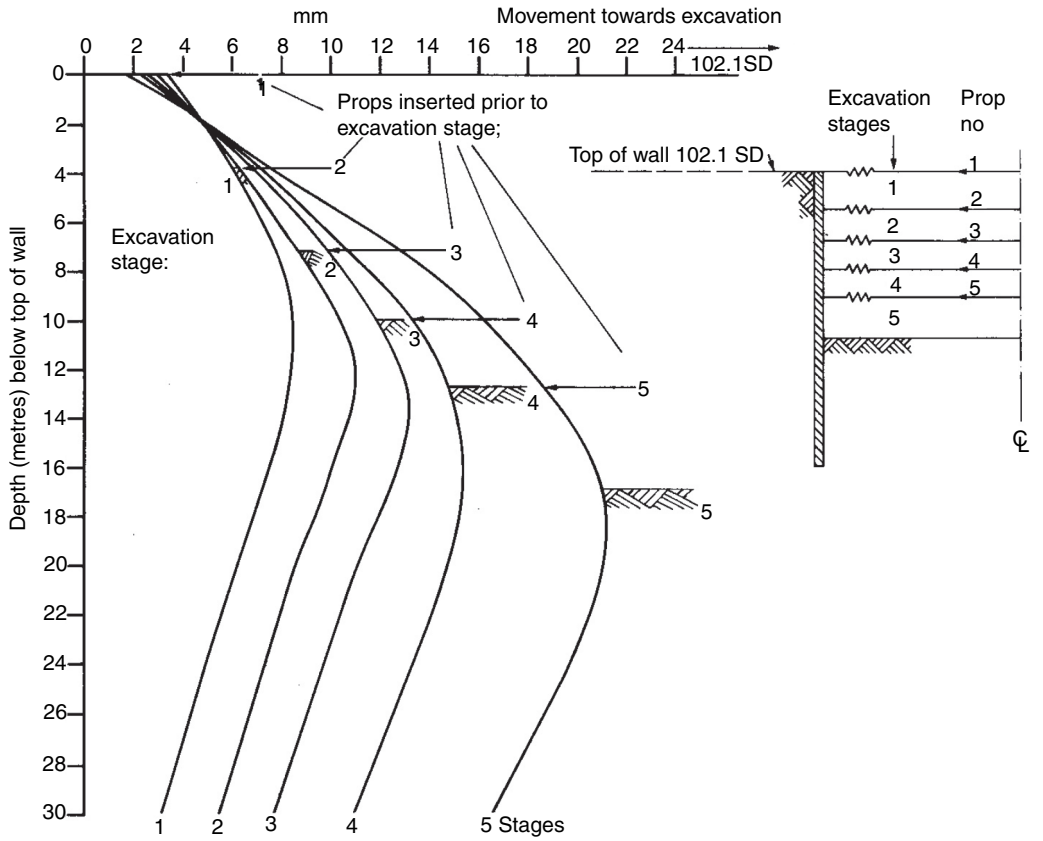


Figure 4.6 Predicted horizontal wall movements during excavation

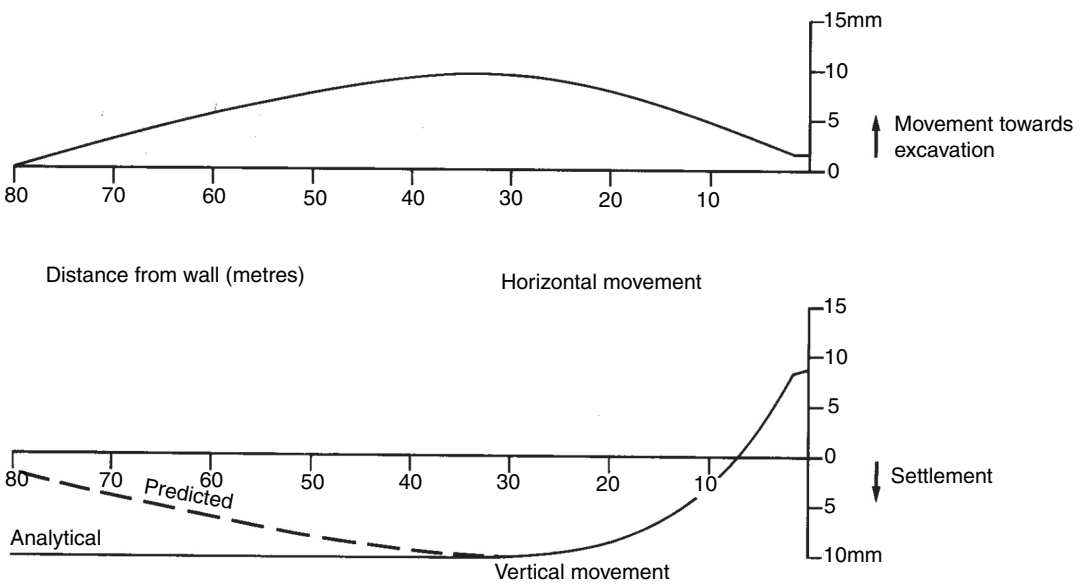


Figure 4.7 Predicted horizontal and vertical ground surface movements outside the excavation

levelling stations into the masonry of the surrounding buildings (Cheney, 1973). Precision settlement surveys were carried out continuously during construction. The frequency of observations was increased close to areas where construction activity was concentrated. Complete geodetic surveys in both plan and elevation were made at intervals of approximately two months over the construction period of about two years. Changes in the verticality of the Clock Tower were monitored daily by means of a Hilger and Watts 'autoplumb'.

The subsurface horizontal deflections of the diaphragm retaining walls were measured by means of inclinometers. Special demountable targets were fitted to the tops of the inclinometer tubes, and their positions were measured in plan and elevation at two-monthly intervals. An important aspect of the below-ground monitoring was to measure the heave at various depths during excavation as a check on the effectiveness of the design measures. Two magnet extensometers (Burland *et al.,* 1972) were installed at the centre of the excavation for this purpose.

A number of standpipes and pneumatic piezometers were installed within the area of the excavation and around its perimeter. They were to be used, in conjunction with the magnet extensometers, to check that hydraulic uplift was not developing. Unfortunately the pneumatic piezometers and many of the standpipes within the excavation were destroyed, but the standpipes outside the excavation all functioned satisfactorily.

4.7.5 Observed behaviour

Figure 4.8 shows the observed deflected shapes of the centre of the south diaphragm wall (inclinometer 8) at the various stages of excavation and support. These may be compared with the predicted behaviour given in **Figure 4.6**, where it is evident that the agreement is very satisfactory. The final predicted shape of the wall is shown as a broken line in **Figure 4.8(a)**. The major difference between the predicted and observed movement is the overestimation of the inward movement beneath the final excavation level. This is a direct result of the deliberate choice of conservative values of the undrained Young's modulus, E_u for the basement beds of the London Clay and the Woolwich and Reading beds.

A large range of deflected shapes were observed for the various wall panels. **Figure 4.8(b)** shows the two extreme cases given by inclinometers 3 and 10. The results demonstrate that, even for relatively uniform ground conditions, widely differing deflected shapes can be anticipated. These differences may be due in part to the precise manner in which the excavation was carried out at various locations. Nevertheless, the differences shown in **Figure 4.8(b)** are a salutary reminder that there are limits to the precision with which predictions can be made.

In **Figure 4.9** the measured total horizontal and vertical surface movements behind the south wall are compared with the predictions given in **Figure 4.7**. The predicted horizontal

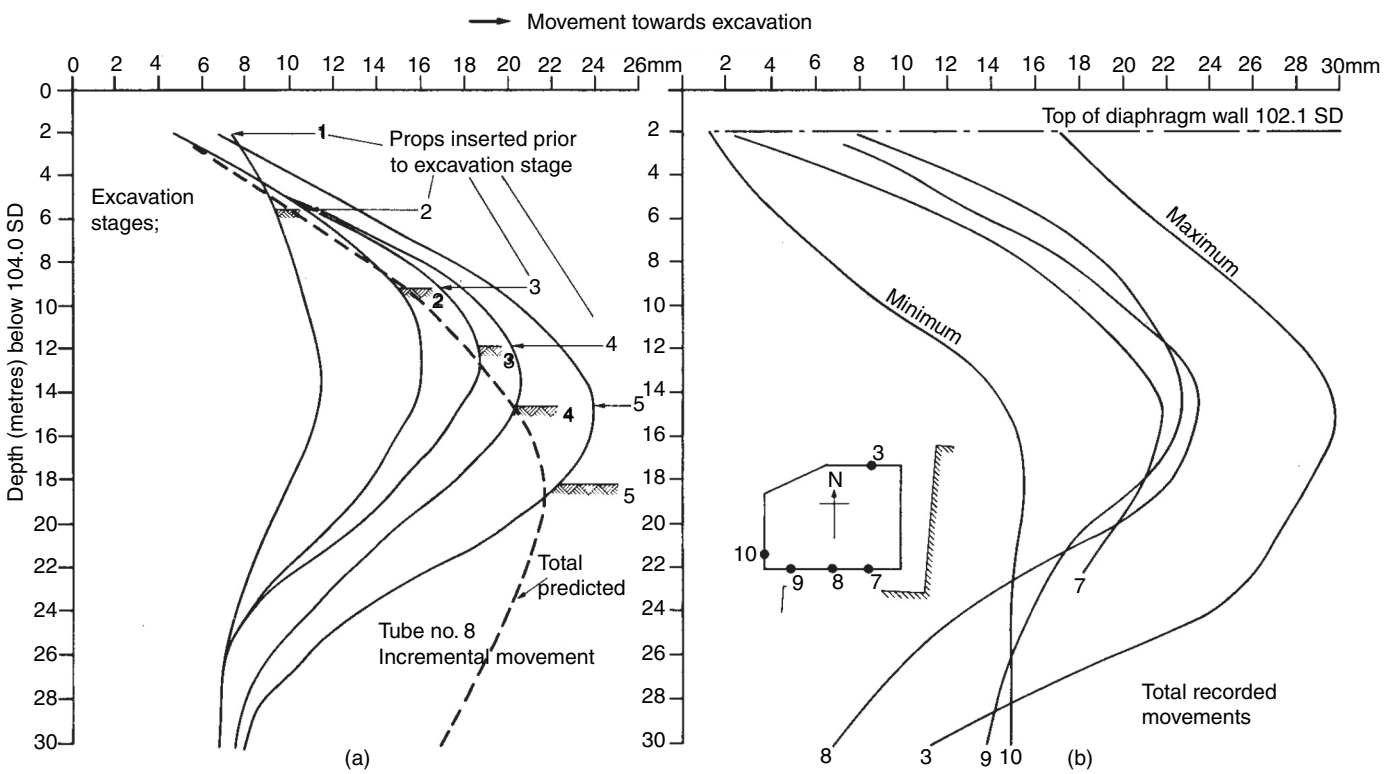


Figure 4.8 Observed movements of the diaphragm walls. (a) Inclinometer tube 8 at various stages of excavation. (b) Range of measured movements at end of excavation

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movements are somewhat less than the observed values within 25 m of the wall, but further away the agreement is good. In general the overall form and magnitude of the prediction is reasonable. However, agreement between the observed and predicted vertical movements is not nearly as good. The upward displacement of the wall has been overestimated, while the maximum settlement has been underestimated. The 'settlement trough' is much nearer the wall than predicted. A consequence was that, whereas the Clock Tower was predicted to tilt away from the excavation by about 1/6000, it actually tilted towards the excavation by about 1/7000. The predicted and observed horizontal displacements were much more satisfactory, being 3 mm and 5 mm, respectively, towards the excavation.

In summary: The detailed ground profile together with experience of previous similar constructions led to the key conceptual design decisions – top-down construction to minimise ground movements, diaphragm walls cutting off horizontal seepage through the layer containing silt and sand partings, and founding the under-reamed piles in the intact clay layer. The stiffness parameters used in the finite element analysis were based on a back analysis of other deep excavations in the London area. Numerical modelling was used primarily to assess the movements in the surrounding Palace of Westminster but it was also used in the structural design of the diaphragm walls. The monitoring of the movements and groundwater pressures was carried out as a check that the behaviour was within acceptable limits. The car park was completed without significant damage to the surrounding buildings.

4.7.6 Subsequent refinements of the ground model

The difference between the predicted and measured vertical ground surface displacements just described proved very puzzling. However, shortly after the measurements were published by Burland and Hancock (1977), Simpson *et al.* (1979) showed that, by using a bilinear stress–strain law with a high initial stiffness, the agreement between observations and predictions could be greatly improved – particularly with respect to the vertical movements, as shown by the broken lines in **Figure 4.10**. At the same as this theoretical work was being carried out, laboratory studies began at Imperial College in which axial strains were measured locally on soil samples instead of between the end plattens, as was traditional. These measurements gave highly *nonlinear* stress–strain behaviour, with stiffnesses at small strains which were much larger than those inferred from traditional measurements. It now became clear that the pattern of ground surface movements observed at New Palace Yard, in which the vertical movements are concentrated

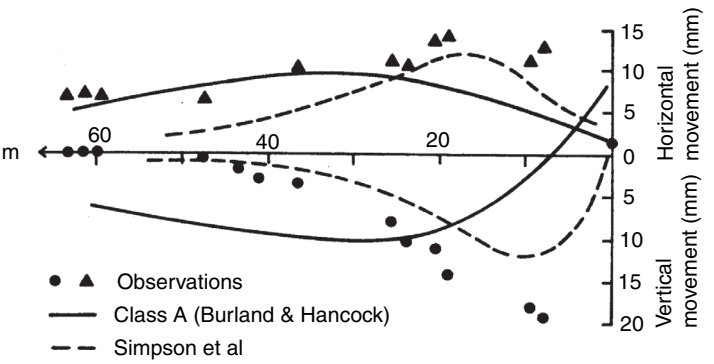


Figure 4.10 Improved predictions of the shape of the settlement trough (shown as a broken line) due to inclusion of small strain *nonlinearity* in the soil model

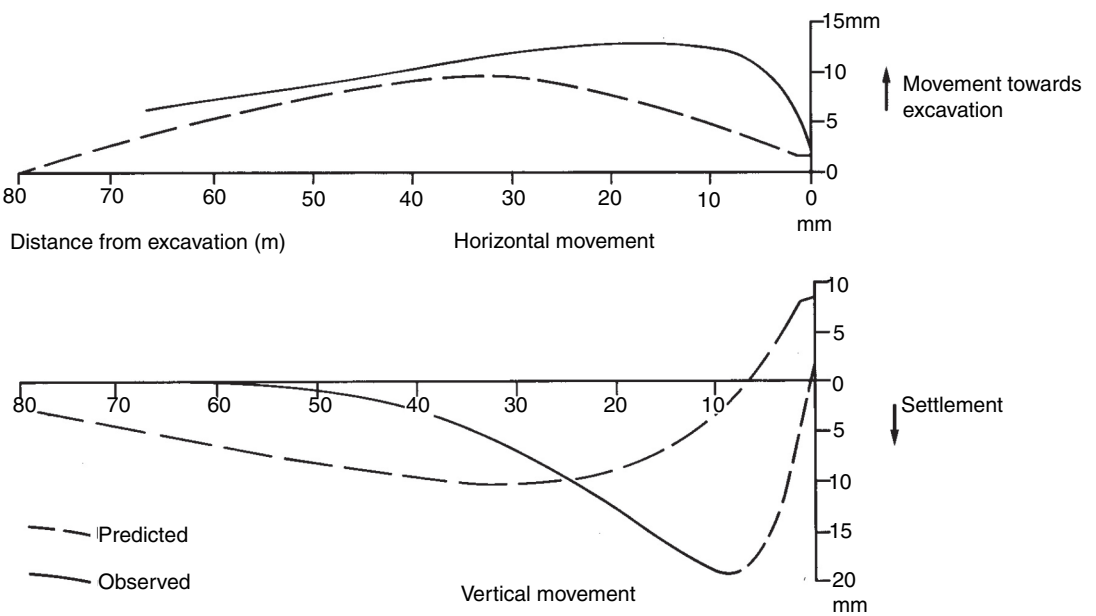


Figure 4.9 Comparison of predicted and measured ground surface movements away from the south wall

close to the edge of the excavation, is due to the *nonlinear* nature of the stress–strain behaviour of the soil.

This process of prior publication of predictions, though uncomfortable at the time, has proved highly beneficial as it caused the author to ponder long and hard as to the explanation for the discrepancies. Without such public disclosure it would have been all too tempting to quietly ignore the discrepancies and move on to other things. The work at New Palace Yard, and the measured response of the Clock Tower, has spawned a whole new important area of study of the behaviour of the ground at small strains – indeed whole international conferences are now devoted to the subject. These studies are proving very important for modelling the key mechanisms governing the interaction effects between ground and structure. Understanding these mechanisms is particularly important in the urban environment, where underground construction is a vital part of infrastructure development.

This case study is a clear demonstration of the importance of publishing well-documented case histories, as they serve to contribute to the profession's collective experience and can be used to calibrate new numerical modelling techniques for many years to come.

4.8 Concluding remarks

This chapter outlines a coherent approach to the key aspects of geotechnical engineering by making use of the *geotechnical triangle* (see **Figure 4.1**). The distinct and rigorous activities of (1) ground investigation, (2) measurement and observation and (3) appropriate modelling are represented as the apexes of the triangle with (4) 'well-winnowed experience' (an equally rigorous activity) linked to all three at the centre of the triangle.

The application of the geotechnical triangle has been illustrated by re-visiting the well-known case history of the underground car park at the Palace of` Westminster, London. A deep excavation was chosen as a case history since this type of problem demands all the traditional skills of the engineer including: reliance on observation and measurement; a deep understanding of both geotechnical and construction materials; the effects of groundwater and seepage; the development of appropriate conceptual and analytical models; and above all, judgment based on a knowledge of case histories and construction methods.

It should be noted that the key conceptual design decisions for the underground car park at Westminster were taken on the basis of a thorough knowledge of the ground profile obtained from the visual and tactile description of the soil samples and measurements of groundwater pressures from standpipes. The choice of stiffness properties for the London Clay was based on the careful and rigorous analysis of other appropriate case histories of deep excavations in the London area, illustrating the importance of case histories and 'well-winnowed experience'. The field measurements made during the excavation of the car park have proved most valuable in gaining an understanding of the influence of small-strain *nonlinear* behaviour of the ground. Even more importantly, these measurements have shown that precise prediction of behaviour is not possible and that successful design requires that inherent uncertainties of response must be taken into account.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

Chapter 5

Structural and geotechnical modelling

John B. Burland Imperial College London, UK

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During his career the author has encountered profound differences in the approaches taken by structural and geotechnical engineers, often leading to a lack of understanding and difficulties in communication. This chapter explores these differences and the reasons for them.

The term *modelling* is used extensively. It is defined as the process of idealising a structure and the underlying ground including the geometry, material properties and loading in order to make the problem amenable to analysis and, hence, assessment for fitness of purpose. It is demonstrated that traditional structural modelling is very different from geotechnical modelling and these differences need to be appreciated if ground–structure interaction problems are to be realistically modelled. It is concluded that concepts such as *ductility* and *robustness* underpin the success of both structural and geotechnical modelling and more explicit recognition of these is needed.

5.1 Introduction

There is always an interaction between a structure and its foundation … whether or not the designers allow for it. In some situations, the interaction can be minimised, e.g. by founding the building on rigid piles. Such an approach can be very costly and is often not feasible. If structure–ground interaction is to be taken into account, structural and geotechnical engineers themselves have to interact.

There have been many occasions when the author has both witnessed and experienced difficulties in communications between structural and geotechnical engineers and it has been a continuing source of interest to him as to why this should be so (Burland, 2006). It is a matter of outstanding importance because poor communication and lack of understanding can lead to poor engineering and even failures.

At the heart of the problem, there are differences in the day-to-day approach to *modelling* structural and geotechnical behaviour. By identifying these differences it is possible to move towards more integrated and realistic approaches to designing for ground–structure interactions.

The term *modelling* is used extensively in this chapter. It is intended to describe the process of idealising a real-life structure, foundations and ground (or parts of them) including the geometry, material behaviour and loading, in order to make them amenable to analysis, carrying out that analysis, reviewing it and then assessing the design for 'fitness for purpose'. Thus, the process of modelling entails very much more than the process of carrying out the analysis itself.

5.2 Structural modelling

The late Edmund Hambly, a most innovative structural engineer who obtained his PhD in soil mechanics, published a little book *Structural Analysis by Example* (Hambly, 1994). It was intended as a handbook for undergraduates and professionals who like to use physical reasoning and off-the-shelf computer modelling to understand structural form and behaviour. Fifty examples are given of structural problems of increasing complexity ranging from simple frames, through beams, columns and slabs, to shear lag and torsion, and then on to whole structures such as offshore platforms and spiral staircases. The book represents an admirable summary of the range of problems and types of analysis that a structural engineer encounters in day-to-day practice.

In terms of *modelling*, it is evident that the geometry of most structures is well defined and reasonably easy to idealise. Rather simple linear elastic material behaviour is usually assumed with a limiting stress imposed. The major idealisations in the modelling process are in the loading, although in Hambly's book this is specified, as is usually the case in codes of practice and standards. It is evident that the process of routine structural modelling mainly consists of idealising the structural form (often termed the 'conceptual model') and carrying out analysis – usually on the computer. Recently, the Institution of Structural Engineers (IStructE, 2002) and MacLeod (2005) set out more formal procedures for the structural modelling process.

5.2.1 Limitations to structural modelling

Using Hambly's book as an exemplar, it is evident that structural engineers think primarily in terms of the forces and stresses (or equivalent elastic displacements) that are the outputs from the structural models that they use in day-to-day practice. Yet most studies on real whole structures (especially buildings) show that the measured strains and displacements bear little resemblance to the calculated ones (Walley, 2001). Classic experiments on large-scale steel structures, carried out in the early 1930s under the direction of Lord Baker, revealed that the measured stresses

under working loads bore very little relation to the calculated values (Baker *et al.*, 1956). National Building Studies Research Paper No 28 records that the biggest change of strain in the steel beams of the Ministry of Defence building in Whitehall was caused by the shrinkage of the concrete after the floors had been cast – this was larger than that induced by the subsequent loading of the floors (Mainstone, 1960). In his own work, the author has found that the thermal and seasonal movements of buildings are often as large as the movements induced by tunnelling and yet these are overlooked when assessing the response of buildings to foundation movements (Burland *et al.*, 2001). These limitations have been appreciated for a long time but are easily forgotten. The fact is that the uncertainties of modelling real projects are dealt with in good practice not only by factors of safety, but also by incorporating appropriate levels of ductility and robustness.

5.2.2 Ductility and robustness

Ductility may be defined as *the ability to undergo inelastic deformations without significant loss of strength*, while robustness can be defined as *the ability to absorb damage without collapse*.

On page 1 of Hambly's book, reference is made to the *safe design theorem*, which, of course, derives from the lower bound theorem of plasticity. This theorem states that:

A structure should be able to carry its design loads safely if:

1. *The calculated system of forces is in equilibrium with the loads and reactions throughout the structure.*
2. *Each component has the strength to transmit its calculated force and the ductility to retain its strength while deforming.*
3. *The structure has sufficient stiffness to keep deflections small and to avoid buckling before design loads are reached.*

If the real structure deforms under load with a different flow of forces from that calculated, it should still be safe as long as the materials are ductile (like steel) and not brittle (like glass). The equilibrium check ensures that any underestimate of the force flowing through one part of the structure is balanced by an overestimate in the force in another part. Then ductility ensures that the component that is overstressed retains its strength while deforming, and sheds the excess force to the parts that have available capacity as a result of the equilibrium check. It is the ductility of structures that ensures the success of current design methods. This is frequently overlooked and, in the author's experience, many structural engineers still seem to believe that their structures behave as calculated – a belief that powerful computer programs and prescriptive codes of practice tend to reinforce. Both Heyman (2005) and Mann (2005) have written on this topic and stressed the need to ensure that the structural details are ductile – also a prerequisite for earthquake design. The inherent ductility and robustness of steel structures is well known and gave rise to modern methods of plastic analysis. Beeby (1997, 1999) summarised the developments in the understanding of ductility for traditional reinforced-concrete design and stressed the importance of designing for robustness.

It is important to note that the majority of structural failures result from defects at joints and connections (Chapman, 2001). Engineers do, of course, carry huge responsibility for ensuring that local instabilities cannot develop – particularly in temporary works and propping systems. For steel structures Burdekin (1999) drew attention to how important it is for designers to be aware of the factors that give rise to brittle fracture and fatigue.

5.2.3 The three- and four-legged stools (Hambly's paradox)

A simple but profound physical model can be used to illustrate what has been discussed so far. **Figure 5.1** shows two stools, one with three legs and one with four (Hambly, 1985). Imagine that each must support a milkmaid who weighs 60 kg, and who always sits with her centre of gravity directly over the middle of the stool. The problem is to determine how much load must be carried by each leg of the three-legged stool and the four-legged stool.

The three-legged stool is straightforward in that one third of the milkmaid's weight must go down each leg, i.e. 20 kg. For the four-legged stool the answer of 15 kg is wrong! Careful inspection of **Figure 5.1** shows that one of the four legs does not quite touch the ground, either because the leg is slightly short or because the ground is uneven; consequently the leg is not carrying any load. The opposite leg will also not be carrying any load because its load must balance that in the short leg when equilibrium of moments is considered about the diagonal through the other two legs. Thus, all of the weight is carried by two legs, i.e. 30 kg per leg, instead of being shared by the four legs. Hence, the paradox (Heyman, 1996) – the addition of a fourth leg to a three-legged stool can increase, rather than decrease, the force likely to be carried by each leg. So what load *should* the legs be designed to carry?

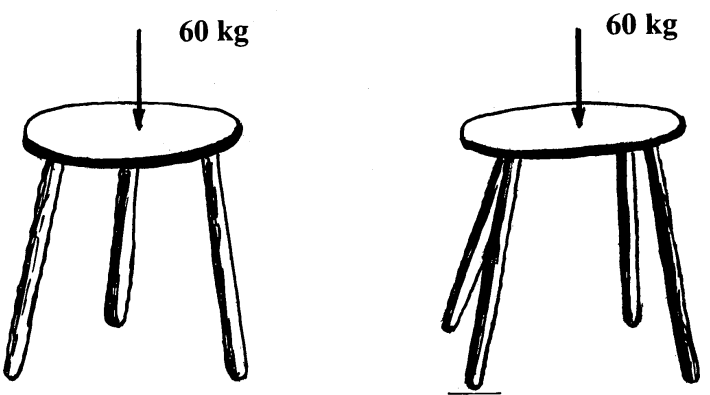


Figure 5.1 Three- and four-legged stools – Hambly's paradox

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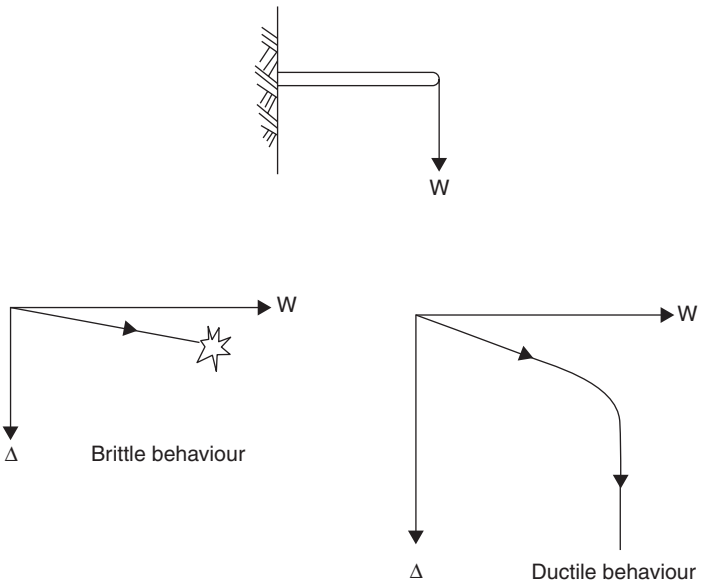


Figure 5.2 Brittle and ductile behaviour of stool leg

It is here that concepts of ductility and robustness come in and the illustrative model can be extended to include material properties. **Figure 5.2** shows the load-displacement properties of a leg, either when acting as a cantilever or in axial compression. In one case, very brittle behaviour is illustrated such that at a certain load the strength of the leg is lost completely, as it would be if it were made of glass. In the second case, the member exhibits ductile behaviour and retains its strength once yield has occurred.

If brittle material is used for the three-legged stool, then accidental overload, due perhaps to a very heavy milkmaid or the cow kicking out at the stool, can easily result in total collapse. Clearly high factors of safety are required to deal with this design. It may be decided to opt for four legs but this may be of little help. The design load for each leg would have to be 1.5 times higher than for the three-legged design. Moreover, accidental overload may cause loss of one member and there is then a risk of progressive collapse. In other words the structure is *fragile*.

If ductile properties are chosen, there is little likelihood of catastrophic collapse if one of the three legs is damaged. Moreover, with four legs there is scope for redistribution of the load once the carrying capacity of a leg is reached. Even accidental removal or serious damage to one member is unlikely to give rise to progressive collapse.

This simple example is very profound and can be extended to other aspects of structural behaviour and design including buckling and ground–structure interaction. Above all, it illustrates the importance of ductility, robustness and redundancy. It is useful to quote Heyman's (1996) conclusions to his study of Hambly's paradox:

> Hambly's four-legged stool stands, of course, for the general problem of design of any redundant structure. It has long been recognised that, in order to calculate the 'actual' state of a structure under specified loading, all three of the basic structural statements must be made – equilibrium, material properties and deformation (compatibility and boundary conditions). However, the calculations do not in fact lead to a description of the actual state. Boundary conditions are, in general, unknown and unknowable; an imperfection in assembly, or a small settlement of a footing, will lead to a state completely different from that calculated. This is not a fault of the calculations, whether elastic or not – it is a result of the behaviour of the real structure. … There is no correct solution to the equations, but one solution that will lead to the greatest economy in material. This is the solution sought by the simple plastic designer, and it is safe and valid provided that no instability is inherent in the structure.

The important message is that, in the process of structural modelling, the inherent uncertainties are such that the precise state of the structure cannot usually be calculated. The art of structural engineering is to use the process of modelling to produce a design that is robust enough to safely cope with the uncertainties, at reasonable cost, and that is fit for purpose. Difficulties arise in interacting with geotechnical engineers when the inherent uncertainties in the structural analysis are not recognised, i.e. when the output from a computer program is believed to represent the true state of a structure.

5.3 Geotechnical modelling

Soil mechanics is a difficult subject and is regarded by many structural engineers as a kind of black art. It is helpful to discuss the reasons for some of the difficulties.

One difficulty is that the soil, unlike concrete or steel, is a particulate material with little or no bonding between the particles. It is made up of an infinite variety of shapes and sizes of particles. This material is usually modelled as a continuum – but it is important never to forget that its properties are determined by its particulate nature.

Because soil is particulate, the water pressures acting within the soil pores are just as important as the stresses applied to its boundaries. This means that changes in the groundwater regime can be crucial in stabilising, or destabilising, a slope, retaining wall or foundation.

In summary, whereas the strength of structural materials, such as steel and concrete, is primarily cohesive and well defined, soil is primarily frictional, so that the confining pressure and pore water pressure determine its strength and stiffness.

5.3.1 The geotechnical triangle

But it is not just the complexity of soil as a material which causes difficulty. As pointed out in Chapter 4 *The geotechnical triangle* there are at least four distinct but interlinked aspects of any geotechnical problem:

- the ground profile – what is there and how it got there;
- the measured behaviour of the ground;

- prediction using appropriate models;
- empirical procedures; judgement based on precedent and 'well-winnowed experience'.

It is the author's experience that the major difficulty in soil mechanics lies, not so much in the complexity of the material as such, but in the fact that, all too often, the boundaries between the above four aspects become confused (Burland, 1987).

The first three of these may be depicted as forming the apexes of a triangle with empirical procedures occupying the centre as shown in **Figure 5.3**. Associated with each of these aspects is a distinct and rigorous activity or discipline.

Ground profile: This is the visual description in simple engineering terms of the successive strata making up the ground together with the groundwater conditions. The importance of handling and describing the soil so as to establish the ground profile cannot be over-emphasised. It is also important to understand how the ground got there and what might have happened to it during its formation – what is termed its *genesis*. As described in Chapter 13 *The ground profile and its genesis*, establishing the ground profile is a key outcome of carrying out a site investigation – often far more important than measuring the material properties.

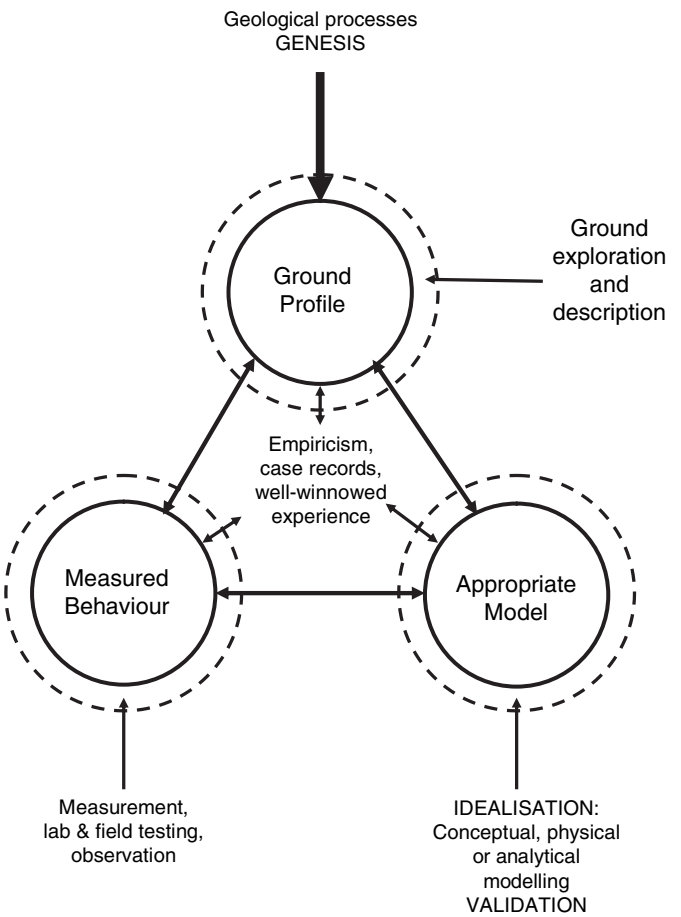


Figure 5.3 The geotechnical triangle. Each activity has its own distinct methodology and its own rigour

Behaviour of the ground: This is established from laboratory or *in situ* tests and usually requires very careful and experienced interpretation. The properties may also be inferred from field measurements of behaviour, e.g. the back analysis of a landslip or of settlement observations. Section 2 *Fundamental principles* of this manual provides a practical introduction to the fundamentals of soil and rock behaviour.

Appropriate modelling: This can be carried out at all sorts of levels. It may be purely conceptual and instinctive, it might be physical modelling or it can involve extremely sophisticated mathematical or numerical work. At whatever level, modelling first involves identifying what is to be modelled, and then follows a process of idealisation (to analyse is first to idealise), after which the analysis itself is undertaken. This should then be followed up by review and assessment – it often isn't! Frequently in soil mechanics, as in structural engineering, it is understanding the basic mechanisms of behaviour that is the key rather than the fine quantitative detail. It may be of interest to note that, whereas the success of much structural modelling relies on the *lower bound theorem of plasticity*, geotechnical modelling has traditionally made use of both the *lower* and *upper bound theorems of plasticity* for the study of limiting equilibrium using stress characteristics and slip-line fields. Thus, both disciplines are firmly grounded in plasticity. For both disciplines, if the material or structure is brittle in its response, extreme caution is required, as the lower and upper bound theorems no longer strictly apply.

Empirical procedures: With a material as complex as the ground, empiricism is inevitable and it is (and will always remain) an essential aspect of ground engineering. Many of our design and construction procedures are the product of what the author has termed *well-winnowed experience.*

Each activity in the geotechnical triangle has its own distinct methodology and rigour. Geotechnical modelling requires that each aspect be considered and that the geotechnical triangle remains 'in balance'. See Chapter 4 *The geotechnical triangle* for a more detailed discussion of the geotechnical triangle.

5.4 Comparisons between structural and geotechnical modelling

It is immediately obvious from the above that the processes of geotechnical modelling usually involve much greater explicit uncertainties and complexities in idealising both the geometry and the material properties than in structural modelling, where both of these are usually specified by the engineer.

In introducing the subject of soil–structure interaction to students, the author starts by asking them to imagine the utopian situation of having unlimited computational power in which, given the *geometry* of the problem, the *material properties* and the *loading*, any problem can be analysed. With this unlimited computational power, how much better would the results be?

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As soon as the idealisations that are involved in most ground–structure interaction problems and the uncertainties of the boundary conditions are examined, it becomes obvious that the results will not be much better than they are today. Any design has to make allowance for these uncertainties.

5.4.1 The geotechnical triangle revisited

Hopefully, the above discussion will serve to help structural engineers to understand why the process of modelling geotechnical problems is inherently less certain than it appears to be for most structural problems. This is because much of the time the structural engineer is working with materials that are specified and manufactured under strict control. Usually the structural form can be idealised in a reasonably straightforward manner. The major uncertainties lie in the loading, in inevitable imperfections and in the connections. In geotechnical engineering, both the geometry (ground profile) and the properties (ground behaviour) of the 'structure' are laid down by nature and not specified. Precise analysis is usually not possible and the key requirement is to understand the dominant mechanisms of behaviour and their likely bounds.

Perhaps the differences between routine structural and geotechnical modelling can best be illustrated by comparing the approach of the structural engineer working on an existing building to that of the geotechnical engineer. In this situation the structural engineer is no longer able to specify the material and the structural form is often difficult to idealise (Burland, 2000). **Figure 5.4** shows an isometric of the West Tower of Ely Cathedral, which was strengthened during 1973 and 1974 as described by Heyman (1976). Also shown in **Figure 5.4** is the geotechnical triangle of **Figure 5.3** but with some descriptions changed to show the key activities undertaken by a structural engineer. For the *soil profile* at the top of the triangle, the *structure of the building and its materials* has been inserted. To establish these requires the most careful examination and investigation. As with the ground, small discontinuities and weaknesses can play a major role in determining the overall response. It is also vital to establish the way the building was constructed and the changes that have taken place historically – this might be called the *genesis of the building* and it is analogous to the geological processes that have formed the ground profile. At the bottom left of the triangle are the *properties* of the materials and the *observed behaviour* of the building. This aspect requires observation, field measurement, sampling and testing. At the bottom right of the triangle there is the need to develop *appropriate predictive models* that take account of the form and structure of the building, its history, its material properties and known behaviour – an almost identical requirement to that for the ground. There is a whole spectrum of models that can be developed, ranging from the intuitive and conceptual right through to highly sophisticated numerical models. The key is to appreciate the inevitable idealisations that have to be made and the limitations that they impose. Finally, as in ground engineering, well-winnowed experience is of supreme importance and well-documented case records are invaluable. It is of interest to note that, in developing his structural understanding of the behaviour of the West Tower of Ely Cathedral, Heyman (1976) drew extensively on the *safe design theorem* and did not attempt to model the 'actual' stress distributions within the structure.

It is evident from the foregoing that, even if engineers were in possession of unlimited analytical power, the uncertainties in both the ground and the structure are so great that precision in the prediction of behaviour would be unlikely to improve significantly. As in so many fields of engineering, modelling is only one of many tools required in designing for the soil–structure interaction. In most circumstances the real value of modelling will be in assisting the engineer to place bounds on the likely overall behaviour, in understanding the mechanisms of behaviour and in beneficially modifying that behaviour if necessary.

5.5 Ground–structure interaction

Both structural engineering and geotechnical engineering adopt a variety of procedures for modelling a wide range of problems. Section 2 *Fundamental principles* of this manual describes the basic approaches to a variety of geotechnical problems such as bearing capacity, settlement prediction, slope stability, etc. The book by Hambly (1994), mentioned previously, summarises the range of problems and types of analysis that a structural engineer encounters in day-to-day practice. Difficulties emerge when it is necessary to combine structural and geotechnical modelling.

Structural engineers are tempted to model the ground as a series of discrete springs since this allows conventional structural analysis software to be used. Sometimes sophisticated springs are used incorporating bilinear or nonlinear load-deflection characteristics to simulate the onset of plastic behaviour. Under certain circumstances it is reasonable to adopt springs to model the interaction with the ground (e.g. in some retaining-wall problems) but in the majority of situations their use is inappropriate and leads to very misleading results.

The simple case of a uniformly loaded flexible slab resting on clay can be used to illustrate the inappropriateness of adopting springs to model the ground – see **Figure 5.5(a)**. It is well known that for the majority of types of ground the slab will settle more in the middle than at the edges and settlement will also take place outside the loaded area, as shown in **Figure 5.5(b)**. Methods of calculating these settlements are described in Chapter 19 *Settlement and stress distributions*. If the soil is represented by a series of springs, then, as shown in **Figure 5.5(c)**, it is evident that the slab will settle uniformly and there will be no settlement outside the loaded area. Moreover, there is no simple standard way of calculating the appropriate spring stiffness from the measured compressibility or stiffness of the ground. Thus, the use of springs to represent the ground in this case will not be capable of capturing the differential settlements across a building founded on the slab or of simulating possible interaction effects with adjacent structures or services.

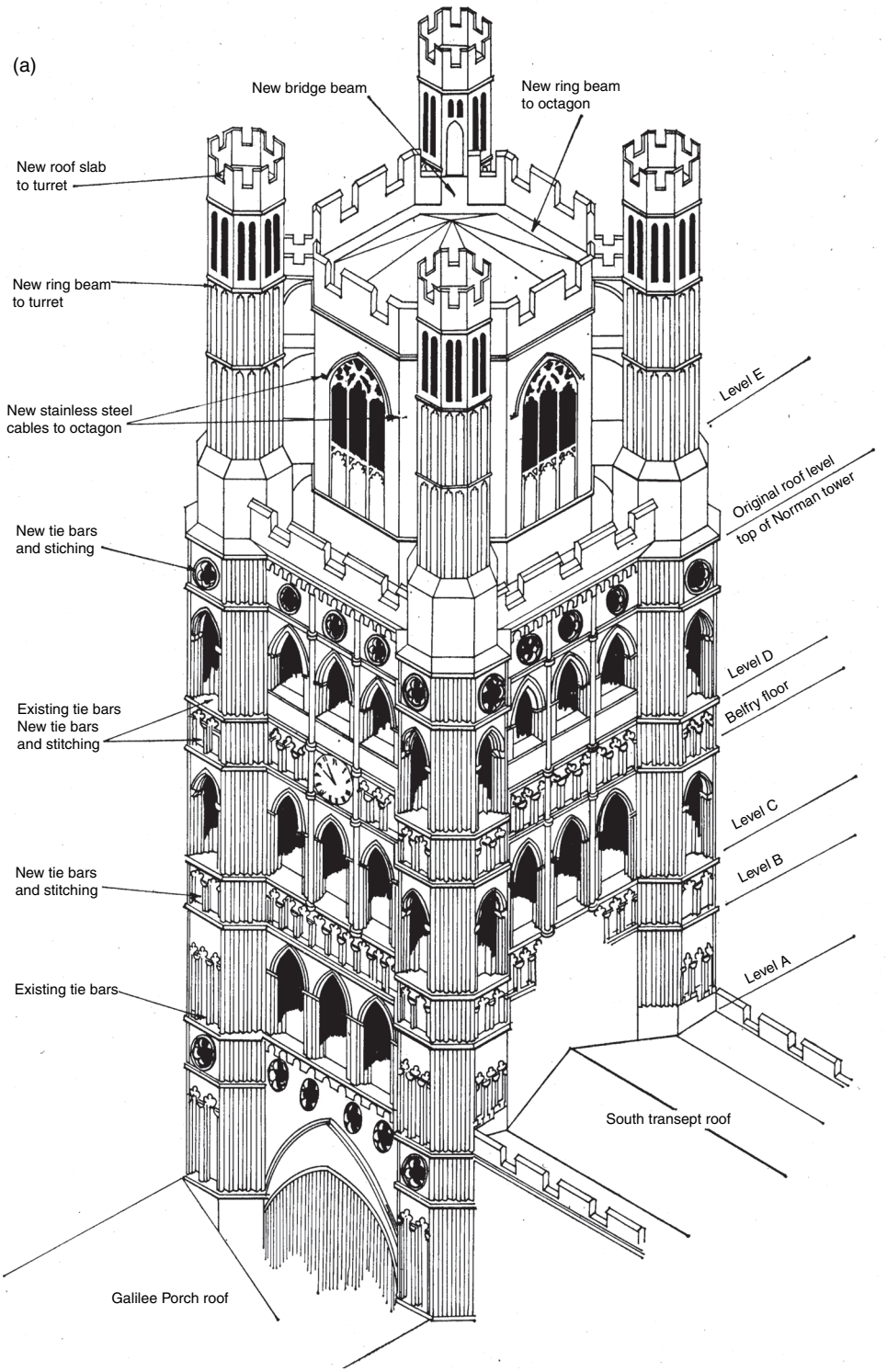


Figure 5.4 (a) The strengthening of the West Tower of Ely Cathedral and associated structural activities

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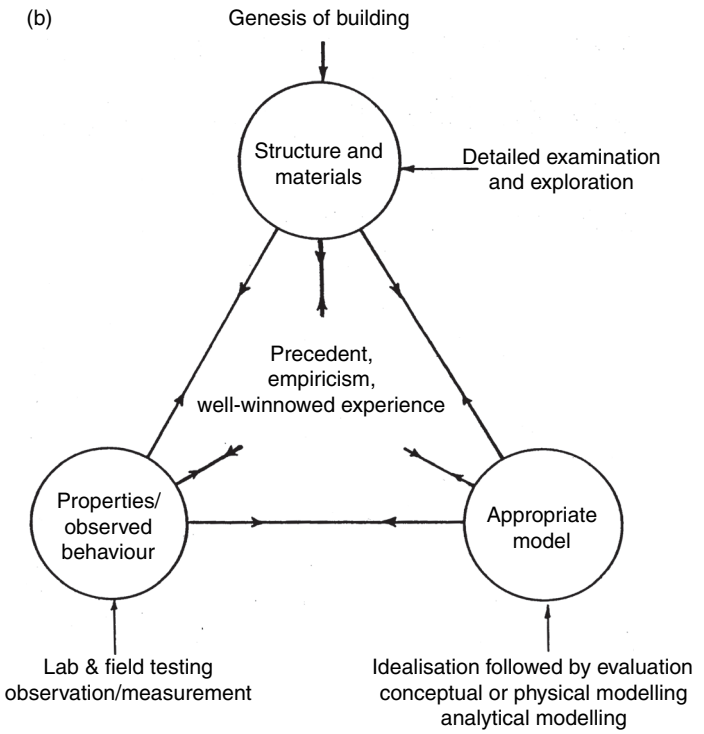


Figure 5.4 cont. (b) The geotechnical triangle changed to show the key activities undertaken by a structural engineer
Derived from Heyman (1976)

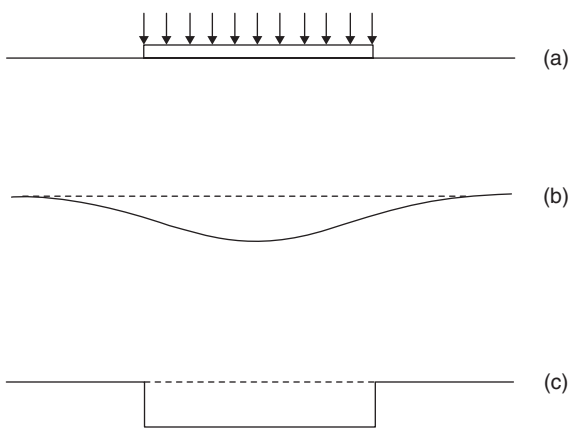


Figure 5.5 (a) Uniformly loaded flexible slab. (b) Settlement profile for slab founded on a clay soil. (c) Settlement profile for soil modelled as springs

In cases where the ground–structure interaction is likely to be important it is essential that the structural engineer and geotechnical engineer interact so that an appropriate approach to modelling can be developed. It is hoped that this chapter will help each to understand the other so as to lead to positive and constructive interaction between them.

5.6 Conclusions

This chapter explores the differences in the philosophy of modelling and analysis between structural and geotechnical engineers. The idealisations that are involved in both disciplines are compared. It has been demonstrated that the approach that has to be adopted by a structural engineer working on an existing historic building is very similar to that of a geotechnical engineer working with the ground.

Superficially it might seem that the idealisations adopted by a structural engineer in routine practice are more certain than those that have to be adopted by a geotechnical engineer. However, on closer examination it has to be concluded that the calculation of the 'actual' state of a structure or building under a known set of loads is very uncertain.

The success of structural design calculations owes much to the inherent ductility of the structural elements, so that the *safe design theorem* applies. Geotechnical engineers, too, owe much to the plastic ductile behaviour of their materials and foundations for the success of their designs. In both cases it is vital to identify brittle behaviour as this invalidates the *safe design theorem* and can lead to progressive collapse.

Structural engineers tend to work and think in terms of forces and stresses. Geotechnical engineers are much more used to working with strain and deformation. This difference becomes particularly apparent for situations in which elements of the building or ground approach their maximum resistance. Structural engineers brought up on concepts of limiting stress find it difficult to accept behaviour that implies the full mobilisation of resistance (which is not the same as 'failure'). Reference to Hambly's three- and four-legged stool (*Hambly's paradox*) greatly aids in the understanding of the above ideas.

Of overriding importance is the need to gain a clear understanding of the mechanisms of behaviour of a soil–structure interaction system. If the analytical, computer or conceptual model does not capture the key mechanism, no amount of sophisticated computation will help solve the problem.

In summary, understanding and designing for the ground–structure interaction requires all the traditional skills of the engineer: reliance on observation and measurement; a deep understanding of materials, both ground and structural; the development of appropriate physical and analytical models to reveal the underlying mechanisms of behaviour; and *well-winnowed experience* based on a discerning knowledge of precedents and case histories.

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5.7.1 Further reading

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

Chapter 6

Computer analysis principles in geotechnical engineering

David Potts Imperial College London, UK
Lidija Zdravkovic Imperial College London, UK

This chapter categorises various forms of computational analysis in geotechnical engineering and explains both positive and negative aspects of their performance. The conventional methods of analysis are considered first, followed by some of the main aspects of advanced numerical analysis.

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6.1 General

Geotechnical analysis is part of the geotechnical design process, performed with the aim of assessing (i) the stability and serviceability of the new construction and (ii) the effects that the new construction imposes on existing adjacent structures and services. With respect to this, there are several forms of stability to be confirmed by geotechnical calculations. Firstly, it must be checked that the structure and its support system are stable as a whole, with no danger of promoting either local failure of the structure itself, or global failure involving the surrounding soil domain. Secondly, the loads on any structural elements must be calculated so that the structures can be designed to carry them safely. Finally, movements of both the structure and the ground must be evaluated, in particular if there are structures and services nearby. It may also be necessary to calculate any structural forces induced in the existing structures and services, to assess whether they can be safely sustained.

As part of the design process, it is necessary for an engineer to perform calculations to obtain estimates of the above quantities. Analysis provides the mathematical framework for such calculations. However, the usefulness of an analysis depends on its input. Only a simulation involving realistic material properties and loading conditions enables a proper understanding of the problem.

6.2 Theoretical classification of analysis methods

For the analysis of a geotechnical problem to be theoretically exact, it has to satisfy the following four requirements: equilibrium, compatibility, constitutive behaviour and boundary conditions.

6.2.1 Equilibrium

Equilibrium under static conditions implies that the applied external loads to the soil domain are fully counteracted by the internal stresses generated within the soil domain. External loads are any concentrated forces and distributed loads on the boundaries of the soil domain, as well as any body forces such as the soil's self-weight. In general three dimensional (3D) stress space the internal stresses have six components, three direct and three shear stresses, in the three coordinate directions. This theoretical requirement can be written in terms of the following equilibrium equations, for each of the coordinate directions:

$$\sum F_x = \frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{yx}}{\partial y} + \frac{\partial \tau_{zx}}{\partial z} + \gamma = 0$$

$$\sum F_y = \frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{zy}}{\partial z} = 0$$

$$\sum F_z = \frac{\partial \tau_{xz}}{\partial x} + \frac{\partial \tau_{yz}}{\partial y} + \frac{\partial \sigma_z}{\partial z} = 0$$

where x, y and z are the coordinate directions; σ_x, σ_y and σ_z are the direct stresses in three coordinate directions; and τ_{xy}, τ_{xz} and τ_{yz} are the three shear stresses. The only external load in the above equation is the self-weight, γ, which is assumed to act in the x-direction.

6.2.2 Compatibility

Compatibility is a requirement related to the soil's deformations. If the applied loads do not result in rupture or overlapping of the soil, such deformations are considered to be compatible. Otherwise, they are non-compatible. Compatible deformations can be ensured by defining consistent strains. These strains are calculated from the displacements of the soil element, which

Method of analysis		Theoretical requirements: Equilibrium	Compatibility	Constitutive behaviour	Boundary conditions: Force	Displacement
Closed form		✓	✓	Linear elastic	✓	✓
Limit equilibrium		✓	✗	Rigid with failure criterion	✓	✓
Stress field		✓	✗	Rigid with failure criterion	✓	✗
Limit analysis	LB (Lower bound)	✓	✗	Ideal plasticity	✓	✗
	UB (Upper bound)	✗	✓		✗	✓
Beam-spring approach		✓	✓	Soil modelled by springs	✓	✓
Full numerical analysis		✓	✓	Any	✓	✓

Table 6.1 Theoretical requirements satisfied by various methods of analysis

have three components, u, v and w, in the three coordinate directions, x, y and z, respectively. They are:

$$\varepsilon_x = \frac{\partial u}{\partial x};\ \varepsilon_y = \frac{\partial v}{\partial y};\ \varepsilon_z = \frac{\partial w}{\partial z}$$

$$\gamma_{xy} = \frac{\partial v}{\partial y} + \frac{\partial u}{\partial y};\ \gamma_{yz} = \frac{\partial v}{\partial z} + \frac{\partial w}{\partial y};\ \gamma_{xz} = \frac{\partial w}{\partial x} + \frac{\partial u}{\partial z}.$$

Similar to stresses, there are three direct strains, ε_x, ε_y and ε_z, and three shear strains, γ_{xy}, γ_{yz} and γ_{xz}.

6.2.3 Constitutive behaviour

For analysis purposes the behaviour of the soil, in terms of its load-deformation characteristics, is expressed through a set of constitutive equations. These provide a link between stresses and strains, and hence a link between equilibrium and compatibility. Depending on the stress–strain relationship, a soil can be characterised as one of the following: ductile, brittle, strain hardening or strain softening.

For calculation purposes the constitutive behaviour can be expressed in the following mathematical form:

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \\ \tau_{yz} \\ \tau_{xz} \end{Bmatrix} = \begin{bmatrix} D_{11} & D_{12} & D_{13} & D_{14} & D_{15} & D_{16} \\ D_{21} & D_{22} & D_{23} & D_{24} & D_{25} & D_{26} \\ D_{31} & D_{32} & D_{33} & D_{34} & D_{35} & D_{36} \\ D_{41} & D_{42} & D_{43} & D_{44} & D_{45} & D_{46} \\ D_{51} & D_{52} & D_{53} & D_{54} & D_{55} & D_{56} \\ D_{61} & D_{62} & D_{63} & D_{64} & D_{65} & D_{66} \end{bmatrix} \cdot \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \\ \gamma_{yz} \\ \gamma_{xz} \end{Bmatrix}$$

or, more succinctly:

$$\{\boldsymbol{\sigma}\} = [\boldsymbol{D}] \cdot \{\boldsymbol{\varepsilon}\}$$

where D_{ij} (i=1,6; j=1,6) are the components of the constitutive matrix $[\boldsymbol{D}]$ and $\{\boldsymbol{\sigma}\}$ and $\{\boldsymbol{\varepsilon}\}$ are vectors of stress and strain components, respectively. The elements D_{ij} of the constitutive matrix represent soil properties (e.g. Young's modulus, Poisson's ratio, angle of shearing resistance) and depend on the complexity of soil behaviour and the constitutive model applied to simulate it.

6.2.4 Boundary conditions

Boundary conditions are any restrictions applied on the boundaries of the soil domain which have to be satisfied in order that equilibrium, compatibility and constitutive behaviour are not violated. They can be applied in terms of forces and displacements.

6.2.5 Categories of analysis methods

As stated at the beginning of this section, all four theoretical requirements have to be satisfied for the analysis solution to be exact. It is therefore useful to review the broad categories of analysis currently in use against these theoretical requirements.

Current methods of analysis can be grouped into the following categories: closed form, simple (or classical) and numerical analysis. The ability of each method to satisfy the fundamental theoretical requirements and provide design information are summarised in **Tables 6.1** and **6.2**.

Method of analysis		Design requirements: Stability	Movements	Adjacent structures
Closed form		✗	✓	✗
Limit equilibrium		✓	✗	✗
Stress field		✓	✗	✗
Limit analysis	LB (Lower bound)	✓	✗	✗
	UB (Upper bound)	✗	Crude estimate	✗
Beam-spring approach		✓	✗	✗
Full numerical analysis		✓	✓	✓

Table 6.2 Design requirements satisfied by various methods of analysis

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6.3 Closed form solutions

A closed form solution is in many ways the ultimate method of analysis. If it is possible, for a particular geotechnical structure, to establish realistic constitutive models for material behaviour and appropriate boundary conditions, and to combine these with the equations of equilibrium and compatibility, then an exact solution can be obtained which satisfies all of the above theoretical requirements.

However, as soil is a highly complex multi-phase material which behaves nonlinearly when loaded, a closed form solution to realistic geotechnical problems is not normally possible. Closed form solutions are only possible for two very simple situations:

(i) The soil is an isotropic linear elastic material. The solution provides an estimate of movements and structural forces, but cannot be used for assessing stability. However, comparison with observed behaviour of geotechnical structures indicates that even the predictions of movement are often not realistic.

(ii) This situation involves elasto-plastic problems, where the geometry of the problem contains sufficient geometric symmetries such that it can be reduced to being essentially one dimensional. Examples of this are solutions for the expansion of spherical and infinitely long cylindrical cavities in an infinite elasto-plastic continuum.

6.4 Classical methods of analysis

Classical methods of analysis enable more realistic solutions to geotechnical problems; however, they also involve approximations. Whereas mathematics is still used to obtain a solution, one of the theoretical requirements may not be satisfied. Therefore, the solution is not exact in a theoretical sense. This approach was adopted by the pioneers of geotechnical engineering, when computer power was non-existent and it was therefore impossible to obtain a complete solution utilising purely manual calculations.

This group of analysis methods consists of the limit equilibrium, stress field and limit analysis methods. They consider only the ultimate limit states when the soil is at failure, but differ in the manner in which they arrive at a solution.

6.4.1 Limit equilibrium method

This method of analysis involves the following steps:

- An arbitrary failure surface is adopted for the problem, i.e. its position and shape, which can be either planar or curved, or a combination of both.
- The failure criterion is assumed to hold everywhere along the failure surface.
- Only the global equilibrium of the rigid blocks of soil between the failure surfaces and the boundaries of the problem is considered; external forces acting on the blocks include the self-weight; internal forces (stresses) are generated only along the failure surfaces.
- The internal stress distribution within the blocks of soil is not considered (i.e. they are assumed to be rigid).

The Coulomb wedge analysis for retaining walls and the method of slices for slopes are examples of limit equilibrium calculations. These calculations do not consider the compatibility requirement and it is therefore not possible to obtain any information about the possible movements of the soil.

6.4.2 Stress field method

As it can be seen in **Table 6.1**, the stress field method satisfies the same theoretical requirements as the limit equilibrium method of analysis and adopts the same assumption for soil behaviour (i.e. rigid with a failure criterion). However, there is an important difference in the assumptions of this method: while the limit equilibrium method assumes failure in the soil only along the failure surface, the stress field method assumes failure everywhere in the soil domain. Therefore, the equilibrium equations in the stress field method have to be written for an infinitesimal soil element in the form of partial differential equations. This complicates the solution process as it leads, when combined with the equations of the failure criterion, to a set of hyperbolic partial differential equations. These equations can only be solved analytically if the soil is assumed to be weightless. While a weightless soil is clearly unrealistic there are some scenarios where the soil's weight does not affect the solution and consequently analytical solutions are applicable. For example, appropriate analytical solutions are available for some boundary value problems where the soil behaves in an undrained manner with a Tresca (S_u) failure criterion (i.e. the bearing capacity of surface footings). For boundary value problems where weight is important and/or the soil behaves in a drained manner with a Mohr–Coulomb failure criterion, the governing partial differential equations must be solved using an approximate finite difference approach. Again, because compatibility is not considered, the method does not provide any information about ground movements and considers only the ultimate limit states (i.e. failure).

Examples of stress field solutions are the Rankine active and passive stress fields for earth pressure calculations and the earth pressure tables of Sokolovski (1960, 1965). The stress field method is also used for solving bearing capacity problems. The Prandtl (1920) and Hill (1950) stress fields for a rigid strip footing are examples. Computer programs are also available for performing stress field calculations for problems with complex geometries (Martin, 2005).

6.4.3 Limit analysis method

Limit analysis calculations provide bounds to the exact solution. The upper bound (UB) approach results in an unsafe estimate of the exact solution and arrives at the solution by ignoring equilibrium. The lower bound (LB) approach provides a safe estimate of the exact solution by ignoring the compatibility requirement. If the two solutions for the same problem are the same, then an exact solution to the problem is obtained.

In contrast to the limit equilibrium and stress field methods, the soil behaviour is assumed to be ideal elasto-plastic rather than rigid.

6.4.3.1 Safe theorem

If a statically admissible stress field covering the whole soil mass can be found, which nowhere violates the yield condition, then the loads in equilibrium with the stress field are on the safe side or equal to the true collapse loads.

This theorem is often referred to as the lower bound theorem. A statically admissible stress field consists of an equilibrium distribution of stress which balances the applied loads and body forces. As compatibility is not considered, an infinite number of stress fields can be postulated. The accuracy of the solution depends on how close the assumed stress field is to the real one for a given problem.

6.4.3.2 Unsafe theorem

An unsafe solution to the true collapse loads can be found by selecting any kinematically possible failure mechanism and performing a work rate calculation. The loads so determined are either on the unsafe side or equal to the true collapse loads.

This theorem is also known as the upper bound theorem. A kinematically possible failure mechanism is one that satisfies the compatibility requirements in the shear zones between blocks of soil in the assumed mechanism. Equilibrium of the work done by external and internal forces is then considered, instead of the equilibrium of the external and internal forces. Because the latter is not considered, an infinite number of failure mechanisms can be assumed. The accuracy of the solution depends on how close the assumed mechanism is to the real one in a given problem.

In practice it is often easier to postulate a realistic failure mechanism than an admissible stress field; consequently more use has historically been made of the unsafe theorem. It is rare to be able to obtain the same solution from both safe and unsafe calculations and when two such solutions are obtained, they usually provide differing answers which bracket the true solution – the true solution being that applicable to the ideal soil behaviour assumed to develop the limit theorems. To the authors' knowledge, there are only two geotechnical boundary value problems in which both safe and unsafe calculations have yielded the same result. One is the short term lateral load that can be sustained by an infinitely long cylindrical pile in an infinite clay deposit, and the other is the bearing capacity of surface strip footings.

Recently, considerable research effort has been expended in combining limit analysis with finite element techniques to develop an analysis tool that can deal with complex boundary value problems involving mixed soils (Sloan, 1988 a, b). Such a tool looks promising and should provide an alternative method for analysing ultimate limit states.

6.5 Numerical analysis

6.5.1 Beam-spring approach

This approach is used to investigate soil–structure interaction. For example, it can be used to study the behaviour of axially and laterally loaded piles, raft foundations, embedded retaining walls and tunnel linings. The major approximation is the assumed soil behaviour and two approaches are commonly used. The soil behaviour is either approximated by a set of unconnected vertical and horizontal springs (Borin, 1989), or by a set of linear elastic interaction factors (Pappin *et al*., 1985). Only a single structure can be accommodated in the analysis. Consequently, only a single pile or retaining wall can be analysed at a time. Further approximations must be introduced if more than one pile, retaining wall, or foundation, interact. Any structural support, such as props or anchors (retaining wall problems) is represented by simple springs (see **Figure 6.1**).

To enable limiting pressures to be obtained, for example on each side of a retaining wall, 'cut offs' are usually applied to the spring forces and interaction factors representing soil behaviour. These cut off pressures are usually obtained from one of the simple analysis procedures discussed above (e.g. limit equilibrium, stress fields or limit analysis). It is important to appreciate that these limiting pressures are not a direct result of the beam-spring calculation, but are obtained from separate approximate solutions and then imposed on the beam-spring calculation process.

Having reduced the boundary value problem to studying the behaviour of a single isolated structure (e.g. a pile, a footing, or

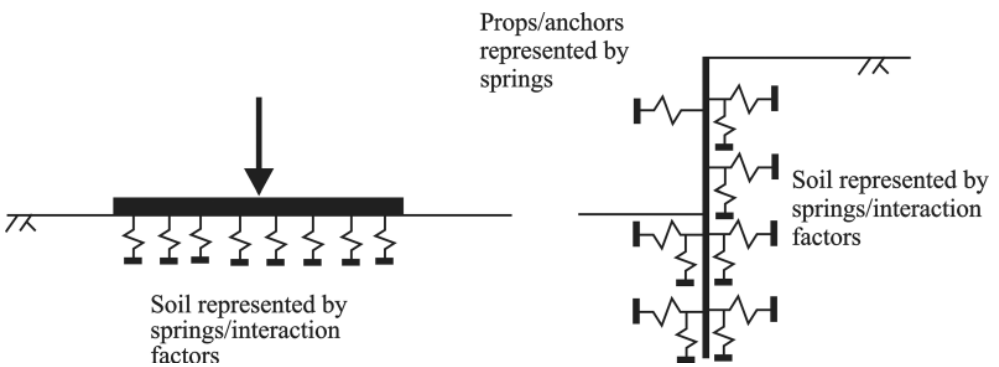


Figure 6.1 Examples of the beam-spring approach

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a retaining wall) and made gross assumptions about soil behaviour, a complete theoretical solution to the problem is sought. Due to the complexities involved, this is usually achieved using a computer. The structural member (e.g. pile, footing, or retaining wall) is represented using either finite differences or finite elements and a solution that satisfies all the fundamental solution requirements is obtained by iteration.

Sometimes computer programs which perform such calculations are identified as finite difference or finite element programs. However, it must be noted that it is only the structural member that is represented in this manner and these programs should not be confused with those that involve full discretisation of both the soil and structural members by finite differences or finite elements, see section 6.5.2.

As solutions obtained in this way include limits to the earth pressures that can develop adjacent to the structure, they can provide information on local stability. This is often indicated by a failure of the program to converge. However, numerical instability may occur for other reasons and therefore a false impression of instability may be given. Solutions from these calculations include forces and movements of the structure. They do not provide information about global stability or movements in the adjacent soil. They do not consider adjacent structures.

It is difficult to select appropriate spring stiffness values and to simulate some support features. For example, when analysing a retaining wall it is difficult to account realistically for the effects of soil berms. Retaining wall programs using interaction factors to represent the soil can have problems in dealing with wall friction and often neglect shear stresses on the wall, or make further assumptions to deal with them. For the analysis of retaining walls, a single wall is considered in isolation and structural supports are represented by simple springs fixed at one end (grounded). It is therefore difficult to account for realistic interaction between structural components such as floor slabs and other retaining walls. This is particularly so if 'pin-jointed' or 'full moment' connections are appropriate. As only the soil acting on the wall is considered in the analysis, it is difficult to model realistically the behaviour of raking props and ground anchors which rely on resistance from soil remote from the wall.

6.5.2 Full numerical analysis

This category of analysis includes methods which attempt to satisfy all theoretical requirements, includes realistic soil constitutive models and incorporates boundary conditions that realistically simulate field conditions. Because of the complexities involved and the nonlinearities in soil behaviour, all methods are numerical in nature. The most widely used approaches are those based on finite difference and finite element methods. These methods essentially involve a computer simulation of the history of the boundary value problem from green field conditions, throughout the construction phase and into the long term.

The ability of full numerical analysis to accurately reflect field conditions essentially depends on (i) the ability of the constitutive model to represent real soil behaviour and (ii) the correctness of the boundary conditions imposed. The user has to define the appropriate geometry, construction procedure, soil parameters and boundary conditions. Structural members may be added and withdrawn during the numerical simulation to model field conditions. Retaining structures composed of several retaining walls interconnected by structural components can be considered and, because the soil mass is modelled in the analysis, the complex interaction between raking struts or ground anchors and the soil can be accounted for. The effect of time on the development of pore water pressures can also be simulated by including coupled consolidation. No postulated failure mechanism or mode of behaviour of the problem is required, as these are predicted by the analysis. The analysis allows the complete history of the boundary value problem to be predicted and a single analysis can provide information on all design requirements.

Potentially, the methods can solve full three dimensional problems and suffer none of the limitations discussed previously for the other methods. At present, the speed of computer hardware restricts analysis of most practical problems to two dimensional plane strain or axisymmetric sections. However, with the rapid development in computer hardware and its reduction in cost, the possibilities of full three dimensional simulations are imminent.

It is often claimed that these approaches have limitations. Usually these relate to the fact that detailed soils information or knowledge of the construction procedure is needed. In the authors' opinion, neither of these are limitations. If a numerical analysis is anticipated during the design stages of a project, it is then not difficult to ensure that the appropriate soil information is obtained from the site investigation. It is only if a numerical analysis is an afterthought, once the soil data has been obtained, that this may present difficulties. In this respect it is noted that site investigations are not always performed to an adequate standard to obtain certain parameters, and that not all parameters for advanced constitutive models can be directly obtained from the site investigation.

If the behaviour of the boundary value problem is not sensitive to the construction procedure, then any reasonable assumed procedure is adequate for the analysis. However, if the analysis is sensitive to the construction procedure then, clearly, this is important and it will be necessary to simulate the field conditions as closely as possible. Therefore, far from being a limitation, numerical analysis can indicate to the design engineer where, and by how much, the boundary value problem is likely to be influenced by the construction procedure. This will enable adequate provision to be made within the design.

Full numerical analyses are complex and should be performed by qualified and experienced staff. The operator must understand soil mechanics and, in particular, the constitutive models that the software uses, and be familiar with the software package to be employed for the analysis. Nonlinear numerical analysis is not straightforward and at present there are several algorithms (i.e. solution procedures) available for solving the nonlinear system of governing equations. Some are more accurate than others and

some are increment-size dependent. There are approximations within these algorithms and errors associated with discretisation. However, these can be controlled by the experienced user so that accurate predictions can be obtained.

Full numerical analysis can be used to predict the behaviour of complex field situations. It can also be used to investigate the fundamentals of soil–structure interaction and to calibrate some of the simple methods discussed above. However, as noted above, numerical analyses involve approximations and there are many pitfalls that can be encountered. Some of these approximations and pitfalls are considered below for the finite element method. However, it should be noted that similar considerations apply to finite difference approaches.

6.6 Overview of the finite element method

The finite element method involves the following steps.

(i) Element discretisation
This is the process of modelling the geometry of the problem under investigation by an assemblage of small regions, termed finite elements. These elements have nodes defined on the element boundaries, or within the element.

(ii) Primary variable approximation
A primary variable must be selected (e.g. displacements, stresses) and rules as to how it should vary over a finite element established. This variation is expressed in terms of nodal values. In geotechnical engineering, it is usual to adopt displacements as the primary variable.

(iii) Element equations
Use of an appropriate variational principle (e.g. minimum potential energy) to derive element equations:

$$[\boldsymbol{K}_E] \cdot \{\Delta \boldsymbol{d}_E\} = \{\Delta \boldsymbol{R}_E\}$$

where $[\boldsymbol{K}_E]$ is the element stiffness matrix, $\{\Delta \boldsymbol{d}_E\}$, is the vector of incremental element nodal displacements and $\{\Delta \boldsymbol{R}_E\}$ is the vector of incremental element nodal forces. It should be noted that the element stiffness matrix $[\boldsymbol{K}_E]$ is derived from the constitutive matrix $[D]$ discussed previously in section 6.2.3.

(iv) Global equations
Combine element equations to form global equations:

$$[\boldsymbol{K}_G] \cdot \{\Delta \boldsymbol{d}_G\} = \{\Delta \boldsymbol{R}_G\}$$

where $[\boldsymbol{K}_G]$ is the global stiffness matrix, $\{\Delta \boldsymbol{d}_G\}$ is the vector of all incremental nodal displacements and $\{\Delta \boldsymbol{R}_G\}$ is the vector of all incremental nodal forces.

(v) Boundary conditions
Formulate boundary conditions and modify the global equations. Loadings (e.g. line and point loads, pressures, body forces, construction and excavation) affect $\{\Delta \boldsymbol{R}_G\}$, while the prescribed displacements affect $\{\Delta \boldsymbol{d}_G\}$.

(vi) Solution of global equations
The above global equations are in the form of a large number of simultaneous equations. These are solved to obtain the displacements $\{\Delta \boldsymbol{d}_G\}$ at all the nodes. From these nodal displacements, secondary quantities – such as stresses and strains – are evaluated.

Several approximations are involved in the above process. Some of these are now considered in detail in the following sections.

6.7 Element discretisation

The geometry of the boundary value problem under investigation must be defined and quantified. Simplifications and approximations may be necessary during this process. For example, restraints on computational resources might impose that plane strain conditions have to be assumed. This geometry is then replaced by an equivalent finite element mesh which is composed of small regions called finite elements. For two dimensional problems, the finite elements are usually triangular or quadrilateral in shape, see **Figure 6.2**. Their geometry is specified in terms of the coordinates of key points on the elements called nodes. For elements with straight sides these nodes are usually located at the element corners. If the elements have curved sides then additional nodes, usually at the midpoint of each side, must be introduced. The set of elements in the complete mesh are connected together by the element sides and a number of nodes.

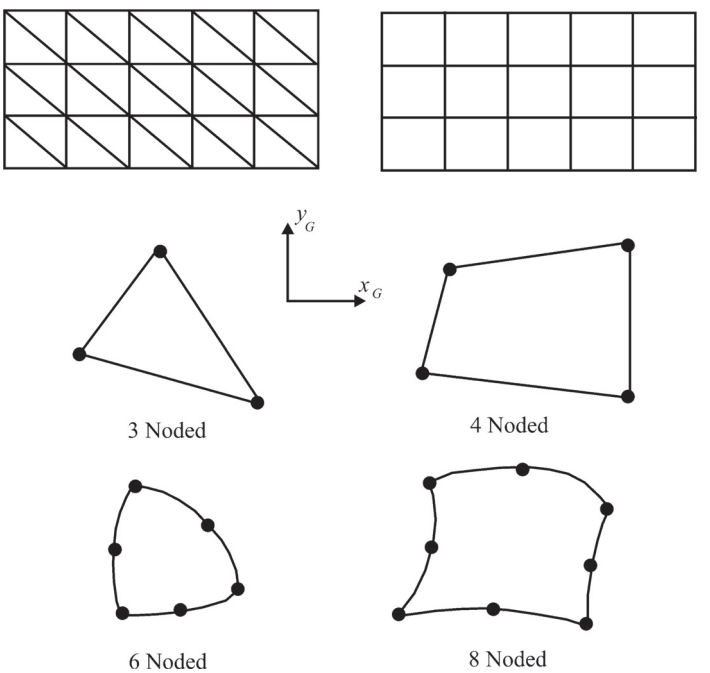


Figure 6.2 Typical 2D finite elements

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When constructing the finite element mesh the following should be considered:

- The geometry of the boundary value problem must be approximated as accurately as possible.
- If there are curved boundaries or curved material interfaces, the higher order elements, with mid-side nodes should be used, see **Figure 6.3**.
- In many cases, geometric discontinuities suggest a natural form of subdivision. For example, discontinuities in boundary gradient, such as re-entrant corners or cracks, can be modelled by placing nodes at the discontinuity points. Interfaces between materials with different properties can be introduced by element sides, see **Figure 6.4**.
- Mesh design may also be influenced by the applied boundary conditions. If there are discontinuities in loading, or point loads, these may be introduced by placing nodes at the discontinuity points, see **Figure 6.5**.

In combination with the above factors, the size and the number of elements depend largely on the material behaviour which influences the final solution. For linear material behaviour, the procedure is relatively straightforward and only the zones where unknowns vary rapidly need special attention. In order to obtain accurate solutions, these zones require a refined mesh of smaller elements. The situation is more complex for general nonlinear material behaviour, since the final solution may depend, for example, on the previous loading history. For such problems, the mesh design must take into account the boundary conditions, the material properties and, in some cases, the geometry, all of which vary throughout the solution process.

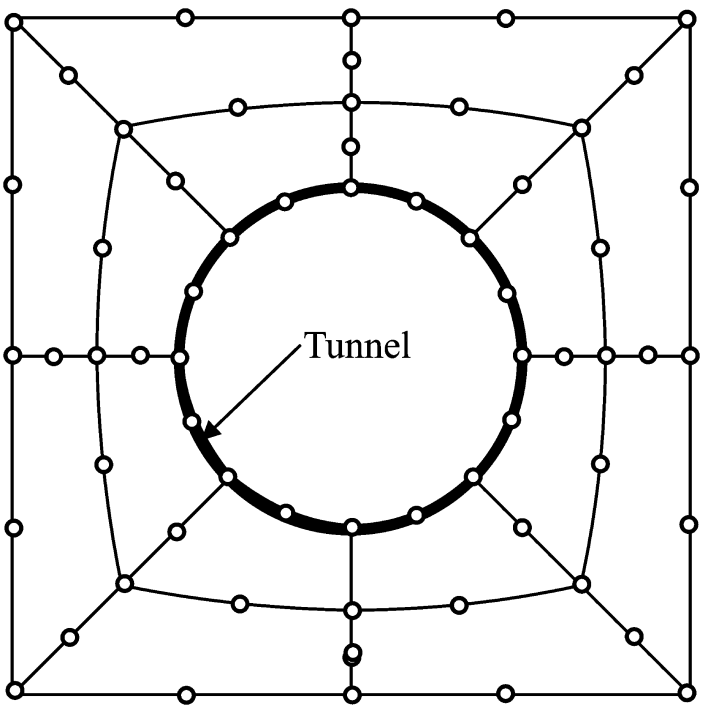


Figure 6.3 Use of higher order elements

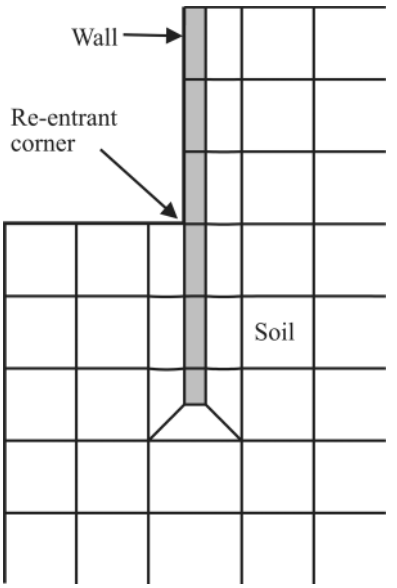


Figure 6.4 Effect of boundary conditions

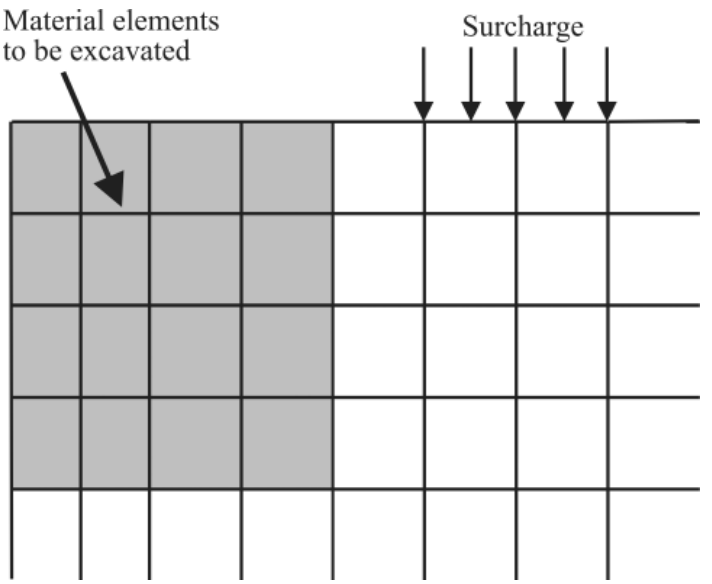


Figure 6.5 Geometric discontinuities

In all cases, a mesh of regularly shaped elements will give the best results. Elements with widely distorted geometries or long thin elements should be avoided.

To illustrate the error that can arise from the discretisation of the problem geometry into finite elements, the behaviour of smooth rigid strip and circular surface foundations under vertical loading will be considered. The soil is assumed to be linear elastic perfectly plastic, with a Tresca yield surface, with the following properties: Young's modulus, $E = 10\,000$ kPa, Poisson's ratio, $\mu = 0.45$ and undrained strength, $S_u = 100$ kPa.

Two alternative finite element meshes are shown in **Figures 6.6** and **6.7**. The mesh shown in **Figure 6.6** has 110 8-noded

isoparametric elements, whereas the mesh shown in **Figure 6.7** has only 35 8-noded elements. Both meshes have been used to perform plane strain (i.e. strip footing) and axisymmetric (circular footing) analyses, in which the foundation is displaced vertically downward to obtain the complete load–displacement curve. From a cursory glance at the two meshes, most users would probably conclude that the 110 element mesh was likely to produce the more accurate predictions due to its larger number of elements and their more even spatial distribution.

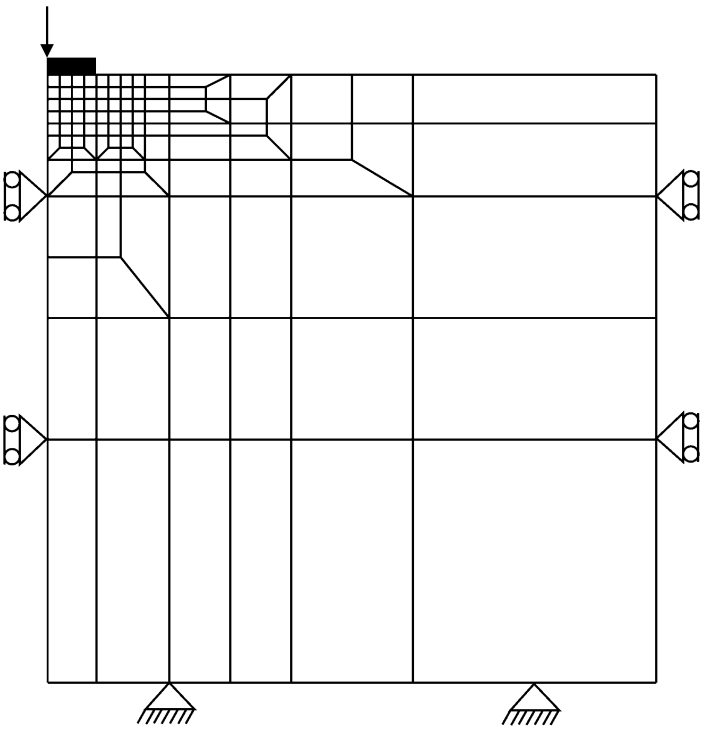


Figure 6.6 Smooth footing using a 110 element mesh

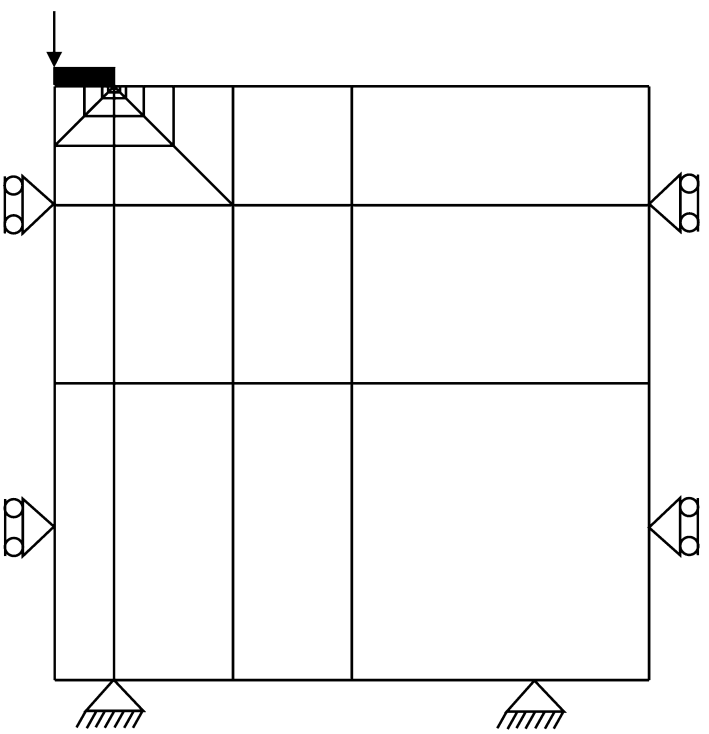


Figure 6.7 Smooth footing using a 35 element mesh

Although there is no analytical solution for the complete load–displacement curve for either the strip or the circular surface footings, there is an analytical solution for the ultimate load for the strip footing which gives $Q_{max} = q_f.2B$ per metre length, where $q_f = N_c S_u$, $N_c = (2 + \pi)$ and $2B$ is the width of the footing. Textbooks often quote an equivalent analytical solution for the circular footing of $Q = q_f.S_u.\pi R^2$, where $q_f = N_c S_u$, $N_c = 5.69$ and R is the radius of the footing. However, this solution is not strictly an analytical one, as it involves the numerical integration of some stress field equations. It is however thought to be reasonably accurate.

Results from finite element analysis using the meshes shown in **Figures 6.6** and **6.7** are presented in **Figure 6.8**. Also marked on this figure are the theoretical limit loads discussed above. Assuming that the theoretical limit loads are correct, then the analyses with the 110 element mesh overestimate the collapse loads by 3.8% for the strip footing, and 8.8% for the circular footing, respectively. However, the errors for the analyses performed using the 35 element mesh are much smaller: 0.5% for the strip and 2% for the circular footing. The reason for the relatively bad performance of the 110 element mesh can be seen in **Figure 6.9**, which shows vectors of incremental nodal displacements for the last increment of the 110 element strip footing analysis. Each vector indicates the magnitude (by its length) and direction (by its orientation) of the associated incremental nodal displacement. While the absolute magnitude of the incremental displacements is not important, the relative magnitude of each of the vectors and their orientation clearly indicates the mechanism of failure. Attention should, however, be focused on the corner of the footing where a rapid change in the direction of the incremental vectors is indicated. The direction changes by approximately 120° between the vector immediately below the corner of the footing and the adjacent node on the soil surface.

Such behaviour clearly indicates that under the corner of the footing there are large stress and strain gradients. In the 110 element mesh, the elements in the vicinity of this corner are too large and of insufficient number to accurately reproduce this concentration of behaviour. In comparison, the 35 element mesh – which has smaller elements and a graded mesh under the corner of the footing, see **Figure 6.10** – can accommodate this behaviour more easily and therefore produces more accurate solutions.

Clearly, it is important that smaller (and therefore more) elements are placed where there are rapid changes in stresses and strains, and therefore directions of movement. Some computer programs will automatically regenerate the finite element mesh to improve the accuracy of the solution. They work by first

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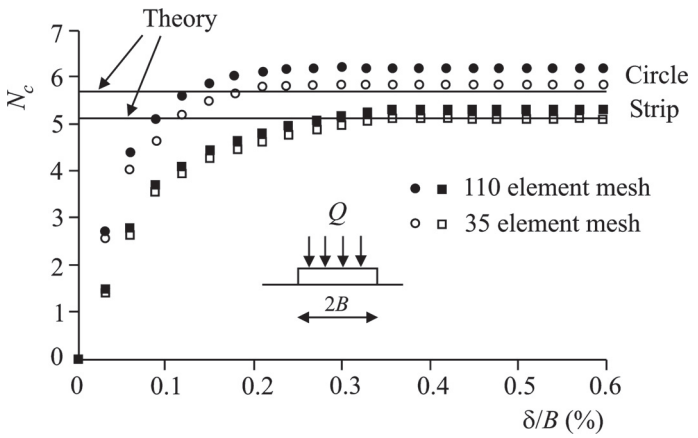


Figure 6.8 Load–displacement curves for a smooth footing

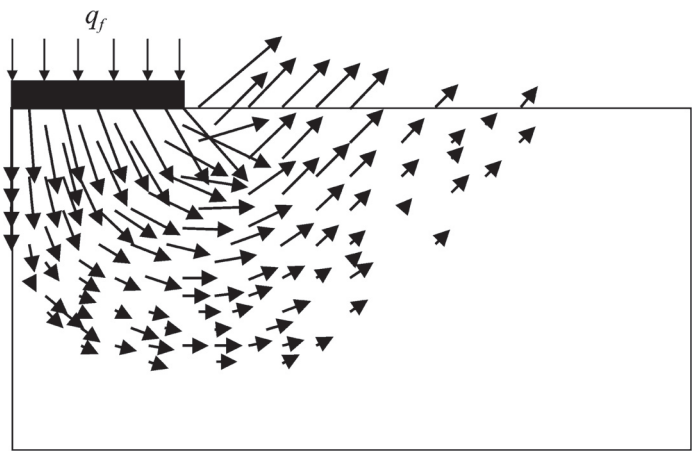


Figure 6.9 Vectors of incremental displacements at failure using a 110 element mesh

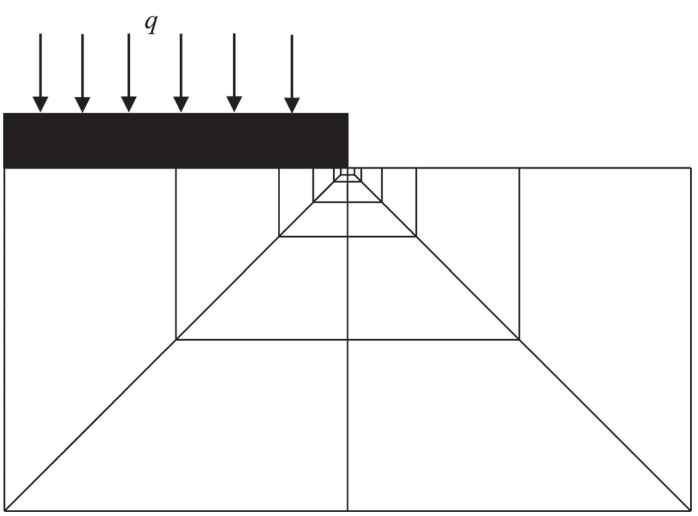


Figure 6.10 Detail from a 35 element mesh

performing a solution with a relatively even mesh. Based on the results of this analysis, a new mesh is generated with more elements in the zones of greatest stress and strain changes. The analysis is then repeated. In principle, this process of refinement could be repeated until the results were unaffected by further mesh refinement.

6.8 Nonlinear finite element analysis

If the soil is nonlinear elastic and/or elasto-plastic, the equivalent constitutive matrix is no longer constant, but varies with stress and/or strain. It therefore changes during a finite element analysis. Consequently, a solution strategy is required that can account for this changing material behaviour. This strategy is a key component of a nonlinear finite element analysis, as it can strongly influence the accuracy of the results and the computer resources required to obtain them.

As noted above, four basic solution requirements need to be satisfied: equilibrium, compatibility, constitutive behaviour and boundary conditions. Nonlinearity introduced by the constitutive behaviour causes the governing finite element equations to be reduced to the following incremental form:

$$\left[\boldsymbol{K}_G\right]^i \cdot \left\{\Delta \boldsymbol{d}_G\right\}^i = \left\{\Delta \boldsymbol{R}_G\right\}^i$$

where $[\boldsymbol{K}_G]^i$ is the incremental global system stiffness matrix, $\{\Delta \boldsymbol{d}_G\}^i$ is the vector of incremental nodal displacements, $\{\Delta \boldsymbol{R}_G\}^i$ is the vector of incremental nodal forces and i is the increment number. To obtain a solution to a boundary value problem, the change in boundary conditions is applied in a series of increments, and for each increment the above equation must be solved. The final solution is obtained by summing the results of each increment. Due to the nonlinear constitutive behaviour, the incremental global stiffness matrix $[\boldsymbol{K}_G]^i$ is dependent on the current stress and strain levels and is therefore not constant, but varies over an increment. Unless a very large number of small increments are used, this variation should be accounted for. Hence the solution of the above equation is not straightforward and different solution strategies exist. The objective of all such strategies is that the solution satisfies the four basic requirements listed above. Three different categories of solution algorithm, which are most popular in existing computer programs, are briefly considered below. For detailed information the reader is referred to Potts and Zdravkovic (1999).

6.8.1 Tangent stiffness method

The tangent stiffness method, sometimes called the variable stiffness method, is the simplest solution strategy. Using this approach, the incremental stiffness matrix $[\boldsymbol{K}_G]^i$ is assumed to be constant over each increment and is calculated using the current stress state at the beginning of each increment. This is equivalent to making a piecewise linear approximation to the nonlinear constitutive behaviour. To illustrate the application of this approach, the simple problem of a uniaxially loaded bar of nonlinear material is considered, see **Figure 6.11**. If this bar is loaded, the true load displacement response is shown in **Figure 6.12**. This might

represent the behaviour of a strain hardening plastic material which has a very small initial elastic domain.

In the tangent stiffness approach the applied load is split into a sequence of increments. In **Figure 6.12** three increments of load are shown, $\Delta \boldsymbol{R}_G{}^1$, $\Delta \boldsymbol{R}_G{}^2$ and $\Delta \boldsymbol{R}_G{}^3$. The analysis starts with the application of $\Delta \boldsymbol{R}_G{}^1$. The incremental global stiffness matrix $[\boldsymbol{K}_G]^1$ for this increment is evaluated based on the unstressed state of the bar corresponding to point **a**. For an elasto-plastic material this might be constructed using the elastic constitutive matrix. The first incremental equation is then solved to determine the nodal displacements $\{\Delta \boldsymbol{d}_G\}^1$. As the material stiffness is assumed to remain constant, the load–displacement curve follows the straight line **ab'** shown in **Figure 6.12**. In reality, the stiffness of the material does not remain constant during this loading increment and the true solution is represented by the curved path **ab**. There is therefore an error in the predicted displacement equal to the distance **b'b**, however in the tangent stiffness approach this error is neglected. The second increment of load, $\Delta \boldsymbol{R}_G{}^2$, is then applied, with the incremental global stiffness matrix $[\boldsymbol{K}_G]^2$ evaluated using the stresses and strains appropriate to the end of increment 1, i.e. point **b'** on **Figure 6.12**. Solving the incremental governing equation then gives the nodal displacements $\{\Delta \boldsymbol{d}_G\}^2$. The load–displacement curve follows the straight path **b'c'** on **Figure 6.12**. This deviates further from the true solution: the error in the displacements now being equal to the distance **c'c**. A similar procedure now occurs when $\Delta \boldsymbol{R}_G{}^3$ is applied. The stiffness matrix $[\boldsymbol{K}_G]^3$ is evaluated using the stresses and strains appropriate to the end of increment 2, i.e. point **c'** on **Figure 6.12**. The load–displacement curve moves to point **d'** and again drifts further from the true solution. Clearly, the accuracy of the solution depends on the size of the load increments. For example, if the increment size was reduced so that more increments were needed to reach the same accumulated load, the tangent stiffness solution would be nearer to the true solution.

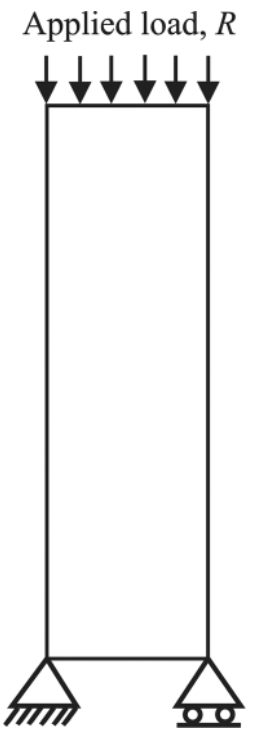


Figure 6.11 Uniaxially loaded bar

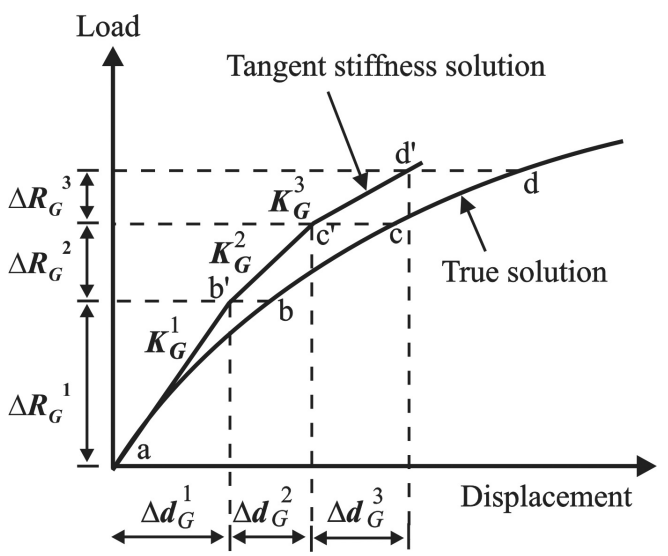


Figure 6.12 Application of the tangent stiffness algorithm

From the above simple example it may be concluded that in order to obtain accurate solutions to strongly nonlinear problems, many small solution increments are required. The results obtained using this method can drift from the true solution and the stresses can fail to satisfy the constitutive relations. Thus the basic solution requirements may not be fulfilled. It can be shown (Potts and Zdravkovic, 1999) that the magnitude of the error is problem-dependent and is affected by the degree of material nonlinearity, the geometry of the problem and the size of the solution increments used. Unfortunately, in general, it is impossible to predetermine the size of solution increment required to achieve an acceptable error.

The tangent stiffness method can give particularly inaccurate results when soil behaviour changes from elastic to plastic or vice versa. For instance, if an element is in an elastic state at the beginning of an increment, it is assumed to behave elastically over the whole increment. This is incorrect if, during the increment, the behaviour becomes plastic and results in an illegal stress state which violates the constitutive model. Such illegal stress states can also occur for plastic elements if the chosen increment size is too large – for example, a tensile stress state could be predicted for a constitutive model which cannot sustain tension. This can be a major problem with critical state type models, such as modified Cam Clay, which employ a v–ln p' relationship (v = specific volume, p' = mean effective stress), since a tensile value of p' cannot be accommodated. In this

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case, either the analysis has to be aborted or the stress state has to be modified in some arbitrary way, which would cause the solution to violate the equilibrium condition and the constitutive model.

6.8.2 Visco-plastic method

This method uses the equations of visco-plastic behaviour and time as an artifice to calculate the behaviour of nonlinear, elasto-plastic, time-independent materials (Owen and Hinton, 1980; Zienkiewicz and Cormeau, 1974).

The method was originally developed for linear elastic visco-plastic (i.e. time dependent) material behaviour. Such a material can be represented by a network of the simple rheological units and is illustrated in **Figure 6.13**. Each unit consists of an elastic and a visco-plastic component connected in series. The elastic component is represented by a spring and the visco-plastic component by a slider and dashpot connected in parallel. If a load is applied to the network, one of two situations occurs in each individual unit. If the load is such that the induced stress in the unit does not cause yielding, the slider remains rigid and all the deformation occurs in the spring (elastic behaviour). Alternatively, if the induced stress causes yielding, the slider becomes free and the dashpot is activated. As the dashpot takes time to react, initially all the deformation occurs in the spring. However, with time, the dashpot moves. The rate of movement of the dashpot depends on the stress it supports and its viscosity. With time progressing, the dashpot moves at a decreasing rate, because some of the stress the unit is carrying is dissipated to adjacent units in the network, which as a result suffer further movements themselves (visco-plastic behaviour). Eventually, a stationary condition is reached where all the dashpots in the network stop moving and are no longer sustaining stresses. This occurs when the stress in each unit drops below the yield surface and the slider becomes rigid. The external load is now supported purely by the springs within the network; but, importantly, straining of the system has occurred not only due to compression or extension of the springs, but also due to movement of the dashpots. If the load was now removed, only the displacements (strains) occurring in the springs would be recoverable, the dashpot displacements (strains) being permanent.

Application to finite element analysis of elasto-plastic materials can be summarised as follows, using the example of a uniaxially loaded bar in **Figure 6.11**. On application of a solution increment, the system is assumed to instantaneously behave linear elastically (see **Figure 6.14**). If the resulting stress state lies within the yield surface, the incremental behaviour is elastic and the calculated displacements are correct. If the resulting stress state violates yield, the stress state can only be sustained momentarily and visco-plastic straining occurs. The magnitude of the visco-plastic strain rate is determined by the value of the yield function – which is a measure of the degree by which the current stress state exceeds the yield condition. The visco-plastic strains increase with time, causing the material to relax with a reduction in the yield function and hence a reduction in the visco-plastic strain rate. A marching technique is used to step forward in time until the visco-plastic strain rate is insignificant. At this point, the visco-plastic strain and the associated stress change are equal to the incremental plastic strain and the incremental stress change respectively.

In order to use the procedure described above, a suitable time step, Δt, must be selected. If Δt is small, many iterations are required to obtain an accurate solution. However, if Δt is too large, numerical instability can occur. The most economical choice for Δt is the largest value that can be tolerated without

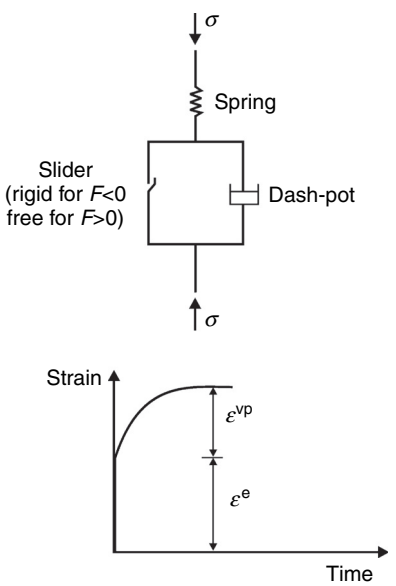


Figure 6.13 Rheological model for visco-plastic material

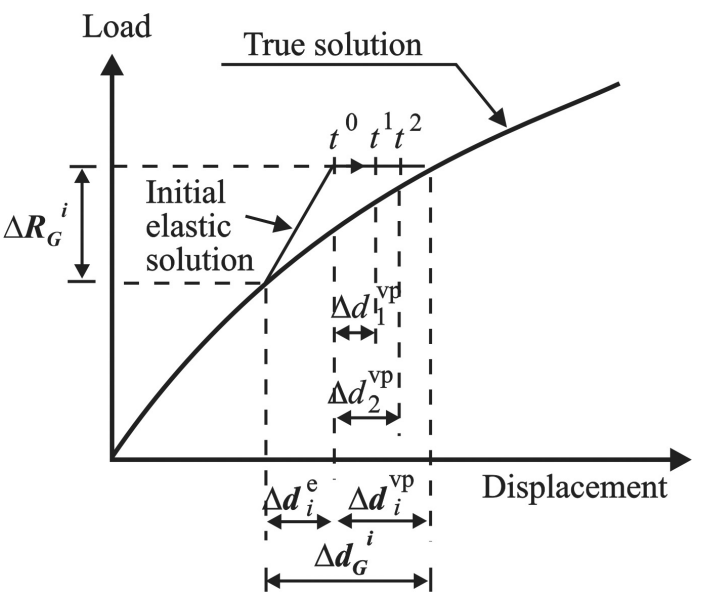


Figure 6.14 Application of the visco-plastic algorithm

causing such instability. For simple constitutive models, such as Tresca and Mohr–Coulomb, it can be shown that the value of the critical time step is dependent only on the elastic stiffness and strength parameters. As these parameters are constant, the critical time step has to be evaluated only once during an analysis. However, for more complex constitutive models, the critical time step is also dependent on the current state of stress and strain and therefore is not constant. It must therefore be continuously evaluated throughout the analyses.

Due to its simplicity, the visco-plastic algorithm has been widely used. However, in the authors' opinion, the method has severe limitations for geotechnical analysis. Firstly, the algorithm relies on the fact that for each increment the elastic parameters remain constant. The simple algorithm cannot accommodate elastic parameters that vary during the increment because, for such cases, it cannot determine the true elastic stress changes associated with the incremental elastic strains. The best that can be done is to use the elastic parameters associated with the accumulated stresses and strains at the beginning of the increment to calculate the elastic constitutive matrix, $[\boldsymbol{D}]$, and assume that this remains constant for the increment. Such a procedure only yields accurate results if the increments are small and/or the elastic nonlinearity is not great. Secondly, a more severe limitation of the method arises when the algorithm is used as an artifice to solve problems involving non-viscous material (i.e. elasto-plastic materials). Here, calculations to determine the plastic strains and elasto-plastic stress changes are evaluated at illegal stress states which lie outside the yield surface. This is theoretically incorrect and results in failure to satisfy the constitutive equations. The magnitude of the error depends on the constitutive model (see Potts and Zdravkovic, 1999).

6.8.3 Modified Newton–Raphson method

In both the tangent stiffness and visco-plastic algorithms, errors can arise because the constitutive behaviour is based on illegal stress states. The modified Newton–Raphson (MNR) algorithm attempts to rectify this problem by only evaluating the constitutive behaviour in, or very near to, legal stress space.

The MNR method uses an iterative technique to solve the system of equations. The first iteration is essentially the same as the tangent stiffness method. However, it is recognised that the solution is likely to be in error and the predicted incremental displacements are used to calculate the residual load, which is a measure of the error in the analysis. The system of equations is then solved again with this residual load, $\{\boldsymbol{\psi}\}$, forming the incremental right hand side vector. The system of equations can be then rewritten as:

$$[\boldsymbol{K}_G]^i \cdot (\{\Delta \boldsymbol{d}_G\}^i)^j = \{\boldsymbol{\psi}\}^j$$

The superscript j refers to the iteration number and $\{\boldsymbol{\psi}\}^0 = \{\Delta \boldsymbol{R}_G\}^i$. This process is repeated until the residual load is small. The incremental displacements are equal to the sum of the iterative displacements. This approach is illustrated in **Figure 6.15** for the simple problem of a uniaxially loaded bar

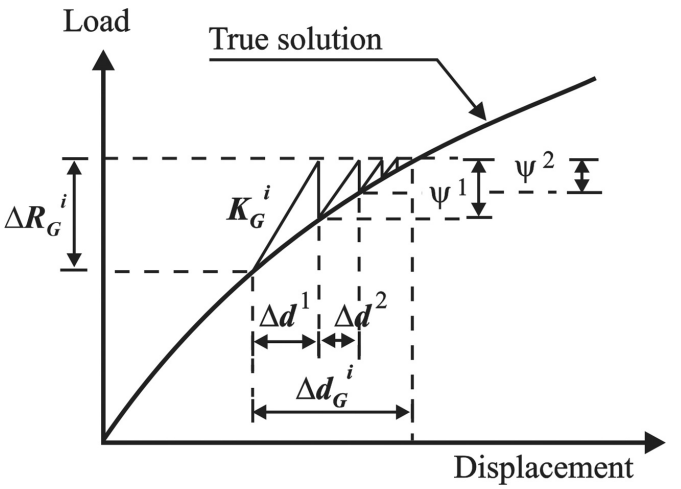


Figure 6.15 Application of the modified Newton–Raphson algorithm

of nonlinear material. In principle, the iterative scheme ensures that for each solution increment, the analysis satisfies all solution requirements.

A key step in this calculation process is determining the residual load vector. At the end of each iteration, the current estimate of the incremental displacements is calculated and used to evaluate the incremental strains at each integration point. The constitutive model is then integrated along the incremental strain paths to obtain an estimate of the stress changes. These stress changes are added to the stresses at the beginning of the increment and used to evaluate consistent equivalent nodal forces. The difference between these forces and the externally applied loads (from the boundary conditions) is the residual load vector. A difference arises because a constant incremental global stiffness matrix $[\boldsymbol{K}_G]^i$ is assumed over the increment. Due to the nonlinear material behaviour, $[\boldsymbol{K}_G]^i$ is not constant but varies with the incremental stress and strain changes.

Since the constitutive behaviour changes over the increment, care must be taken when integrating the constitutive equations to obtain the stress change. Methods of performing this integration are termed *stress point algorithms* and both explicit and implicit approaches have been proposed in various literature. There are many of these algorithms in use and, as they control the accuracy of the final solution, users must verify the approach used in their software (see Potts and Zdravkovic, 1999).

The process described above is called a Newton–Raphson scheme if the incremental global stiffness matrix $[\boldsymbol{K}_G]^i$ is recalculated and inverted for each iteration, based on the latest estimate of the stresses and strains obtained from the previous iteration. To reduce the amount of computation, the modified Newton–Raphson method only calculates and inverts the stiffness matrix at the beginning of the increment and uses it for all iterations within the increment. Sometimes the incremental global stiffness matrix is calculated using the elastic constitutive matrix, $[\boldsymbol{D}]$, rather than the elasto-plastic matrix, $[\boldsymbol{D}^{ep}]$.

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Clearly there are several options here and many software packages allow the user to specify how the MNR algorithm should work. In addition, an acceleration technique is often applied during the iteration process (Thomas, 1984).

6.8.4 Comparison of the solution strategies

A qualitative comparison of the three solution strategies presented above suggests the following. The tangent stiffness method is the simplest, but its accuracy is influenced by increment size. The accuracy of the visco-plastic approach is also influenced by increment size (and by time step size) if complex constitutive models are used. The MNR method is potentially the most accurate and is likely to be the least sensitive to increment size. However, considering the computer resources required for each solution increment, the MNR method is likely to be the most expensive in terms of time required for an analysis; the tangent stiffness method is likely to be the cheapest and the visco-plastic method is probably somewhere in between. However, it may be possible to use larger and therefore fewer increments with the MNR method to obtain a similar accuracy. Thus, it is not obvious which solution strategy is the most economic for a particular solution accuracy.

To illustrate the difference between the solution strategies, two examples will be considered. Firstly, a smooth rigid strip footing subjected to vertical loading, as depicted in **Figure 6.16**, has been analysed. The soil has been modelled using a form of modified Cam Clay and has been assumed to behave undrained. The distribution of undrained strength with depth is shown in **Figure 6.16**. For further information on the constitutive model and the input parameters see Potts and Zdravkovic (1999). The finite element mesh is shown in **Figure 6.17**. Note that due to symmetry about the vertical line through the centre of the footing, only half of the problem needs to be considered in the finite element analysis. Plane strain conditions are assumed. Before loading the footing, the coefficient of earth pressure at rest, K_o, was assumed to be unity, and the vertical effective stress and pore water pressure were calculated using a saturated bulk unit weight of the soil of 20 kN/m^3 and a static water table at the ground surface. The footing was loaded by applying a series of equally sized increments of vertical displacement until the total displacement was 25 mm.

The load–displacement curves for the tangent stiffness, visco-plastic and MNR analyses are presented in **Figure 6.18**. For the MNR method, analyses were performed using 1, 2, 5, 10, 25, 50 and 500 increments to reach a footing settlement of 25 mm. With the exception of the analysis performed with only a single increment, all analyses gave very similar results and plot as a single curve (marked MNR on this figure). The MNR results are therefore insensitive to increment size and show a well defined collapse load of 2.8 kN/m.

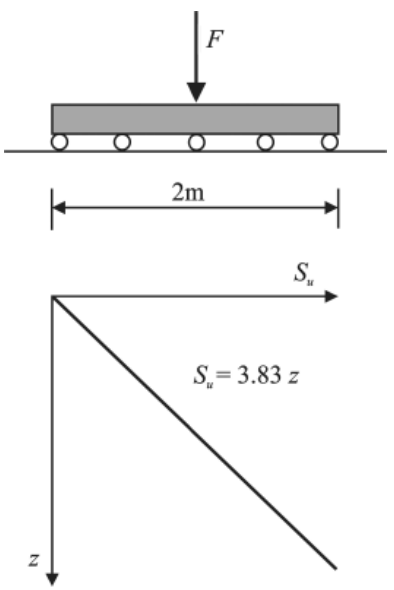


Figure 6.16 Geometry of footing

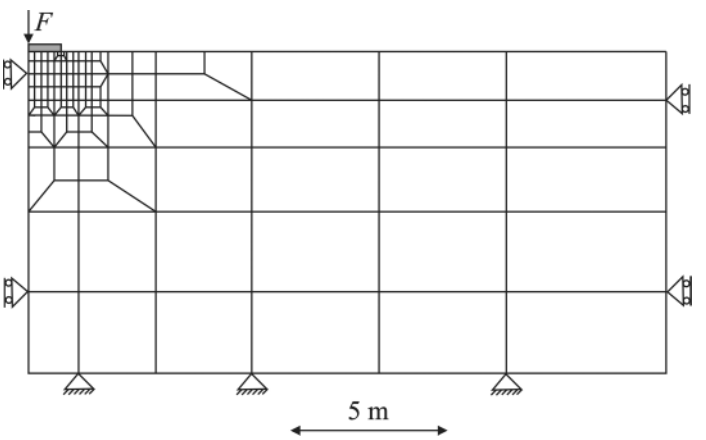


Figure 6.17 Finite element mesh for footing analyses

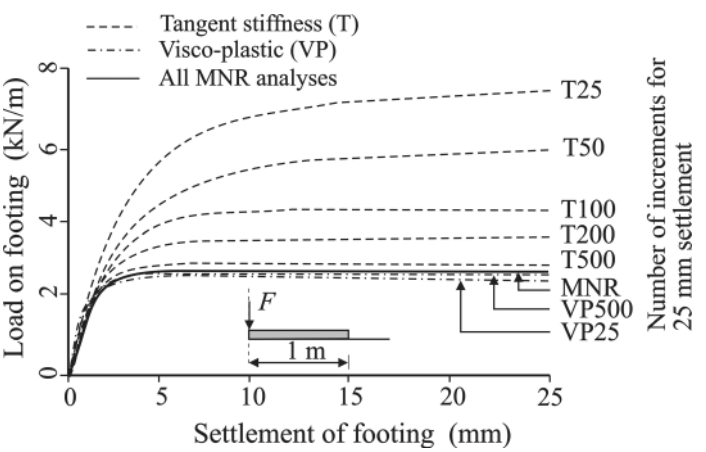


Figure 6.18 Footing load–displacement curves

For the tangent stiffness approach, analyses using 25, 50, 100, 200, 500 and 1000 increments have been carried out. Analyses with a smaller number of increments were also attempted, but illegal stresses (negative mean effective stresses, p') were predicted. As the constitutive model is not defined for such stresses, the analyses had to be aborted. Some finite element packages overcome this problem by arbitrarily resetting the offending negative p' values. There is no theoretical basis for this and it leads to violation of both the equilibrium and the constitutive conditions. Although such adjustments enable an analysis to be completed, the final solution is in error.

Results from the tangent stiffness analyses are shown in **Figure 6.18**. When plotted, the curve from the analysis with 1000 increments is indistinguishable from those of the MNR analyses. The tangent stiffness results are strongly influenced by increment size – the ultimate footing load decreasing from 7.5 to 2.8 kN/m as the applied displacement increment size reduces. There is also a tendency for the load–displacement curve to continue to rise and not to reach a well defined ultimate failure load for the analysis with large applied displacement increments. The results are unconservative, over predicting the ultimate footing load. There is also no indication from the shape of the tangent stiffness load–displacement curves as to whether the solution is accurate, since all the curves have similar shapes.

Visco-plastic analyses with 10, 25, 50, 100 and 500 increments were also performed. The 10 increment analysis had convergence problems in the iteration process; it would initially converge, but then diverge. Similar behaviour was encountered for analyses using still fewer increments. Results from the analyses with 25 and 500 increments are shown in **Figure 6.18**. The solutions are sensitive to increment size, but to a lesser degree than the tangent stiffness approach.

The load on the footing at a settlement of 25 mm is plotted against the number of increments, for tangent stiffness, visco-plastic and MNR analyses in **Figure 6.19**. The insensitivity of the MNR analyses to increment size is clearly shown. In these analyses the ultimate footing load only changed from 2.83 to 2.79 kN/m as the number of increments increased from 2 to 500. Even for the MNR analyses performed with a single increment; the resulting ultimate footing load of 3.13 kN/m is still reasonable and more accurate than the value of 3.67 kN/m obtained from the tangent stiffness analysis with 200 increments. Both the tangent stiffness and visco-plastic analyses produce ultimate failure loads which approach 2.79 kN/m as the number of increments increases. However, tangent stiffness analyses approach this value from above and therefore over predict, while visco-plastic analyses approach this value from below and so under predict. The CPU times and results for selected analyses are shown in **Table 6.3**. It should be noted that these analyses were completed many years ago, so the absolute time values are not indicative of analyses performed on modern computers. However the relative times are still valid.

The second example considers the mobilisation of the stresses in the soil immediately adjacent to a pile shaft during drained

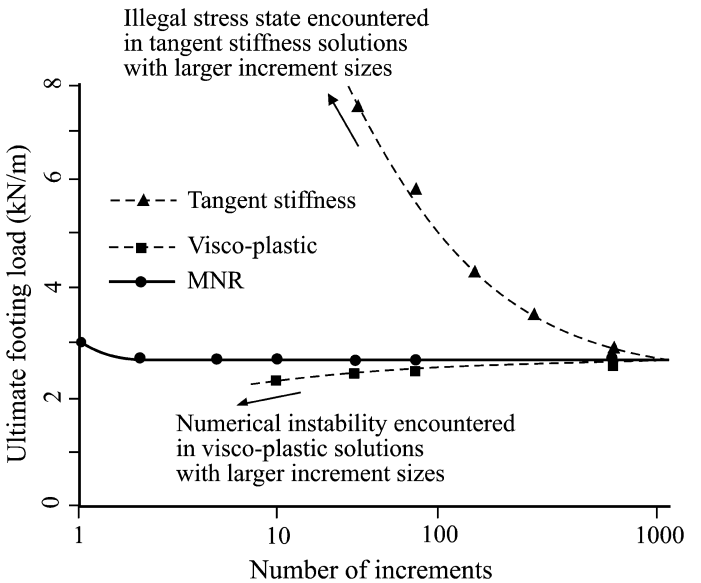


Figure 6.19 Ultimate footing load vs number of increments

Solution type	Number of increments	CPU times	Relative CPU times	Collapse load kN
MNR	10	15 345	1	2.82
Tangent stiffness	500	52 609	3.4	3.01
Tangent stiffness	1 000	111 780	7.3	2.82
Visco-plastic	25	70 957	4.6	2.60
Visco-plastic	500	1 404 136	91.5	2.75

Table 6.3 CPU times and failure loads for the footing analyses

loading. The behaviour of a segment of an incompressible 2 m diameter pile, well away from the influence of the soil surface and pile tip, is examined (see **Figure 6.20**). This boundary value problem has been discussed in detail by Potts and Martins (1982) and the alternative methods of representing the problem in finite element analysis have been explored by Gens and Potts (1984). The soil is assumed to be normally consolidated, with initial stresses $\sigma_v' = \sigma_h' = 200$ kPa. The parameters for the modified Cam Clay model are the same as those used for the footing problem and are discussed in detail in Potts and Zdravkovic (1999). Axisymmetric conditions are applicable to this problem and the finite element mesh is shown in **Figure 6.20**. Loading of the pile has been simulated in the finite element analyses by imposing a series of equally sized increments of vertical displacement to the pile shaft to give a total displacement of 100 mm. The soil was assumed to behave in a drained manner throughout.

Results from analyses using all three solution strategies and with varying numbers of increments are presented in **Figure 6.21** in the form of mobilised shaft resistance, τ, against vertical pile displacement. The MNR analyses, with 20 increments and above, plot as a single curve on this figure and are represented by the upper solid line. MNR analyses

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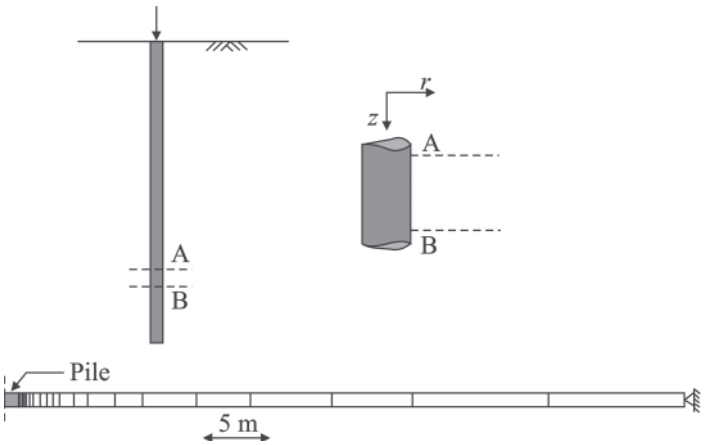


Figure 6.20 Geometry and finite element mesh for the pile problem

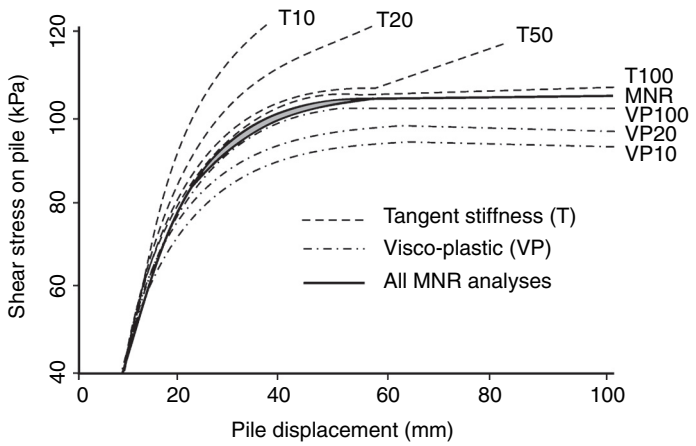


Figure 6.21 Mobilised pile shaft resistance vs pile displacement

with smaller numbers of increments (i.e. with larger applied displacement increments) showed minor differences with the above results between pile displacements of 25 and 55 mm. The MNR analysis with five increments is shown by the lower solid line in the figure. The spread of results between the MNR analyses with five and 20 increments is therefore given by the shaded band. Because this band is small it may be concluded that, as with the footing problem, the MNR results are insensitive to increment size.

Results from some of the tangent stiffness and visco-plastic analyses are also shown on **Figure 6.21**. If a large number of small increments of applied displacement are used, both these solution strategies give results which approach those of the MNR analyses. However, analyses using both these approaches are sensitive to increment size, and analyses with larger increments of applied displacement are inaccurate. The tangent stiffness over predicts and the visco-plastic under predicts the mobilised shaft resistance for any given pile displacement. The visco-plastic analyses predict that the mobilised pile shaft resistance increases to a peak value and then slowly reduces with further pile displacement. This behaviour is not predicted by the MNR analyses which show that once the maximum shaft resistance is mobilised, it remains constant. For the tangent stiffness analyses with less than 100 increments, the mobilised pile shaft resistance continually climbs and no peak value is reached.

Values of the mobilised pile shaft resistance, τ_f, at a pile displacement of 100 mm (the end point on the curves in **Figure 6.21**) are plotted against the number of increments used in **Figure 6.22**. As noted above, the MNR results are insensitive to increment size, with τ_f only increasing from 102.44 to 103.57 kPa as the number of increments increases from 1 to 500. The results from the tangent stiffness and visco-plastic analyses are much more dependent on the size of the applied displacement increment. Over 100 increments for the tangent stiffness and 500 increments for the visco-plastic analyses are needed to give reasonably accurate results. As the tangent stiffness and visco-plastic solutions approach those of the MNR when the applied displacement increment reduces, the 500 increment MNR analysis may be assumed to be 'correct'. The error in any analysis may therefore be expressed as:

$$\text{Error} = \frac{\tau_f - 103.6}{103.6}$$

where τ_f for the 500 increment MNR analysis equals 103.6 kPa. This error is plotted against CPU time in **Figure 6.23** for all analyses. The results clearly show that as well as being the least accurate, the visco-plastic analyses are also the most expensive in terms of CPU time consumed. As the MNR analyses give accurate solutions with small error values, typically less than 0.2%, the results plot along the CPU time axis in this figure. These CPU times compare favourably with those of the tangent stiffness analyses.

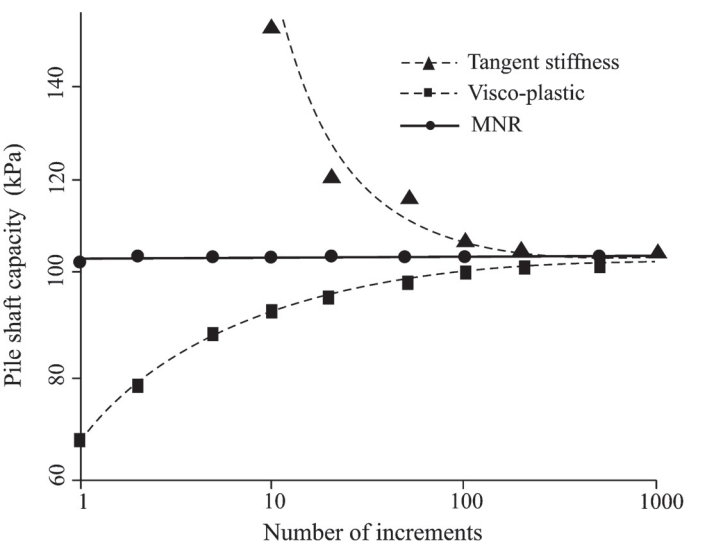


Figure 6.22 Pile capacity vs number of increments

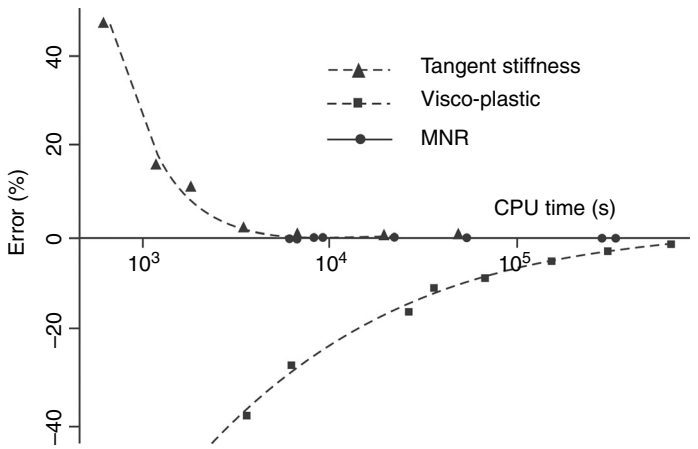


Figure 6.23 Error in pile analysis vs CPU time

The relatively bad performance of the visco-plastic strategy should be noted. This contrasts to its better performance for the footing problem. However, the footing problem involved undrained soil behaviour, whereas for the pile problem the soil is assumed to remain drained. Results from the analysis of a range of boundary value problems presented in Potts and Zdravkovic (1999) also indicate that the relative performance of the visco-plastic strategy is worse for drained than undrained sites. It may be concluded that for the analyses with the modified Cam Clay model, the visco-plastic approach is likely to be better behaved and less sensitive to the size of the solution increment when soil behaviour is undrained than when the soil is drained.

6.8.5 Comments

Results from the tangent stiffness analyses of both boundary value problems are strongly dependent on increment size. The error associated with the tangent stiffness analyses usually results in unconservative predictions of failure loads and displacements in most geotechnical problems. For the footing problem, large over predictions of failure loads are obtained, unless a very large number of increments (>1000) are employed. Inaccurate analyses based on too large an increment size produced ostensibly plausible load–displacement curves. Analytical solutions are not available for most problems requiring a finite element analysis. Therefore it is difficult to judge whether a tangent stiffness analysis is accurate on the basis of its results. Several analyses must be carried out using different increment sizes to establish the likely accuracy of any predictions. This could be a very costly exercise, especially if there was little experience in the problem being analysed and no indication of the optimum increment size.

Results from the visco-plastic analyses are also dependent on increment size. For boundary value problems involving undrained soil behaviour these analyses are more accurate than tangent stiffness analyses with the same increment size. However, if soil behaviour is drained, visco-plastic analyses are only accurate if many small solution increments are used. In general, the visco-plastic analyses use more computer resources than both the tangent stiffness and MNR approaches. For both the footing and pile problems, the visco-plastic analyses under predict failure loads if insufficient increments are used and are therefore conservative in this context.

Close inspection of the results from the visco-plastic and tangent stiffness analyses indicates that a major reason for their poor performance is their failure to satisfy the constitutive laws. This problem is largely eliminated in the MNR approach, where a much tighter constraint on the constitutive conditions is enforced.

The results from the MNR analyses are accurate and essentially independent of increment size. For the boundary value problems considered, the tangent stiffness method requires considerably more CPU time than the MNR method to obtain results of similar accuracy, in fact over seven times more for the foundation problem. Similar comparisons can be found between the MNR and visco-plastic solutions: the tangent stiffness or visco-plastic method with an optimum increment size is likely to require more computer resources than an MNR analysis of the same accuracy. Though it may be possible to obtain tangent stiffness or visco-plastic results using less computer resources than with the MNR approach, this is usually at the expense of the accuracy of the results. Alternatively, for a given amount of computing resources, an MNR analysis produces a more accurate solution than either the tangent stiffness or visco-plastic approaches.

The study has shown that the MNR method appears to be the most efficient strategy for obtaining an accurate solution to problems using critical state type constitutive models for soil behaviour. However, it should be noted that the MNR method is highly sensitive to the stress point algorithm (see section 6.8.3) that it uses, which is code-dependent. The large errors in the results from the tangent stiffness and visco-plastic algorithms in the present study emphasise the importance of checking the sensitivity of the results of any finite element analysis to increment size.

6.9 Modelling of structural members in plane strain analysis

While the main feature of many geotechnical problems might be adequately represented by a plane strain idealisation, other, smaller, components of the same problem might not be. For example, let us consider a typical urban excavation problem as shown in **Figure 6.24**. Here we have an embedded retaining wall, supported by ground anchors and a foundation slab tied down by tension piles. This could represent a cross-section of an excavation for a road. Let us now consider the structural members in turn.

6.9.1 Walls

If the wall is of the concrete diaphragm type, then it is best modelled using solid finite elements with the appropriate geometry and material properties. There is no serious modelling problem here as the wall satisfies the plane strain assumption. However,

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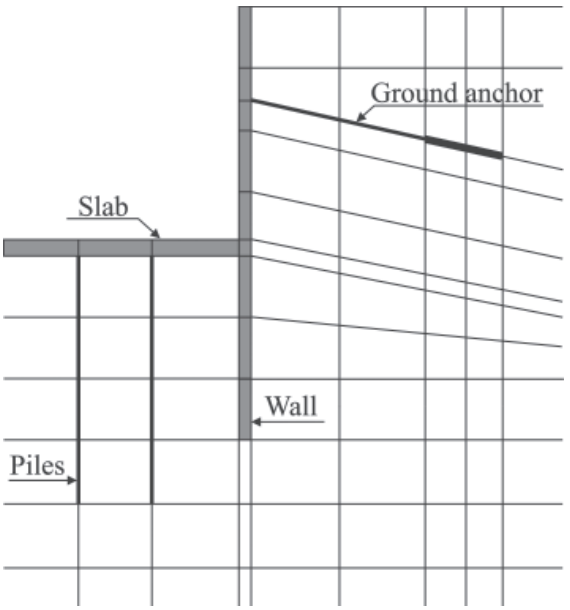


Figure 6.24 Typical urban excavation

if the wall is made from secant or contiguous concrete piles, steel sheet piles, or some combination of steel columns and sheeting, then the properties and geometry of the wall will vary in the out-of-plane direction and therefore will not satisfy the requirements of plane strain. Modelling of these components in a plane strain analysis will therefore involve some approximations. It is usually possible to estimate the average axial (EA) and bending stiffness (EI) per meter length of wall. If beam elements are used to model the wall, these parameters can be input directly as the material properties. However, if solid elements are to be used, then the EA and EI must be converted into an equivalent thickness t of the solid elements and an equivalent Young's modulus, E_{eq}. This is done by solving the following two simultaneous equations:

$$\text{EA:} \quad t \cdot E_{eq} = E \cdot A$$

$$\text{EI:} \quad \frac{E_{eq-t^3}}{12} = E \cdot I$$

where E is the Young's modulus of the wall, A is the cross-sectional area, and I the moment of inertia per meter length. It may also be necessary to calculate some form of average strength, but this will depend on the constitutive model employed to represent the wall. Clearly the above procedure only treats the wall in an approximate manner and it will not be possible to accurately estimate the details of the stress distribution within the separate components of the wall.

6.9.2 Piles

Modelling the piles below the base slab involves additional assumptions as the piles are not continuous in the out-of-plane direction, but are separated by relatively large expanses of soil. While it is again possible to estimate EA and EIs per unit length in the out-of-plane direction, and to calculate equivalent parameters as was shown for the wall above, modelling the piles with solid or beam elements implies that the soil is not able to freely move between the piles. This is because the piles are essentially modelled as a wall in the out-of-plane direction, see **Figure 6.25**. As a consequence, the lateral movements of the soil below the excavation will be restricted. It may therefore be more realistic to neglect the EI of the piles so that they provide no resistance to lateral soil movement. A spring or a series of membrane elements can then be used to represent the piles.

If a linear spring is used to represent the pile only, elastic behaviour of the pile can be represented. In addition, if the spring is connected between a node on the concrete slab and a node in the soil (i.e. nodes A and B in **Figure 6.26**), only forces at these two nodes will be accounted for. In other words, the pile and soil are not connected between A and B. It will then

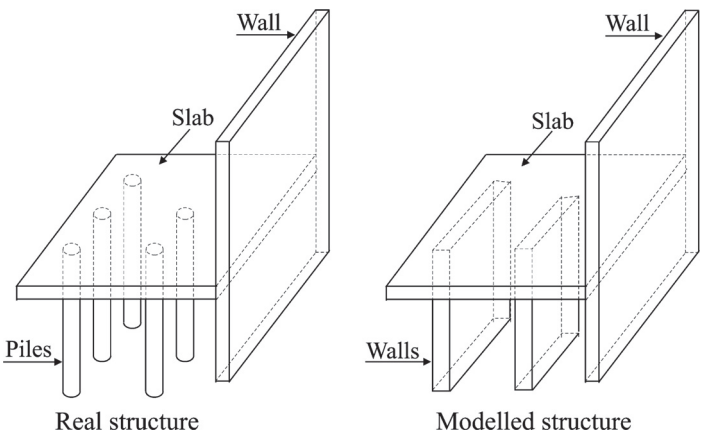


Figure 6.25 Comparison between real and simulated conditions for piles

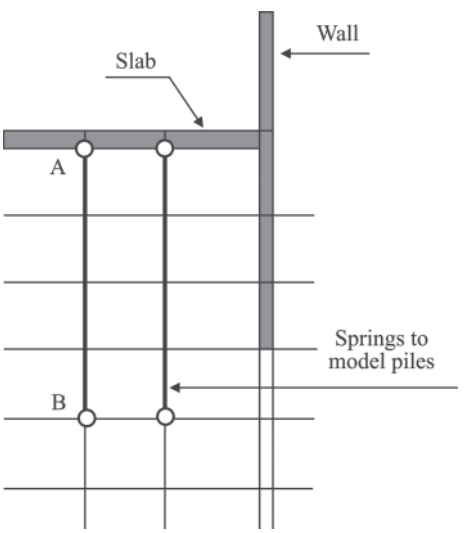


Figure 6.26 Springs used to model piles

not be possible to account for shaft friction nor to set a limit to the end bearing resistance of the pile (i.e. at node B).

If a series of membrane (or beam) elements are used to model the pile, then the pile will be connected to the soil along its shaft and consequently mobilisation of shaft resistance will be simulated to some limited extent (see **Figure 6.27**). However, the zero thickness of the elements in the plane of analysis implies that base resistance will be mobilised through a single node; it is therefore difficult to impose a limit to the end bearing resistance.

6.9.3 Ground anchors

The ground anchors located behind the wall comprise of two parts: the *fixed* and the *free* anchor lengths, see **Figure 6.28**. As with the piles, equivalent EA and EIs per unit length in the out-of-plane direction can be calculated for both components of a row of anchors. Account must be taken of the spacing of the anchors in the out-of-plane direction when calculating these equivalent values. Again, it is probably best to ignore the EI and model the two anchor components with either springs or membrane elements, or a combination of both (see **Figure 6.29**). The situation shown in **Figure 6.29(c)**, where a spring is used to model the free anchor length and a series of membrane elements the fixed length, is probably the more realistic approach. It is usual to neglect any shear stresses mobilised between the soil and the free anchor length. Consequently, the springs are connected to the wall at one end (point A in **Figure 6.29(c)** and the soil (and fixed anchor length) at the other (point C in the same figure). As with the piles, modelling the anchors in this way is rather crude as it is not possible to accurately account for the limiting end and shaft capacity of the fixed anchor length.

An alternative way of modelling the fixed anchor length is shown in **Figure 6.30** and involves the use of solid and interface elements. It is now possible to simulate the mobilisation of both shaft and end bearing resistance and to apply limits to these quantities. However, it is necessary to assign equivalent properties to the solid and interface elements to take account

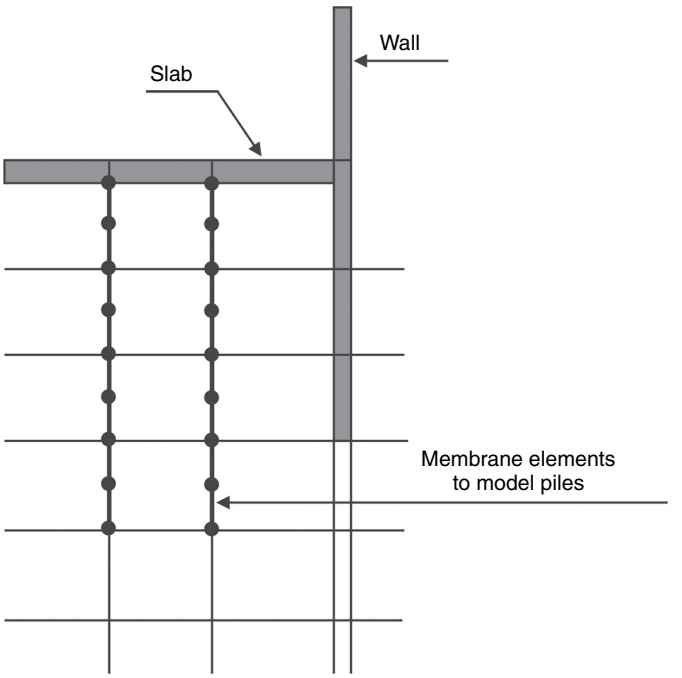


Figure 6.27 Membrane elements used to model piles

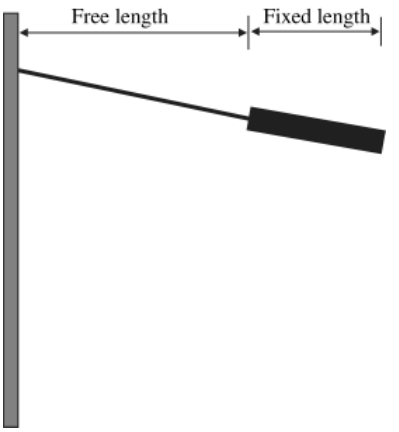


Figure 6.28 Ground anchor

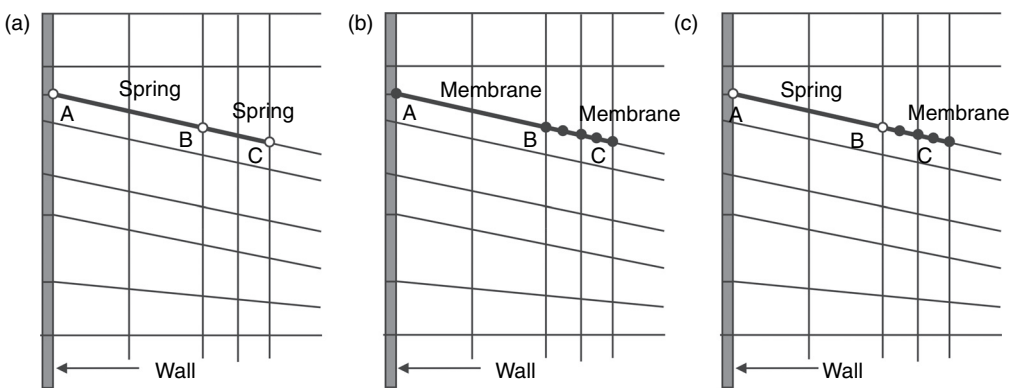


Figure 6.29 Modelling ground anchors using (a) springs, (b) membrane elements, and (c) a combination of spring and membrane elements

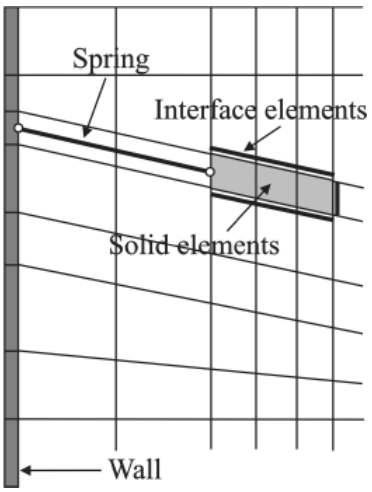


Figure 6.30 Modelling a ground anchor with solid elements

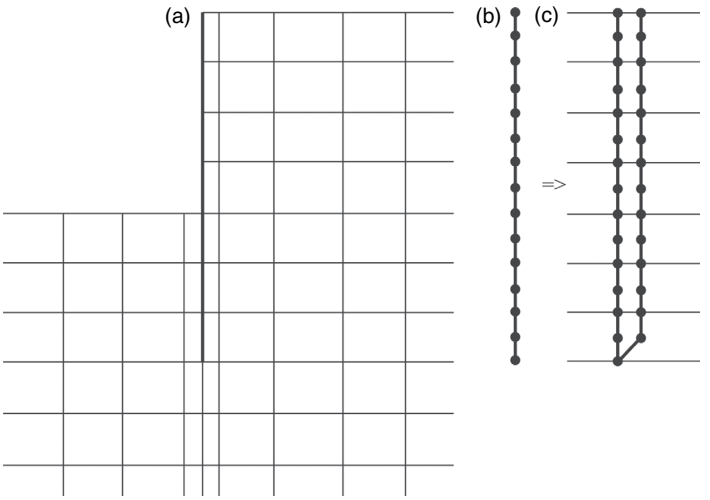


Figure 6.31 Dealing with impermeable line elements in consolidation problems (use of interface elements)

of the spacing of the anchors in the out-of-plane direction. This is often not straightforward. By assigning an equivalent Young's modulus to the solid elements, the fixed anchor length will have EI. This will limit soil movements perpendicular to the anchor. In reality, the soil will be able to deform between the anchors.

The above discussion indicates that while it is possible to approximate the wall behaviour in a realistic manner, much greater approximations are involved with the modelling of both the piles and ground anchors. This arises because these structural elements are not continuous in the out-of-plane direction.

6.9.4 Structural members in coupled analyses

A further pitfall can occur when using membrane or beam elements to model structural members in coupled consolidation analyses. The problem is best explained by considering the example of the retaining wall shown in **Figure 6.31**. The wall is modelled using beam elements and therefore shares a common set of nodes with both the elements in front of, and behind it, see **Figure 6.31(b)**. As the solid elements on either side of the wall have common nodes at the wall, the wall will be implicitly assumed to be permeable in a consolidation analysis. Water will therefore flow freely through the wall which may not be the desired outcome. If the wall is to be assumed impermeable, this can be achieved by placing interface elements along one side of the wall as shown in **Figure 6.31(c)**. If these elements are non-consolidating, they will provide an impermeable break between the solid elements on either side of the wall.

6.9.5 Structural connections

Potential problems can arise when modelling connections between different structural components, e.g. the connection between props and a retaining wall. Such connections can be modelled as *simple*, *pin-joined* or *full moment* connections. These conditions and the possible alternative ways that can be used to model them in plane strain analyses are shown in **Figures 6.32** to **6.34**. It is often all too easy to use the options given in **Figure 6.34** when in reality the connection is pin-jointed.

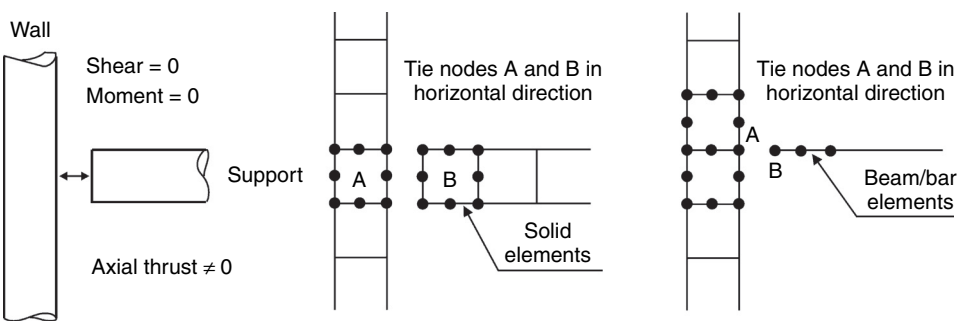


Figure 6.32 Simple connection between wall and beam

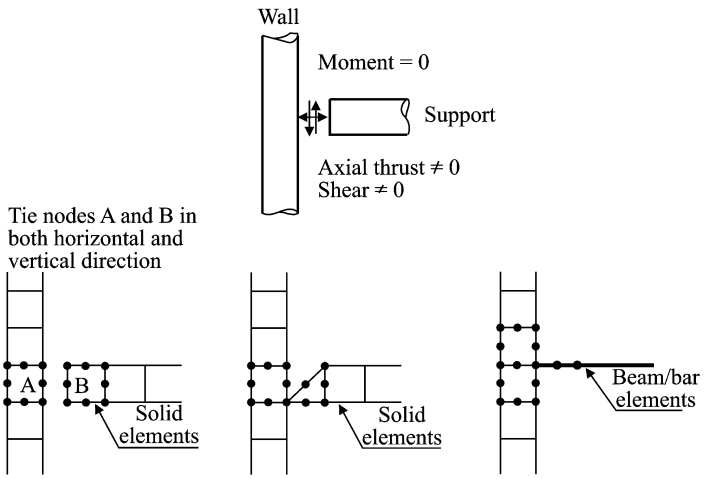


Figure 6.33 Pin-jointed connection between wall and prop

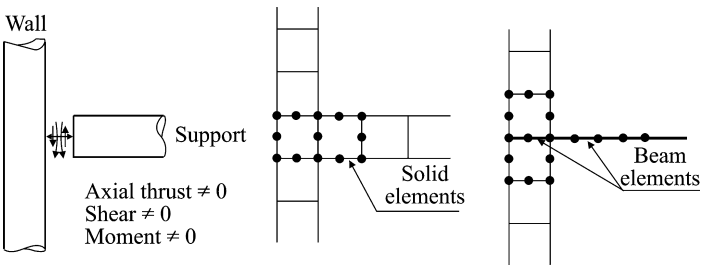


Figure 6.34 Full connection between wall and prop

6.9.6 Segmental tunnel linings

Many tunnel linings are constructed from segments. Between eight and 12 segments are usually required to form a complete circular ring, see **Figure 6.35(a)**. Sometimes the segments are bolted to each other and sometimes not. In either event, there is little bending resistance between the segments. It is often convenient to model the lining using beam or solid elements. However, if these elements are placed as a sequential ring (illustrated in **Figure 6.35(b)**), a full moment connection between the segments will be implied. If this is not what is required, then an alternative approach must be used. For example, special beam elements which represent the interface between the segments can be used. Each segment is modelled as a group of beam elements and these are then joined together by these small special beam elements; see **Figure 6.35(c)**.

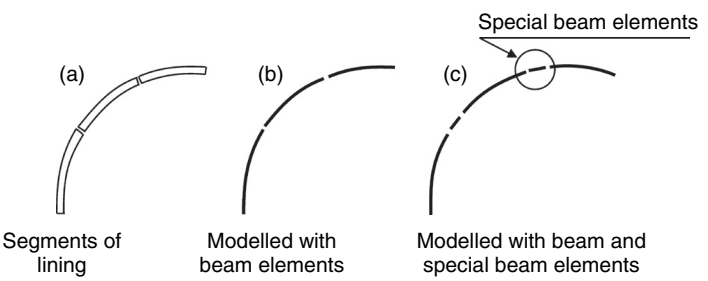


Figure 6.35 Modelling tunnel lining

6.10 Some pitfalls with the Mohr–Coulomb model

The Mohr–Coulomb constitutive model is the simplest representation of soil behaviour and forms part of most geotechnical software packages. It is also the most widely used constitutive model in geotechnical engineering practice. This is a linear elastic, perfectly plastic model, requiring only a few input parameters that can be obtained from standard laboratory testing, namely Young's modulus E and Poisson's ratio μ – to describe the elastic part of the model, cohesion c' and angle of shearing resistance ϕ' – to describe the plastic (failure) part of the model. If no other input parameter is required, this implies associated plasticity for the model and negative (dilative) plastic volumetric strains. Problems with such a formulation of the model are two-fold: (i) soil dilation is usually smaller than that implied by associated plasticity (i.e. angle of dilation $\nu = \phi'$); and (ii) once the soil starts to dilate, it will dilate forever, without reaching a limit load for volumetrically constrained problems.

6.10.1 Drained loading

An example of the performance of the Mohr–Coulomb model in drained conditions is given in **Figure 6.36**, which shows load–displacement curves for a vertically loaded pile, 1.0 m diameter and 20 m long, in a drained soil with $c' = 0$ and $\phi' = 25°$ (Potts and Zdravković, 2001). If the angle of dilation, ν, is equal to ϕ' (i.e. associated plasticity), the pile never reaches a limit load, no matter how far it is pushed into the ground. Even if the model has

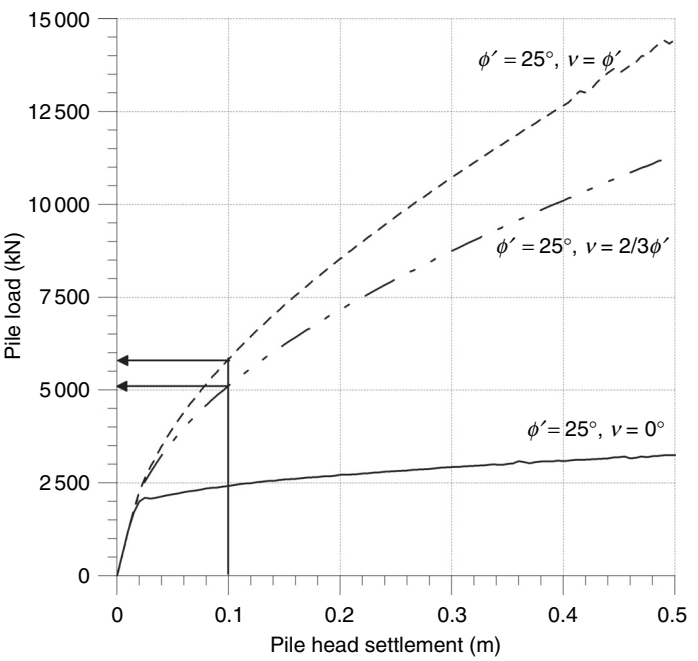


Figure 6.36 Load–displacement curves for vertically loaded pile

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the flexibility to input a value of ν smaller than ϕ', for any value of ν greater than zero, the model still does not predict a limit load. The magnitude of ν makes a difference only in the magnitude of the dilative volumetric plastic strains, which reduce with a reduction of ν and hence reduce the magnitude of the pile load for a particular displacement of the pile head. Consequently, a practical approach in determining a limit load from such analysis has been to arbitrarily adopt the load value that corresponds to the displacement equal to 10% of the pile diameter (0.1 m in this case). This can result in a significant overestimate in the load capacity of the pile in the case of associated plasticity, as soil dilation is generally over predicted. The pile can only reach a limit load if $\nu = 0$. This is a conservative prediction as most soils dilate to some extent, but it is at least a theoretically correct value obtained without any arbitrary decisions from the user.

6.10.2 Undrained loading

As noted above, the Mohr–Coulomb model can be used with a dilation angle ν ranging from 0 to ϕ'. This parameter controls the magnitude of the plastic dilation (plastic volume expansion) and remains constant once the soil is on the yield surface. This implies that the soil will continue to dilate indefinitely if shearing continues. Clearly such behaviour is not realistic as most soils will eventually reach a critical state condition, after which they will deform at constant volume if sheared any further. While such unrealistic behaviour does not have a great influence on boundary value problems which are unrestrained (i.e. the drained surface footing problem), it can have a major effect on problems which are constrained, as noted above, for the drained pile problem, due to the restrictions on volume change imposed by the boundary conditions. In particular, unexpected results can be obtained in undrained analysis in which there is a severe constraint imposed by the zero total volume change restriction associated with undrained soil behaviour. To illustrate this, two examples will now be presented.

The first example considers ideal (no end effects) undrained triaxial compression tests ($\Delta\sigma_v > 0$, $\Delta\sigma_h = 0$) on a linear elastic Mohr–Coulomb plastic soil with parameters $E' = 10\,000$ kPa, $\mu = 0.3$, $c' = 0$, $\varphi' = 24^o$. As there are no end effects, a single finite element is used to model the triaxial test with the appropriate boundary conditions. The samples were assumed to be initially isotopically consolidated with $p' = 200$ kPa and zero pore water pressure. A series of finite element runs were then made, each with a different angle of dilation, ν, in which the samples were sheared undrained. Undrained conditions were enforced by setting the bulk modulus of water to be 1000 times larger than the effective elastic bulk modulus of the soil skeleton, K', see Potts and Zdravkovic (1999). The results are plotted in **Figure 6.37**: (a) in the form of deviatoric stress (J) vs mean effective stress (p'), and (b) in the form of deviatoric stress (J) vs axial strain (ε_z). It can be seen that in terms of J vs p', all analyses follow the same stress path. However, the rate at which the stress state moves up the Mohr–Coulomb failure line differs for each analysis. This can be seen in **Figure 6.37(b)**. The analysis with zero plastic dilation, $\nu = 0$, remains at a constant J and p' when it reaches the failure line. However, all other analyses move up the failure line; those with the larger dilation moving up more rapidly. They continue to move up this failure line indefinitely with continued shearing. Consequently, the only analysis that indicates failure (i.e. a limiting value of J) is the analysis performed with zero plastic dilation.

The second example considers the undrained loading of a smooth rigid strip footing. The soil was assumed to have the same parameters as those used for the triaxial tests above. The initial stresses in the soil were calculated on the basis of a saturated bulk unit weight of 20 kN/m^3, a ground water table at the soil surface and a coefficient of earth pressure at rest $K_o = 1 - \sin \phi'$. The footing was loaded by applying increments of vertical displacement and undrained conditions were again enforced by setting the bulk modulus of the pore water to be 1000K'. The results

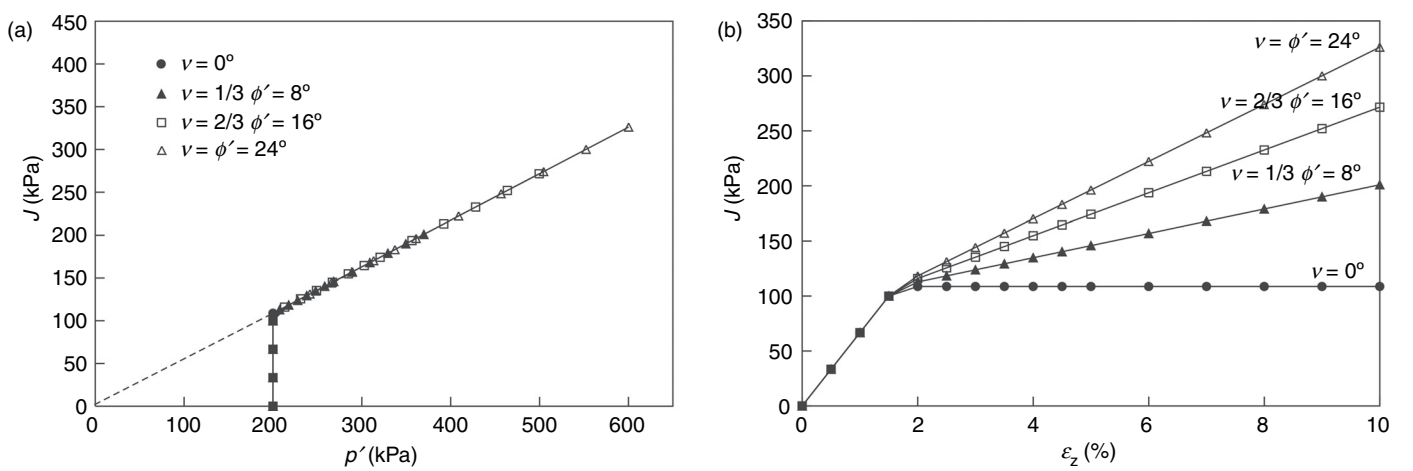


Figure 6.37 Prediction of (a) stress paths and (b) stress–strain curves in undrained triaxial compression using Mohr–Coulomb model with different angles of dilation

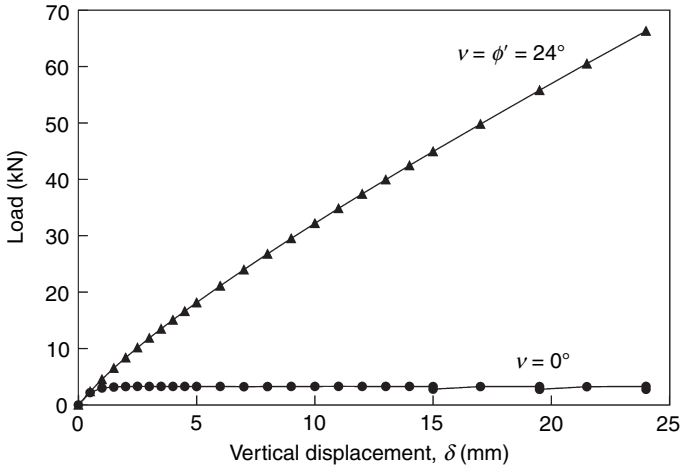


Figure 6.38 Load–displacement curves for a strip footing, using the Mohr–Coulomb model with different angles of dilation

of two analyses, one with $\nu = 0^{o}$ and the other with $\nu = \phi'$, are shown in **Figure 6.38**. The difference is quite staggering: while the analysis with $\nu = 0^{o}$ reaches a limit load, the analysis with $\nu = \varphi'$ shows a continuing increase in load with displacement. As with the triaxial tests, a limit load is only obtained if $\nu = 0^{o}$.

It can be concluded from these two examples that a limit load will only be obtained if $\nu = 0^{o}$. Consequently, great care must be exercised when using the Mohr–Coulomb model in undrained analysis. It could be argued that the model should not be used with $\nu > 0$ for such analysis. However, reality is not that simple and often a finite element analysis involves both an undrained and a drained phase (i.e. undrained excavation followed by drained dissipation). It may be then necessary to adjust the value of ν between the two phases of the analysis. Alternatively, a more complex constitutive model which better represents soil behaviour may have to be employed.

6.11 Summary

This chapter identifies the main design requirements for a geotechnical structure, for which calculations have to be performed in order to confirm the stability and safety of the design. These requirements are:

- local stability;
- overall stability;
- structural forces;
- ground movements.

Several methods of analysis are available for the above geotechnical calculations and they are classified into three main groups:

- closed form solutions;
- classical analysis (limit equilibrium, stress fields and limit analysis);
- numerical analysis (beam-spring approach and full numerical analysis).

In terms of satisfying the main theoretical requirements, it is important to note that:

- Closed form solutions are possible to obtain in some cases only if the soil is assumed to be linear elastic. However, this is a significant simplification of real soil behaviour and may result in the wrong predictions of ground movements. Additionally, these solutions cannot be used to assess stability of geotechnical structures.
- None of the classical methods of analysis satisfy all of the theoretical requirements; hence in general they do not produce an exact solution. An exception is when calculations using both the 'safe' and 'unsafe' theorems of limit analysis result in the same answer. However, such cases are rare.
- The beam-spring approach can analyse only one structural element with the soil represented by a number of springs. The main issue in this approach is to assign the appropriate spring stiffness.
- Full numerical analysis, in terms of finite elements or finite differences, is the ultimate analysis method which satisfies all theoretical requirements, discretises both the soil and the structure in the same way and can apply a constitutive behaviour which models the soil in a realistic manner.

However, it is important to recognise that full numerical analysis should not be performed blindly, without understanding the numerical methods (algorithms) and software used in the analysis. Some of the main causes of error in finite element analysis discussed in this chapter are:

- element discretisation (i.e. the type and size of finite element);
- choice of the nonlinear solver (i.e. solution of the system of nonlinear finite element equations);
- modelling of structural elements within the soil continuum;
- choice of appropriate constitutive models.

The number of potential errors increases as the analysis becomes more advanced – involving, for example, coupled problems (i.e. both mechanical behaviour and fluid flow, where the choice of correct permeability models and hydraulic boundary conditions becomes important), and soil dynamics and earthquake engineering problems (where appropriate constitutive models, boundary conditions and time integration schemes become important).

If all the above aspects of numerical analysis are applied properly, then it has an enormous potential to explain the engineering behaviour of geotechnical structures and produce an optimal and safe design.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

All figures and tables in this chapter have been reproduced from Potts and Zdravkovic (1999) and Potts and Zdravkovic (2001).

Chapter 7

Geotechnical risks and their context for the whole project

Tim Chapman Arup, London, UK

This chapter provides an understanding of how project developers consider construction projects and their huge vulnerability to project delays, and when it can be profitable to proceed with a development. By example, it shows the proportion of geotechnical-related costs for a typical London office project and compares those costs to the tiny investments in geotechnical risk mitigation by desk study and ground investigation. It explains that geotechnical risks can be 'wildly random' instead of 'mildly random' and so the consequences of a problem can be hugely disproportionate to the initial cost. It compares typical frequencies of ground-related problems and finds that even the average cost of mildly random problems have consequences that far outweigh the very modest cost of a good ground investigation. It concludes that management of geotechnical risk by contingency is an approach fraught with peril and that management by mitigation is much more likely to produce the desired outcome.

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7.1 Introduction

The aim of this chapter is to provide data to enable geotechnical engineers to communicate simply and clearly with their clients, and to persuade them of the merits of timely geotechnical investigations and other interventions.

It is well known that:

- construction problems occur too frequently;
- the consequential delays and their associated costs are huge;
- the problems could often have been avoided.

The aim of this chapter is to quantify (to some extent) the first and second two points, and to offer some potential solutions to the third.

This chapter considers financial risks only and not safety ones – it examines the overall design and construction process; safety risks are usually triggered by errors in discrete operations, such as a missing toe-board. Projects where the overall design and construction process is skimped may share characteristics with those that suffer the sorts of organisational breaches that lead to accidents. Delays due to injuries and scrutiny by the Health and Safety authorities can have a devastating effect on programme and out-turn costs, not to mention the human impact on all involved. Safety in geotechnical engineering is discussed in more detail in Chapter 8 *Health and safety in geotechnical engineering*.

7.2 Motivation of developers

> Heathrow's new terminal is on time and on budget. How odd.
> Headline in *The Economist*, 20 August 2005

To understand the critical issues that affect developments, it is worth examining the motives of developers, who fall into three categories:

- Commercial developers – who create developments essentially to make a profit; either speculatively where the eventual occupier is not yet known, or pre-let to a bespoke tenant.
- Self developers – where the intention of the development is to facilitate other operations such as building an office or a manufacturing function for a large company. Sometimes assistance from a commercial developer is sought on a consultancy basis.
- State developers – where the state creates the national infrastructure to allow the rest of the economy to function (e.g. roads, railways, schools and hospitals).

In each case, a rational cost benefit analysis can be carried out to decide whether a particular development should proceed.

To assist in understanding the sensitivity of commercial drivers on a development, it is helpful to study a hypothetical example. The example created by Chapman and Marcetteau (2004) has been altered and extended to show the main issues that influence the commercial success of a development.

The building under consideration is:

- A commercial office building in the City of London.
- Currently occupied by a 30-year-old building with a gross area of 16 000 m^2 (172 000 ft^2).
- Occupying a site which could be replaced by a new building with a gross area of about 20 000 m^2 (215 000 ft^2); with 85% efficiency the net lettable area would be 17 000 m^2 (183 000 ft^2). The site value, based on net lettable area, is estimated to be £376/ft^2 (a typical range would be £250–£500/ft^2) – so the land on which it is to be built would cost some £68.8 million.

The developer first needs to decide whether it is worth maintaining the existing asset, or proceeding with a new development. The type of development contemplated is shown in **Figure 7.1**, and its details are summarised in Appendix A of this chapter. The relative costs for the development are summarised in **Figure 7.2**. The basis for his choice to proceed with the development is given in **Table 7.1**. The new building will be attractive to a broader range of occupiers, including those able and willing to pay higher rents.

From the calculations in **Table 7.1**, proceeding with the development makes commercial sense, based on the increase in demand (and hence rent) for the newer and bigger building. The table omits the prospect of capital growth, which has also made property development a popular investment in good times. It also omits any requirement to provide affordable housing as a proportion of the extra space created, which can be a tax that inhibits new developments. **Table 7.2** shows a comparison of typical investment returns in 2001, given by RIBA (2003), showing why investment in property was favoured at that time.

By 2005 in a stronger property market, most income yields had remained the same but capital growth for equities had accelerated to 8% giving a total return of 11%, while capital growth for commercial property peaked at 11% giving a total return of 18%. Both had declined by early 2008, making the choice to invest in commercial property considerably more marginal. *The Economist* (Anon, 2008) reported that office rents in the city of London had risen by 22% from September 2006 to September 2007, and predicted that they may fall by 5% over the following year. At the time, it was predicted to lead to a fall in property value of up to 30% over the next three years, potentially the biggest collapse in commercial property prices since the Second World War. Of the speculative offices under construction now in the City of London, not all are forecast to find tenants in the short term. At the time of writing (2009), banks had become more cautious about lending to this sector and the drop in collateral prices is not encouraging further lending to developers for this purpose. By late 2009, with returns on all assets at long-term lows and liquidity re-entering the financial system, some signs of banking investment interest are returning. This has persisted with the start of a full recovery in London by mid 2011 but the property market elsewhere in the UK still in recession.



Figure 7.1 Typical new 6-storey London office development

A developer can make a simple profit by using his own funds for the whole development, as shown in **Table 7.3** (RIBA, 2003).

Developers can leverage their success by use of a bank loan, but it can also magnify failure. This is illustrated in a simplified form in **Table 7.4**.

Thus the developer can make a much greater profit by using a loan, but can also make a much greater loss if the project costs more than expected. Developers therefore crave certainty even above cheapness. Once a site has been bought (potentially with an outlay of over half the development cost) and demolished (so no further income will be received until the new building is occupied), the developer is very vulnerable. He is particularly vulnerable to any:

- Increase in costs – distorting the investment basis.
- Delay in completion – hence delaying the start of the income stream to pay for it. This is particularly painful when the development funds are borrowed and interest has to be found to be paid over the interim period until rental cash starts coming in.
- Increase in interest rates, particularly if unexpected.
- Reduction in standard of the completed building – making it less attractive to potential clients and hence lowering its rental value.

This is illustrated in **Figure 7.3**; this shows how rental rates need to develop rapidly to catch up with the mounting bank interest changes.

These calculations have been carried out for a City of London development, where demand generally tends to be strong and potential rental rises may offset extra costs due to any problems. In other locations, the calculations underpinning a new development are often more marginal and the prospect of a favourable rate rise is more remote.

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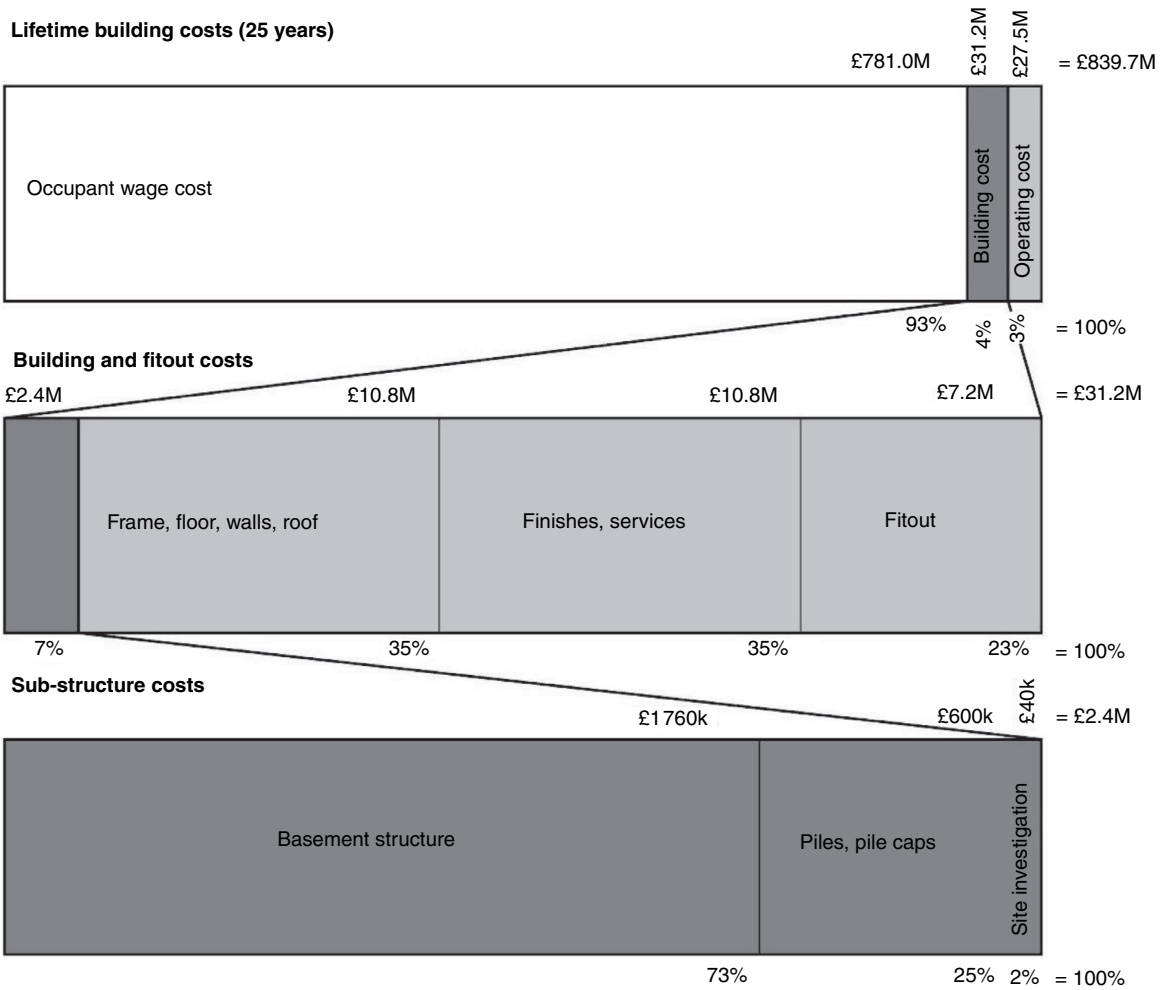


Figure 7.2 Breakdown of costs for a typical building

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	No new development	New building
Annual rent per square foot (£)	39	55
Floor area	172 000 ft² (16 000m²)	215 000 ft² (20 000 m²)
Annual rent (£)	6.7 million	11.8 million
Payback period considered (years)	25	25 – 3 = 22 (3-year construction/rent-free period)
Total rent (£)	168.0 million	259.6 million
Less construction cost (£)	–	31.2 million
Net revenue (£)	168.0 million	228.4 million (i.e. 60.4 million extra)
Notes	Probably longer fallow periods between tenants; possibly shorter rental periods	Probably reduced maintenance and operating costs

Table 7.1 Comparison of value from new development to refurbishment of the old building

	Building society	Gilts (10 year bond)	Equities (FTSE 500)	Commercial property
Income	4%	5%	3%	7%
Capital	0%	0%	6%	3%
Total	4%	5%	9%	10%

Table 7.2 Typical investment returns in 2001

Data taken from RIBA (2003)

7.3 Government guidance on 'optimism bias'

> 229 If a builder builds a house for someone, and does not construct it properly, and the house which he built falls in and kills its owner, then that builder shall be put to death.
>
> Hammurabi's Code of Laws 1700BC, Mesopotamia

The UK Government has studied the tendency of project costs and works durations to increase and has come up with advice in its Green Book (HM Treasury, 2003a) to redress the systematic optimism (which it terms 'optimism bias') that has historically afflicted the appraisal process for major projects.

Costs	£ million
1. Land, including fees and demolition	68.8
2. Design and construction costs	31.2
3. Leasing – at 20% of rent	2.0
4. Interest and financing	3.9
5. Void – 1 year rent free	11.8
Total cost	**117.7**
Value	
6. 215 000 ft² at £55/ft²	11.8/annum
7. Investment yield 7% (income from commercial property)	165.2
Dividing (7) by (6) gives a multiplier of 14	
Surplus	
8. Value less total cost (165.2–117.7)	47.5
Surplus as % of cost (47.5×100/117.7)	40%

Table 7.3 Overall consideration of costs and value for a new development
Data taken from RIBA (2003)

Costs (as % of cost of a successful project)	Successful project		A failure	
	Without a loan	With a loan	Without a loan	With a loan
Total cost	100	100	160	160
Loan		–70		–70
Developer's equity outlaid	100	30	160	90
Sale proceeds	140	140	140	140
Loan repayment		–70		–70
Outcome	140	70	140	70
Surplus at completion	40	40	–20	–20
Return on developer's equity (surplus/outlay)	40%	133%	–13%	–22%

Table 7.4 Illustration of the effect of a loan on project success or failure

It suggests that the two main causes for this bias in capital cost estimates are:

- poor definition of the scope and objectives of projects in the business case, due to poor identification of stakeholder requirements; and
- poor management of projects during implementation, so that schedules are not adhered to and risks are not mitigated.

In *Supplementary Green Book Guidance – Optimism Bias* (HM Treasury, 2003b) values are suggested for the extent of optimism bias typically found and are recommended for use unless robust evidence is available to support other values. The ranges are given in **Table 7.5**. These are based on various

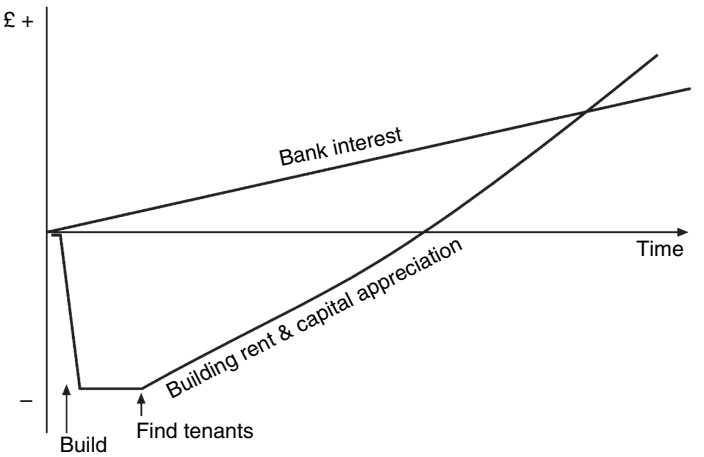


Figure 7.3 Comparison of building investment vs bank interest
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Project type	Optimism bias (%)*			
	Works duration		Capital expenditure	
	Upper	Lower	Upper	Lower
Standard buildings	4	1	24	2
Non-standard buildings	39	2	51	4
Standard civil engineering	20	1	44	3
Non-standard civil engineering	25	3	66	6
Equipment/development	54	10	200	10

* Note that these are indicative starting values recommended and not highest or lowest possible values.

Table 7.5 Range of expected values for optimism bias for different sorts of projects
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contributory factors, listed in **Table 7.6**. Unfortunately, these contributory factors are not broken down into the project phase in which the problems occurred, so the proportion of problems originating in the ground is not discernible. **Table 7.7** shows an extract from the Supplementary Guidance illustrating the sorts of steps taken to mitigate risk for an example project.

From the guidance, it can be deduced that:

- Non-standard projects carry more risk, and non-standard building projects carry more risk than non-standard civil engineering projects. The reason for this is not obvious, but may be due to building projects involving a greater mix of different skills or perhaps because civil engineering projects procured by the state have more rigorous stage reviews and design checks (such as Category 3 checking).
- Building and civil engineering projects seem to be less risky than government projects to procure equipment, which includes software projects.
- Building projects' greatest failings have, as prime causes, inadequacy of the business case by the client and poor procurement, leading to disputes and claims (presumably due to poor specification).

		Non-standard buildings		Standard buildings		Non-standard civil engineering		Standard civil engineering	
Upper bound optimism bias (%) *		**39**	**51**	**4**	**24**	**25**	**66**	**20**	**44**
		Work duration	**Capital expenditure**	**Works duration**	**Capital expenditure**	**Works duration**	**Capital expenditure**	**Works duration**	**Capital expenditure**
Contributory factors to upper bound optimism bias (%) **		**Non-standard buildings**		**Standard buildings**		**Non-standard civil engineering**		**Standard civil engineering**	
Procurement	Complexity of contract structure	3	1	1		4			
	Late contractor involvement in design	6	2	3	2	<1			3
	Poor contractor capabilities	5	5	4	9	2		16	
	Government guidelines								
	Dispute and claims occurred	5	11	4	29	16			21
	Information management								
	Other (specify)					1	2		
Project Specific	Design complexity	2	3	3	1	5	8		
	Degree of innovation	8	9	1	4	13	9		
	Environmental impact						5	46	22
	Other (specify)	5	5			3			18
Client specific	Inadequacy of the business case	22	23	31	34	3	35	8	10
	Large number of stakeholders			6					
	Funding availability	3		8			5	6	
	Project management team	5	2		1		2		
	Poor project intelligence	5	6	6	2	3	9	14	7
	Other (specify)	1	2		<1				
Environment	Public relations			8	2				9
	Site characteristics	3	1	5	2		5	10	3
	Permits / consents / approvals	3	<1	9					
	Other (specify)	1	3						
External influences	Political	13				19			
	Economic		13		11	24	3		7
	Legislation / regulations	6	7	9	3		8		
	Technology	4	5			6	8		
	Other (specify)		2			<1	1		

* Indicative starting values – optimism bias profile for a project will change during its life cycle.
** Contributions from each area as a % of the recorded optimism bias. Rounding errors may result in the sum of values not adding up to 100%.

Table 7.6 Optimism bias upper bound guidance
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Contributory factor	Project cost % contribution to optimism bias	Works duration % contribution to optimism bias	Example cost of risk management
Poor contractor capabilities	5	5	£0
Design complexity	3	2	£140 000
Inadequacy of the business case	23	22	£700 000
Poor project intelligence	6	5	£10 000
Site characteristics, e.g. ground investigation *	1	3	£40 000

* In the example, it is suggested that a Trust has owned a site for at least 20 years with a comprehensive site investigation carried out within the last 5 years, therefore only desk study and limited site investigation are needed.

Table 7.7 Main contributory factors to optimism bias for project cost and works duration for an example project given by HM Treasury
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- Civil engineering projects suffer from environmental impact (presumably mitigation too late in the process), poor contractor capabilities and poor procurement leading to disputes and claims.

It is worth noting that better geotechnical design is an excellent way to mitigate the uncertainties that the allowance for optimism bias is meant to counter. By doing so, the allowance for optimism bias can be reduced – making the project more affordable. This service is of significant advantage to the project promoter.

7.4 Typical frequency and cost of ground-related problems

> When anyone asks me how I can best describe my experience in nearly forty years at sea, I merely say, uneventful. Of course there have been winter gales and storms and fog and the like but, in all my experience, I have never been in an accident … of any sort worth speaking about. I have seen but one vessel in distress in all my years at sea. I never saw a wreck and have never been wrecked nor was I ever in any predicament that threatened to end in disaster of any sort.
>
> Captain Edward John Smith, 1907
> (5 years before he became captain of the RMS Titanic for her maiden voyage.)

A review of 5 000 industrial buildings by the National Economic Development Office (NEDO, 1983) found 50% of the projects overran by at least a month. Study of a representative group of these projects showed that 37% of the overruns were due to ground problems. Of 8 000 commercial buildings, a third overran by more than a month and a further third by up to a month (NEDO, 1988). Half of a representative group of these had suffered delays due to unforeseen ground conditions. The National Audit Office (2001) reported little improvement by 2001: it cites an Office of Government Commerce study which found that 70% of a range of public projects were delivered late and 73% were over the tender price. Contemporary reports from the author's experience of legal disputes and the construction press suggest that incidences of unforeseen ground conditions still occur all too frequently. The UK airport operator BAA, quoted in an article in The Economist in 2005 (Anon, 2005), estimated the industry average for significant delays at 40% (with their own record at just under 20% due to their use of partnering and other ways of aligning their objectives with those of their project collaborators). These data are presented graphically in **Figure 7.4** – roughly one third of the projects studied were delayed by up to a month, and a further third suffered more serious delays. Where data on the cause are available, about a third to a half of delays were blamed on troublesome ground conditions. Thus significant delays due to ground conditions probably occur some 17–20% of the time. It is notable from **Figure 7.4** that the incidence of delays does not seem to have improved with time.

Van Staveren and Chapman (2007) examined figures on the financial impact of project failures, which were huge. In the Netherlands alone, failure figures are assessed between 5% and 13% of the yearly expenditure of the Dutch construction industry (for all failures, not just in the ground). As Dutch spending on construction totals some 70 billion euro per year, between 3.5 and 9 billion euro are spent to respond to failures (van Staveren, 2006). Chapman and Marcetteau (2004) showed that in the UK, about a third of construction projects are significantly delayed and of those, half of the delays are caused by problems in the ground. Applying this ratio to the overall Dutch construction failure rate would suggest that an average of some 2.5 billion euro are due to ground-related causes in the Netherlands, or probably equal to about 50 billion euro across the whole of the European Union; (the extrapolation is based on the size of the Dutch GDP in 2006 at US$613 billion to the whole of the European Union at US$13 600 billion; CIA factbook, 2007).

These ground-related problems often originate in an earlier phase than the phase in which they occur, as highlighted by Chowdhury and Flentje (2007), based on the extensive work of Sowers (1993). This latter study showed that in 57% of the projects studied, the geotechnical problems originated from flaws in the design phase and in 38% of the cases from the construction phase. However, these geotechnical problems actually materialised 41% of the time in the construction phase and 57% in the operational phase of the project.

An important part of the structural and geotechnical design processes should be for the whole project team to work together to reduce risks in the ground. Ground risks are one of the major causes of project delay and when they occur they are seldom resolved easily or quickly. As the installation of foundations

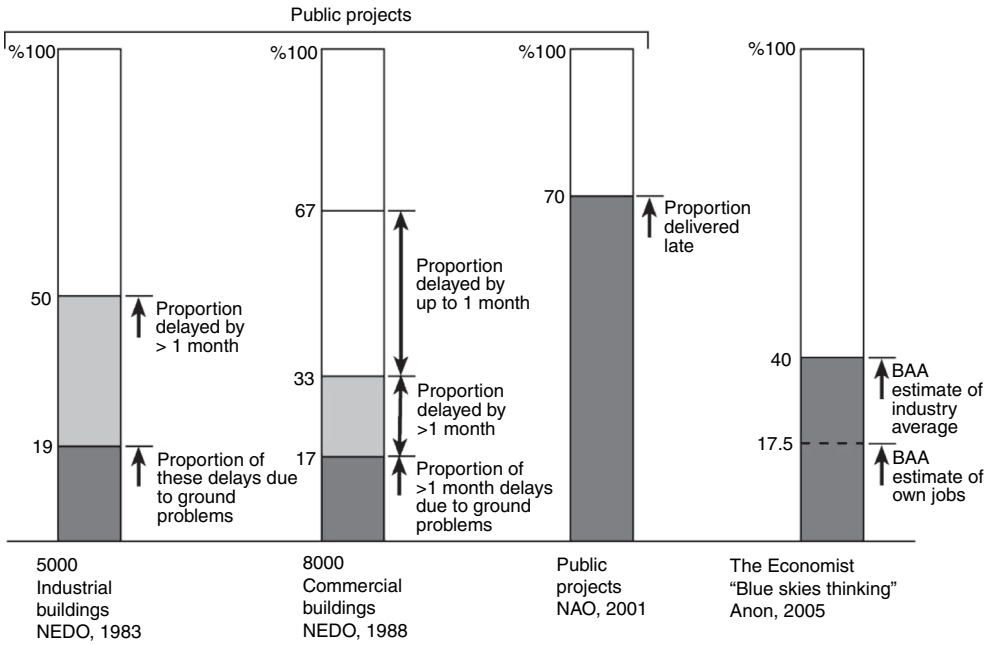


Figure 7.4 Typical UK delays for projects against time
The sources, shown below each project type, are all listed in the reference section of this chapter

and construction of basements are usually on the project's critical path, any delay affects all follow-on activities, and can impact directly on completion date.

7.5 Expect the unexpected

> … as we know, there are **known knowns**; there are things we know we know. We also know there are **known unknowns**; that is to say we know there are some things we do not know. But there are also **unknown unknowns** – the ones we don't know we don't know.
>
> Former U.S. Secretary of Defense, Donald Rumsfeld, explained a state of intelligence at a US Defense Department Briefing on February 12, 2002

To a large extent, this awareness of what might happen underpins successful geotechnical design. Managing risks for the 'unknown unknowns' is difficult as it is tricky to identify a likelihood and severity for unimagined events, which are not amenable to statistical understanding.

The concept of 'the 'Black Swan' was introduced by Nassim Nicholas Taleb (2007) – the name based on the unassailable belief by people in the Old World that all swans were white, confirmed by centuries of empirical evidence, until black swans were encountered in Australia in January 1697 (by Dutch explorer Willem de Vlamingh, http://birding.about.com/od/birdsswans/a/blackswan.htm). Taleb has created a whole theory based on the Black Swan, to which he attributes the following characteristics:

- as an outlier event, it lies outside the realm of regular expectations because no prior experience can point to the possibility;
- it has an extreme impact;
- despite being an outlier, once happened, human nature can concoct explanations for its occurrence – it is therefore obvious, but only with the benefit of hindsight.

Professor Taleb identifies many examples, ranging from the 9/11 events to the 'winner takes all' phenomenon for the wages of pop stars, footballers and artists. His thesis is that Black Swan events happen all too frequently and tend to dominate the success of many endeavours. He distinguishes between 'mild randomness' and 'wild randomness'. Dealing with ground risks is potentially a wildly random type of Black Swan because the consequences can be so severe, and so it is important that the possibility of unexpected events should not be discounted in geotechnical assessments.

It is possible by good planning to minimise the likelihood of a Black Swan. However, by the very nature of a Black Swan, their incidence can never be entirely eliminated. Those who manage complex geotechnical projects need to appreciate the possibility that they may be subject to Black Swans – a sense of humility is needed, plus an understanding of the range of catastrophic discoveries that can arise on any apparently innocuous site. Often Black Swans arise because engineers rely purely on previous experience and don't consider what might happen. This point is considered in greater detail in the example towards the end of this paper.

7.6 Importance of site investigation

> Without site investigation, ground is a hazard.
>
> Site Investigation Steering Group, 2003

As explained in Section 4 of this manual, site investigation consists of two main activities, as explained in BS5930 (BSI, 1999):

- Desk study, also known as *Preliminary Investigation* by Eurocode 7 (BSI, 2004) – the gathering of information about a site from documentary sources, made easier by free online aerial photography and other easily available resources, such as Envirocheck in the UK. A very valuable part of the desk study process is often to summarise all the ground-related hazards onto one site plan which can be used throughout the whole design and construction process. For larger (and more complex) sites, a geographic information system (GIS) can fulfil the same function.
- Ground investigation – the physical investigation of a site by boreholes and trial pits, with laboratory testing and reading of piezometers, to establish the ground and groundwater conditions. The ground investigation should be driven by the hazards identified in the desk study, together with an assessment as to how those hazards may affect the intended development proposals and construction processes.

In addition to normal geotechnical considerations like stratigraphy and stratum properties, it is important that the following aspects of a site are also investigated:

- potential presence of valuable archaeological remains;
- potential presence of contaminated ground or groundwater in the vicinity;
- potential presence of obstructions or other impediments to foundation installation, as well as neighbouring foundations, tunnels, etc.;
- buried cables and other services.

Often these man-made issues are a bigger risk for the project than normal geological variability. Desk studies that only consider, for instance, easily available mapping and not mapping data that go back to the start of the site's development, should be seen as potentially deficient.

The site investigation should also address constructability – it is therefore important that likely foundation types and processes needed on the site should be considered before the site investigation takes place. The risk of delay will be considerably increased if the contractors cannot derive useful information on the efficacy of installation for different types of foundation from the site investigation reports.

It is worth remembering that a site investigation is a careful but selective sampling of the ground, therefore the vast majority of the ground will not be encountered until main construction takes place. Using the soil volumes described in Appendix A of this chapter, a typical ground investigation will only recover some 0.03% of the soil beneath the site; only a fraction of that will be examined by a professional geologist or engineer.

The GeoQ process, introduced by van Staveren in 2006, is a formalisation of normal good geotechnical practice, emphasising the need for early data gathering. **Figure 7.5** shows a conventional contract, with the tendering stage between design and construction. A simplified example of applying the GeoQ steps, during a number of normal project stages, is shown in **Table 7.8**.

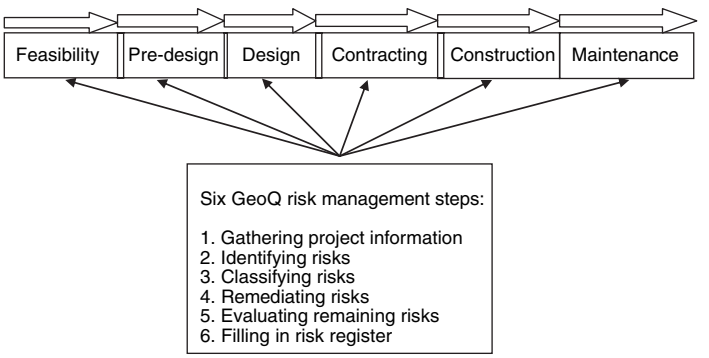


Figure 7.5 The GeoQ risk management model

GeoQ Step	Normal project stage	Typical example
1. Gathering project information	Desk study	Collecting historical and geological maps and other information
2. Risk identification		Discovery of old maps showing a filled-in watercourse crossing the site
3. Risk classification	Normally done during the **ground investigation** stage, when the magnitude and extent of the hazard can be quantified	Prove that channel is up to 10 m in depth and infilled with soft organic alluvium
4. Risk remediation	Normally done during **design**, when the development can protect itself against the main risks	Design of different foundations for that part of the site; inclusion of methane protection measures for basement
5. Risk evaluation	**Prior to tender**; collection of unmitigated risks so they can be managed and/or transferred to the contractor as part of the contract documents	Assess remaining extent of issues connected with channel
6. Risk mobilisation – Filling in the risk register		Highlight to contractor that his plant may get bogged down in alluvial formation; care of piling rig stability

Table 7.8 A simplified example of GeoQ steps during a few normal project stages

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7.7 Costs and benefits of site investigation

> Quality is never an accident; it is always the result of intelligent effort.
>
> John Ruskin (1819–1900)

Geotechnical risks can be managed and mitigated. Sound geotechnical practices – for example hazard recognition and exploration by means of the desk study and ground investigation – can reduce risks substantially. The value of these investments needs to be seen in the context of the risks reduced.

For the building considered earlier and described in Appendix A of this chapter, a site investigation could consist of:

Desk study		£5 000
Ground investigation		
5 boreholes 20–40 m	£25 000	
10 trial pits	£5 000	
Contamination testing	£5 000	
Soils laboratory testing	£5 000	£40 000
		£45 000

This needs to be seen in the context of the £24 million building cost excluding fit-out costs (or £13.2 million structural cost or £2.4 million substructure cost). This site investigation is equal to 0.19% of the building cost, 0.34% of the structure cost, and 1.9% of the substructure cost. Compare this with Rowe's (1972) estimate for site investigations of between 0.05% and 0.22% of building cost.

The relative costs for delay can be compared to the savings that may have contributed to them. Using the statistics given earlier, it can be assumed that some 20% of projects are significantly delayed by ground conditions, by perhaps 1 month or more. From the perspective of the developer, the cost of 1 month's delay can be calculated as follows:

Land purchase cost	£68.8 million
Building cost (with fit-out)	£31.2 million
Total Cost	£100 million

Taking 7% annual return, the annual cost to service the loan is £7 million or £583 000 per month. If the delay prevents the occupier from working in the building then the costs will be much higher, as they will also have to include the lost productive effort of the people working there, which is assumed to exceed the cost of employing them – a further £31.25 million per year (equivalent to £2.6 million per month) or nearly five times more than the building finance costs.

Of course the loss of use of the building could be much higher as these figures do not include:

- missing the top of a property rental cycle, so that the achieved rent over subsequent years is less;
- missing an important season, such as Christmas for a shop or summer for a holiday facility;
- having to pay any dispute costs, which can be several times more than the original loss being argued over.

The developer's art involves predicting, potentially several years ahead, when there will be most demand for property and initiating the planning, design and construction process in sufficient time to provide property that meets the demand. This is illustrated in **Figure 7.6**. Thus, he is already vulnerable to uncertainties in the state of the property market when his facility is completed, as well as uncertainties in construction cost and programme.

If the developer skimps on the site investigation, a saving of half of the investigation cost might translate into £22 500 'saved' (but often a poorly thought-out site investigation makes very little saving).

Omitting dispute and occupier costs (as well as contractor liquidated damages), the delay costs for 1 month might be:

Client – delay interest		£583 000
Main contractor		
Establishment	£100 000	
Extra investigation	£30 000	
Redesign	£30 000	£160 000
		£743 000

There may also be increased construction costs due to a less efficient foundation design caused by the inadequate site investigation. As an example, the London District Surveyors Association (2000) allows a design basis for piles with a good site investigation that is 20% more efficient than one that is deficient. Thus a poor site investigation may result in piles that are some 20% less efficient, which may cost 10% more to construct. Thus for the example under consideration, the extra foundation costs may be £60 000 more, increasing the £743 000 to £803 000.

The extra 'average' cost for deficient ground investigation can be calculated as £209 000, spread over all developments. This figure includes an average extra delay cost due to unexpected ground conditions of £149 000 based on 20% of the calculated typical delay cost of £743 000, and an allowance for less efficient design of £60 000.

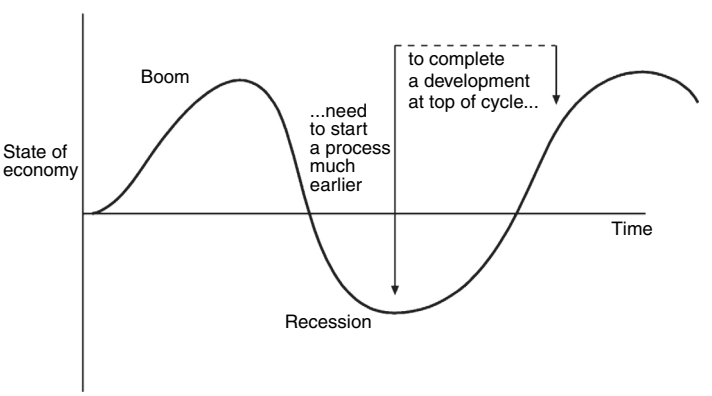


Figure 7.6 Graph showing how the start point of a development process is key to hitting the top of the economic cycle
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Thus a saving of half of the site investigation budget of £22 500 would have to be balanced against a conservative estimate for a delay of more than £800 000 on the fifth of projects where the risks materialise, or an average extra cost over all projects of £209 000. In many cases, the consequences are much more severe than the fairly optimistic assumptions made in this illustration.

7.8 Mitigation not contingency

> It is unwise to pay too much, but worse to pay too little; when you pay too much, you lose a little money, that's all. When you pay too little, you sometimes lose everything, because the thing you bought was incapable of doing the things it was bought to do. The common law of business balance prohibits paying a little and getting a lot. It can't be done. If you deal with the lowest bidder, it is as well to add something for the risk you run. And if you do that, you will have enough to pay for something better. There is hardly anything in the world that someone can't make a little worse and sell a little cheaper – and people who consider price alone are this man's lawful prey.
>
> John Ruskin (1819–1900)

Often when faced with a low tender price, clients concentrate on the direct saving and not on the consequential losses to which they may be subjected. They may believe that a contingency will provide them with protection, rather than choosing to resolve the issues directly. This can be valid when the consequences are 'mildly random' but not when they may cause the sort of severe 'wildly random' impact on project finances that were illustrated earlier.

Many developers bear the financial risk themselves, and if things go wrong in a way that invalidates the assumptions in their financial model, they are left to sort out the consequences. The main problems are usually delays, resulting in late handover (delaying receipt of income streams), or higher costs. Often a disappointed developer will wish to recover losses from his design and construction team, for instance by liquidated damages for delay from the contractor, or via the designers' professional indemnity insurance policies. These are not reliable methods for recovering losses, and especially consequential losses. Better ways of covering risks are to insure them explicitly by project insurance that covers the whole team, or by latent defects insurance (LDI) that covers the final building (although LDI doesn't always cover consequential losses). While these insurance products encourage a shared approach to resolving problems, they don't let designers and constructors completely off the hook. For instance, the insurer may later decide to try to recover some of his losses (by a subrogated claim) if he suspects negligence was a cause.

LDI should not be seen as a solution by itself, in the same way that ever tighter liabilities on contractors and designers are unlikely to resolve a deteriorating position to the benefit of a developer. The best way to avoid the sort of issues that result in delays and losses is to avoid the problems in the first place. Thus spending more on proper investigation of ground issues is likely to be more fruitful than engaging lawyers to devise more onerous contracts to impose on project participants.

7.9 Mitigation steps

> The rain came down, the streams rose, and the winds blew and beat against that house; yet it did not fall, because it had its foundation on the rock. But everyone who hears these words of mine and does not put them into practice is like a foolish man who built his house on sand.
>
> Bible – Matthew 7: 25–26

The standard project work stages (based on RIBA's Outline Plan of Work – RIBA, 2007) are shown in **Table 7.9**, along with typical geotechnical design activities.

Timely intervention with appropriate advice at the best time is needed for all projects. Geotechnical advisors need to be aware of the timescale over which the rest of the project is being designed and the potential implications of late advice, particularly for more complex projects where there should be much early interaction with other parts of the design team, especially the structural designers.

Often geotechnical elements will present a major constraint on other parts of the development – for instance on a tight site, a complete retaining wall system might be up to 2 m wide (e.g. 0.15 m for guide wall, 0.9 m for hard/soft secant wall, 0.15 m for tolerances and protuberances to specification, reinforced concrete facing wall of 0.30 m, void of 0.15 m, blockwork wall of 0.15 m, giving a total thickness of 1.80 m), yet sometimes architects hope that a 0.3 m reinforced concrete wall (implicitly expecting an open-cut solution) will be possible. This will have severe implications for space planning around the complete perimeter of the building, and may have worse consequences if a parking layout has been derived for the smaller wall size. It is therefore important for the geotechnical team to make sure that their indicative solutions are provided early. So, for instance, not providing a retaining wall design until the results of some advanced laboratory tests are known (and hence relatively late in the overall design process) – while protecting the geotechnical designer from possibly having to change his solution – may cause consternation amongst the rest of the design team.

Whenever a test is carried out on a site, it carries the risk of not conforming to the specification. It is therefore wise to test earlier (and ideally off the project's critical path) instead of leaving important test results to be measured later and at a time when a non-conformance will bring greatest disruption. As examples:

- A geotechnical baseline report (GBR), as recommended by the British Tunnelling Society (2003), is a good way of ensuring clarity on risk sharing for all project types. Rather than giving comprehensive coverage of the ground conditions as in a geotechnical interpretative report, a GBR sets out the expected conditions simply, so that deviations from them are clear. It therefore seeks to provide clarity regarding when extra payment to a contractor is due, thereby reducing the likelihood of disputes.
- A ground investigation should concentrate on the greatest uncertainties usually revealed by the desk study. A ground investigation

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Project phase	RIBA work stage	Corresponding geotechnical activities
Preparation	A. Appraisal Identification of client's needs and objectives, business case and possible constraints on development Preparation of feasibility studies and assessment of options to enable the client to decide whether to proceed	■ Preliminary desk study, identifying major hazards that may profoundly affect development proposals
	B. Design brief Development of initial statement of requirements of the design brief by or on behalf of the client, confirming key requirements and constraints Identification of procurement method, procedures, organisational structure and range of consultants and others to be engaged for the project	■ Full desk study, identifying all key hazards
Design	C. Concept Implementation of design brief and preparation of additional data Preparation of concept design including outline proposals for structural and building services systems, outline specifications and preliminary cost plan Review of procurement route	■ Ground investigation ■ Geotechnical interpretative report ■ Initial designs for various options ■ Advice on preferred options ■ Agree requirement for site supervision
	D. Design development Development of concept design to include structural and building services systems, updated outline specifications and cost plan Completion of project brief Submit application for detailed planning permission	■ Design development ■ Detailed calculations for preferred option ■ Agree requirement for site supervision if design relies on it ■ Final calculation package and meet building control standards
	E. Technical design Preparation of technical design(s) and specifications, sufficient to coordinate components and elements of the project; confirm that they meet statutory standards and construction safety	■ Start on tender documents for initial comment ■ Construction design and management submission
Pre-construction	F. Production information F1. Preparation of detailed information for construction. Application for statutory approvals F2. Preparation of further information for construction required under the building contract. Review of information provided by specialists	
	G. Tender documentation Preparation and/or collation of tender documentation in sufficient detail to enable a tender or tenders to be obtained for the project	■ Complete tender documents ■ Identify preferred foundation contractors ■ Contractors to identify principal construction hazards
	H. Tender action Identification and evaluation of potential contractors and/or specialists for the project Obtaining and appraising tenders; submission of recommendation to the client	■ Tender reviews ■ Agreement to winning contractors – name, design, proposals, etc.
Construction	J. Mobilisation Letting the building contract, appointing the contractor Issuing of information to the contractor Arranging site handover to the contractor	■ Brief resident engineer (RE)
	K. Construction to practical completion Administration of the building contract to practical completion Provision to the contractor of further information as and when reasonably required Review of information provided by contractors and specialists	■ RE supervision ■ Act as geotechnical advisor to the contractor ■ Assist with claims ■ Piling close-out report

Table 7.9 (continued)

Use	L. Post-practical completion
	L1. Administration of the building contract after practical completion and making final inspections
	L2. Assisting building user during initial occupation period
	L3. Review of project performance in use

Table 7.9 RIBA work stages with normal corresponding geotechnical activities
Data taken from RIBA (2007) and ACE (2004)

based on a poor desk study will inevitably be hit-and-miss. Considering that less than 0.03% of the soil beneath is seen, it will tend to miss more than it hits. A good ground investigation will target the most likely hazards, which will often be the near-surface stratum boundaries as man's activities will tend to be more erratic (in most cases) than geological processes.

- The initial choice of foundations can have a profound effect on the processes that follow; piles in silty soils with a high water table tend to go wrong more frequently. Choosing shallow foundations on dense sandy silts above the water table can obviate the hazard. However, if piles have been selected, and particularly if they are specified with the design to be done by the piling contractor, then a shallow foundation solution is unlikely to be selected later.
- Early testing of a preliminary test pile provides confidence in both the constructability and capacity of a chosen pile type, leaving sufficient time for the design or pile choice to be amended if problems develop. The same principle applies to other forms of early testing; for instance, testing for the efficacy of a ground improvement method or for problems during trial boring or trial drives.
- Late working pile tests (pile testing during the works), in addition to obstructing progress, also raise the possibility of adverse results with no time to react. Completion of pile load testing after installation of all of the works piles means that every pile may need to be augmented if an unexpectedly low pile capacity is measured, which can be very difficult to resolve.
- As well as reducing risks, doing more pile tests before and fewer tests during works pile installation is often cheaper and causes less disruption to a construction programme.

The process should be to consider all risks from the outset, then mitigate them according to their likelihood and potential severity. Risks should be owned and managed by those best able to do so, not just delegated or passed on. The stages for doing so are summarised against RIBA workstages in **Table 7.9**.

7.10 Example

> He who has not first laid his foundations may be able with great ability to lay them afterwards, but they will be laid with trouble to the architect and danger to the building.
>
> Extract from *The Prince*, Niccolo Machiavelli (1469–1527)

For an example of how the occurrence and severity of a geotechnical Black Swan event can be mitigated by sound engineering processes for various levels of study and investigation by a design team, see **Table 7.10**.

In the case of poor investigation of the risks before starting, there would be no inkling of the possibility of a potentially catastrophic feature; the risk would initially be evaluated based on an expected range of mildly pessimistic assumptions. It is unlikely that any allowance would be made in the cost planning for such a catastrophic feature with potentially dire consequences. Its occurrence would cost heavily – it could have the potential to cause serious injuries to site workers, especially if the collapse was exacerbated by the large construction equipment used, the possible disposal of water, or by significant vibrations from construction plant. In the first case, there is also a possibility that the feature might not even be discovered during construction. Often such features can collapse subsequently, resulting in the closure of an operational building with huge financial costs – and possibly with a risk of injuries or fatalities.

7.11 Conclusions

> Everybody, sooner or later, sits down to a banquet of consequences.
>
> Robert Louis Stevenson (1850–1894)

A frequent cause of problems on projects is a failure to resolve issues at an early stage. There are many issues where the associated risk can be effectively mitigated during:

- initial data gathering;
- design;
- procurement of construction.

Many clients procure their design consultants on the assumption that all will provide design products of similar quality and that price should be a major basis for differentiating between them. For simpler projects with easily defined deliverables and where external influences are more easily controlled, price can be a valid comparator, especially if the design consultants have been pre-qualified for their skill and relevant abilities. For more complex projects, what matters most is the quality of the output, such as:

- collection of data into a very coherent and useful desk study report;
- consideration of buildability and ways to resolve such issues;
- better integration of temporary and permanent works solutions.

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Site status – amount of investigatory work undertaken	Example of what might be known with the stated extent of investigation	Likely engineering approach given state of knowledge	Potential consequence if foundations were to be installed with state of knowledge	Rumsfeldian classification of feature when its presence was detected	Validity of approach
No desk study with hit-and-miss ground investigation	No knowledge of a large old mineshaft on the site	Evaluate site as normal	Major safety risk for site workers, e.g. collapse failure of a foundation during installation	Unknown unknown	Low known costs but high risk of subsequent failure. Also likely to have long dispute about costs and responsibility
Just a desk study, but no ground investigation	Knowledge there could possibly be a mineshaft somewhere on the site	Know of possible presence but not its poor condition	Feature's location and condition to be discovered so it is likely that there will be site delay while condition of feature is investigated	Known unknown	Possibly low known costs but still risk of significant delay and dispute
Desk study and ground investigation	Know there is an old mineshaft, its location and dimensions. Also know that its condition is poor and it is at risk of collapse	All aspects known – best foundation solution chosen at start; contractor knows how to cope with issue at tender time	Condition known in advance so low risk of extra direct costs or delay. Therefore unlikely to be a dispute	Known known	Potentially higher known costs but much lower risk. Development should be on time and to budget without debilitating row

Table 7.10 Illustration of risk mitigation and consequences

The mine shaft example in **Table 7.10** illustrates the very different consequences from various levels of preparation and study by the design team prior to the start on site.

If total design fees are 5% of the project cost, then an improvement in output quality with 20% more effort (more time or more experienced input) can translate into a 1% increase in project cost. Hopefully this will translate into much bigger savings in out-turn project cost in terms of:

- a lower incidence of hazards causing problems;
- a more deliverable programme;
- smarter ideas resulting in smaller capital cost.

For projects where viability is very tight, a greater investment up-front is even more important, otherwise the project is more likely to encounter unexpected events that will turn it into a financial failure. Perversely, it is often on such projects where most effort is made to minimise design costs that the likelihood of a Black Swan event occurring is inadvertently increased.

This chapter has sought to demonstrate:

- That while foundations are a relatively small part of the overall project cost (perhaps 7%), their problems can be responsible for half of significant project delays, occurring some 17–20% of the time.
- Typically only 0.05–0.22% of the project's building cost are spent investigating those risks, even though 'average' consequences are many times more.
- Developers' financial models are very vulnerable to unexpected shocks – such as project delays, especially when funded by loans, which is usually the case.
- Geotechnical uncertainties can be 'wildly random' rather than 'mildly random' – the effects of a problem can quickly magnify and be disproportionately expensive to solve. Therefore management by mitigation is far more effective than management by contingency.
- A coherent geotechnical risk reduction process can significantly reduce risks. A comprehensive desk study is one of the most cost-effective steps.
- The design, by admitting the possibility (or even probability) of some uncertainty, can go further in reducing the incidence of risk.
- Employment of experienced contractors who make proper allowance for uncertainty will generally provide a more reliable solution than a cheap solution by a less experienced contractor. Poor procurement and poor contractor capabilities were highlighted by the UK Treasury as major contributors to cost and programme overruns on government-funded projects.
- A testing strategy that confronts risks at an early stage by early testing, e.g. load testing of piles, and which reduces the incidence of unexpected or anomalous results during the main construction works will reduce programme vulnerability.
- Clarity of risk ownership with the best strategy being ownership of the risk by those best able to manage it – not just dumping of risks on those at the bottom of the supply chain. Early consideration and mitigation at the opportune time are better than late reaction to a deteriorating situation.
- Reliance on damages and negligence is not an effective way for developers to manage their risks with their constructors and designers – they will seldom recover all their consequential losses which will usually dwarf any direct losses.

- Only very seldom are ground engineering problems caused by gaps in engineering knowledge. Most problems are caused by a failure to apply known processes adequately.
- Communication and information sharing will often result in earlier identification of problems, enabling them to be solved earlier and more easily.

This chapter illustrates that there are no short cuts to gathering pertinent information, digesting it into a form that can be shared across the whole team and using it throughout the whole design and construction process. The main aim should therefore be for robust solutions and overall reliability. The reduction of ground risk needs better application of existing processes, not necessarily better science.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

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Appendix A
Example development

This example is an idealisation to provide indicative total costs for a typical building development. It is acknowledged that the total and relative costs will vary enormously for different developments. Assumptions are based on a site slightly smaller than 1 ha (1 hectare = 10 000 m^2) in a central area of London. As the site will accommodate 17 000 m^2 (183 000 ft^2) of net lettable area, the land price was set at £68.8 million (the site value, based on net lettable area is assumed to be £376/ft^2 – mid-way in the 2009 typical range of £250–500/ft^2). On the site is placed a 20 000 m^2 gross area high quality air conditioned office building, six storeys high with a single level basement for cycle and car parking and plant. The net lettable area is therefore 85% of gross floor area. A generous allowance of 17 m^2 per person gives a total building population of about 1 000. An average salary for the occupants of this building has been taken as £25 000 per annum. Employer's payroll costs are taken as an extra 25%, giving an average employee cost of £31 250. The total salary roll per year is therefore £31.25 million. Over a 25 year building life, the total employment cost will have been about £781 million (expressed in constant 2009 £).

A building cost has been calculated as typically £1 200/m^2, giving a total of £24 million. The costs have been apportioned as about 10% for substructure (foundations and basement), 45% for frame, floors, walls and roof, and 45% for finishes and services. Of the £2.4 million for substructure, it has been assumed that £600 000 went on piles and pile caps with the rest for the basement structure. Category A fit-out of the building has been assumed to cost some 30% of the total building cost (£7 million). The tenant's fit-out has been taken as £10/m^2 (£200 000), giving a total of about £7.2 million for fit-out. The total building costs for construction and fit-out are therefore £31.2 million, which equates to £1 560/m^2 or £145/ft^2. Professional fees are assumed to be included in the construction cost figures. The annual operating costs are assumed to be approximately £55/m^2 per year, giving a total of £1.1 million per year, or a total of £27.5 million over a 25 year design life. The site investigation costs have been taken as £5000 for a desk study and £40 000 for a ground investigation, including interpretation. These yield a total site investigation cost of £45 000. This represents about 0.19% of the building cost excluding fit-out (which was £24 million); this is typical for the site investigation proportion of structural costs which vary between 0.05% and 0.22% for building projects collected by Rowe in 1972.

Based on the hazards identified in the desk study, it has been assumed that a site less than 1 ha will need some 5 boreholes to depths ranging from 20 m to 40 m, 10 trial pits and associated testing for geotechnical properties and contamination, both on-site and in the laboratory. Assuming a site of 90 m × 90 m and considering the relevant soil depth that might influence the development down to 40 m, the total volume of soil for which geotechnical representation is needed is 324 000 m^3. The soil actually seen on site will be as follows:

- boreholes: $5 \times (0.15\,m)^2\ \pi/4 \times 30\,m$ average depth = 2.65 m^3
- trial pits: $10 \times 3\,m \times 1\,m \times 3\,m = 90\,m^3$.

This gives a total of around 93 m^3. The proportion of soil recovered by the ground investigation is therefore some 0.03% of the total. The proportion examined by a professional engineer or geologist will be a much smaller proportion again.

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Chapter 8

Health and safety in geotechnical engineering

Delwynne Ranner Balfour Beatty Ground Engineering, Basingstoke, UK
Tony Suckling Balfour Beatty Ground Engineering, Basingstoke, UK

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Construction is a dangerous occupation with on average 75 deaths and 3 330 major injuries reported every year in Great Britain. Safety is everyone's responsibility from client and designer to contractor and we must all work towards reducing the human cost of our profession. Every construction project is unique with a particular combination of contractors, subcontractors, processes, materials, hazards and risks. Maintaining safety is the primary focus of any site; there are legal obligations to be met by all directly involved in any project and all associated with it. It is the responsibility of all to identify risks and then to mitigate against them. Geotechnical engineering has its own specific hazards and risks. Safety on each and every project must be planned, continuously monitored and improved by competent personnel as part of the 'way we work' every day. Responsibility for safety cannot be delegated.

8.1 Introduction

Each and every construction project will have a unique combination of particular contractors, subcontractors, processes, materials, hazards and risks. Maintaining safety is the primary focus of any site, and if it isn't it should be. There are legal obligations to be met by all directly involved in any project and all associated with it. Clients must ensure that competent people are appointed to manage the project, designers must ensure that risks are identified and eliminated during the design phase, the principal contractor and contractors on site must manage and monitor the processes to ensure the work is completed safely. It is everyone's responsibility to identify risks and to mitigate against them. The accident record of the construction industry is improving, however in relation to most other industries construction remains dangerous with on average 75 deaths and 3 300 major injuries reported every year in Great Britain. Everyone, from the most senior to the most junior on any project, has the right to expect to work in a safe place and to return home at the end of the working day. Geotechnical engineering has its own specific hazards and risks, and these are generally described. Safety on each and every project must be planned, continuously monitored and improved by competent personnel as part of the 'way we work' everyday. Responsibility for safety cannot be delegated.

8.2 An introduction to the legislation

Health and safety has taken on great importance in the construction industry, partly due to increased health and safety legislation and partly due to pressure exerted on the industry because of its poor safety record. This chapter gives a brief overview of some of the liabilities under such legislation and is not intended to be a detailed reference on the subject. It should be noted that much of the legislation described applies only to England and Wales; however, the legislation applicable to Scotland and Northern Ireland is almost identical in most respects. For projects outside the United Kingdom different legislation will apply.

Most health and safety legislation is goal setting: the legislation describes broad principles which are expected to be achieved; the detail of how to achieve these principles is primarily left to the employer. Any breach of health and safety legislation is a criminal offence and as such both fines and/or imprisonment can be imposed, the levels of penalties are set out in the Health and Safety (Offences) Act 2008. Conviction can result in a criminal record. Health and safety law places a reverse burden of proof on any defendant. It is up to the defendant to demonstrate that they have complied with the legislation rather than the prosecutor having a responsibility to demonstrate that they have not.

The Health and Safety at Work Act 1974 (HSAWA) imposes duties on all parties (including the employer and the employee) with the purpose of ensuring the health and safety of people at work. The HSAWA is an enabling act which means there are many regulations, often accompanied by approved codes of practice (AcoPs) and guidance notes that take effect under the act. The HSAWA gives appointed inspectors wide reaching powers of investigation and enforcement, including the serving of improvement and prohibition notices.

The Management of Health and Safety at Work Regulations (Health and Safety Executive, 1999) places duties on employers and the self-employed in relation to others affected by their work. The main duties placed on employers are the assessment and reduction of risk, the need to establish emergency procedures, health surveillance, and to provide information and training for employees. The duties in respect of risk assessment in this legislation are the basis of all risk assessment in the workplace.

The Construction (Design and Management) Regulations (CDM) were first enacted in 1994 and were substantially revised in 2007 (Health and Safety Executive, 2007). The CDM places duties on everyone involved in construction works and is one of the most significant pieces of legislation governing

them. The 'duty holders' are divided into six key categories: client, CDM coordinator, designer, principal contractor, contractor and worker. Each of these duty holders have responsibilities if the project is 'notifiable' (notification to the Health and Safety Executive is required if the project is expected to last more than 30 days or 500 person days). However, one of the significant changes to the 2007 regulations was the imposition of duties on some duty holders when a project is classed as 'non-notifiable'. A summary of the main responsibilities of the duty holders is provided in **Table 8.1**.

CDM requires that anyone who wishes to appoint or engage someone to do a construction activity, or undertake design work, must be satisfied that the person they want to appoint has sufficient knowledge and experience to carry out the tasks. Accidents and major incidents often occur because the people carrying out potentially complex tasks have been appointed because they are the cheapest and can do the job the quickest. This is as true for geotechnical works as for any other construction activity. There have been many incidents where contractors have not understood the correct sequences (see Chapter 79 *Sequencing of geotechnical works*) or have not been experienced enough to undertake the job – thereby being ignorant of the hazards and risks involved. All persons have this duty; so a structural designer who needs to engage the services of a geotechnical engineer must ensure that the person is competent, and not appoint on cost alone.

A client should ultimately be satisfied that his whole design, procurement and implementation team is competent. He should therefore ensure that if members of the project team appoint sub-contractors, etc., they have rigorous procedures in place for assessing competencies.

Designers from all disciplines have a contribution to make in reducing and avoiding health and safety risks which are inherent in the construction process and subsequent work, e.g. maintenance and demolition. The most important contribution a designer can make to improve health and safety will often be at the concept and feasibility stages. For geotechnical works, maintenance and demolition are often ignored as being irrelevant but they should always be considered.

The main duty of designers of 'structures' is to assess the implications of their design on the health and safety of people affected by the building when in use and during maintenance of the 'structure'. This is a significant departure from previous safety legislation where most responsibility was placed on contractors. The definition of a 'structure' is wide-ranging and includes items such as tunnels, bridges, pipelines, drainage works, earthworks, lagoons, caissons and earth-retaining structures. It also refers to any formwork, scaffold or falsework which is designed to provide support or access during construction.

Once construction is complete, CDM requires the client to retain the health and safety file. This file is also commonly referred to as the operations and maintenance manual (O&M Manual), and the revision of the regulations in 2007 significantly reduced the amount of detail required in it. The focus of the O&M Manual is to provide information on the 'structure' relevant to the health and safety of those using, maintaining, repairing or renovating the structure. Information on the safe demolition of the 'structure' should also be included.

CDM (Health and Safety Executive, 2007) also made changes relating to the provision of welfare. The Construction (Health, Safety and Welfare) Regulations 1996 were revoked and their requirements included within CDM. Both the client and the principal contractor have responsibilities for ensuring that suitable welfare facilities are available prior to construction works starting.

It is important to note that there is no specific health and safety legislation for geotechnical engineering. British Standards such as BS 8008 (1996) *Safety Precautions in the Construction of Large Diameter Boreholes for Piling and Other Purposes* provide accepted standard practice; whilst they have no legal status they are often referred to in order to establish if legal duties have been adequately discharged. However, the specific requirements of geotechnical design and construction have to be considered within the legislative requirements for the whole industry.

Other legislation which may need to be considered includes, but is not limited to, the following:

- Control of Substances Hazardous to Health (COSHH) Regulations (1999)
- Manual Handling Operations Regulations (1992)
- The Provision and Use of Work Equipment Regulations (1998)
- The Lifting Operations and Lifting Equipment Regulations (1998)
- Personal Protective Equipment Regulations (1992)
- The Work at Height Regulations (2005)
- The Control of Vibration at Work Regulations (2005)
- The Control of Noise at Work Regulations (2005).

For more detailed information refer to the *ICE Manual of Health and Safety in Construction* (McAleenan and Oloke, 2010).

8.3 Hazards

A hazard can be described as the property of an item which has the potential to cause harm. This could apply to almost any item if used in a particular way; combining two or more items would give an infinite number of hazards which may need to be considered on any project.

Ground investigation works are hazardous as the personnel are often the first onto the site as part of the development. The principal contractor may not have been appointed at this stage, so the specialist ground investigation organisation will be required to perform his duties whilst on site. As these works are, by their very nature, an investigation, it should always be anticipated that there will be hazards which will not have been identified or even anticipated by the desk study. Also, if these works are to be undertaken inside an abandoned building, then consideration must be given to particular hazards

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	All construction projects	Additional duties for notifiable projects
Clients	Check competence and resources of all appointees	Appoint CDM coordinator*
	Ensure there are suitable management arrangements for the project welfare facilities	Appoint principal contractor*
	Allow sufficient time and resources for all stages	Make sure that the construction phase does not start unless there are suitable welfare facilities and a construction phase plan is in place.
	Provide pre-construction information to designers and contractors	Provide information relating to the health and safety file to the CDM coordinator
		Retain and provide access to the health and safety file
CDM coordinators		Advise and assist the client with his/her duties
		Notify HSE
		Coordinate health and safety aspects of design work and cooperate with others involved with the project
		Facilitate good communication between client, designers and contractors
		Liaise with principal contractor regarding ongoing design
		Identify, collect and pass on pre-construction information
		Prepare/update health and safety file
Designers	Eliminate hazards and reduce risks during design	Check client is aware of duties and CDM coordinator has been appointed
	Provide information about remaining risks	Provide any information needed for the health and safety file
Principal contractors		Plan, manage and monitor construction phase in liaison with contractor
		Prepare, develop and implement a written plan and site rules. (Initial plan completed before the construction phase begins)
		Give contractors relevant parts of the plan
		Make sure suitable welfare facilities are provided from the start and maintained throughout the construction phase
		Check competence of all appointees
		Ensure all workers have site inductions and any further information and training needed for the work
		Consult with the workers
		Liaise with CDM coordinator regarding ongoing design
		Secure the site
Contractors	Plan, manage and monitor own work and that of workers	Check client is aware of duties and a CDM coordinator has been appointed and HSE notified before starting work
	Check competence of all their appointees and workers	Cooperate with principal contractor in planning and managing work, including reasonable directions and site rules
	Train own employees	Provide details to the principal contractor of any contractor whom he engages in connection with carrying out the work
	Provide information to their workers	Provide any information needed for the health and safety file
	Comply with the specific requirements in Part 4 of the Regulations	Inform principal contractor of problems with the plan
	Ensure there are adequate welfare facilities for their workers	Inform principal contractor of reportable accidents, diseases and dangerous occurrences
Workers	Check own competence	
	Cooperate with others and coordinate work so as to ensure the health and safety of construction workers and others who may be affected by the work	
	Report obvious risks	

* There must be a CDM coordinator and principal contractor until the end of the construction phase

Table 8.1 The main responsibilities of the duty holders
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(e.g. asbestos) and a structural assessment must be undertaken prior to the ground investigation.

Most geotechnical works are specialist and are normally undertaken by specialist subcontractors. The subcontractor often undertakes both the design and the construction of their element of the works and so has the best understanding of the particular hazards and risks associated with the product or process. The principal contractor and the engineer are most likely to have the best understanding of the specific risks and hazards of the overall project. Therefore all parties must work together, both before and during the works, to identify the hazards and introduce appropriate mitigation or control measures.

A list of general hazard areas which must be considered on any construction project is provided below, with more detailed discussion of some of the hazards associated with geotechnical works provided in the clauses which follow. This list is not exhaustive and the unique combination of factors which go to make up any project means that each project must be reviewed on its own merits. Some areas for consideration are:

- Access and egress to the place of work for vehicles and pedestrians.
- The movement of vehicles on site, including the need to separate vehicles and pedestrians wherever practical.
- Protection of the public (almost every construction project will have some interaction with the public; consideration must be given as to how to ensure they remain safe and that the project inconveniences them as little as possible).
- Protection of the environment (watercourses, ground water, habitat, species, etc.).
- Provision of welfare for workers (this can be a particular challenge on geotechnical projects which may be small projects of a transient nature or large projects which stretch over many miles).
- The effects of weather (provision must be made for ensuring adequate lighting during winter months, workers must be able to dry wet clothes, the effect of wind speed on tall plant may need to be considered, and how to maintain safe ground conditions in heavy rain or snow).
- Site security (not only must provision be made to ensure that members of the public do not inadvertently access the works areas, but consideration should also be given to maintaining records of who is on site and what they are doing).
- Designing for safety (consideration must be given to designing out risks wherever practical; this should include any temporary works structures and geotechnical works in the temporary state).
- Emergency situations (consideration needs to be given to dealing with incidents on site such as the provision of first aiders, the location of the closest hospital and mechanisms for workers to report emergency situations).
- Management of works on site (including the programming of works to ensure that risks are minimised both from a design point of view and taking into account the physical working space).
- The elimination of the need to work at height, or the control of the risk of falling (this should include the risk of falling into excavations).
- Protection of workers' health (workers' short- and long-term health can be affected by a number of things such as noise, dust, biological hazards, manual labour and vibration. These hazards must be identified and controlled).

8.3.1 Shallow excavations

The stability of the sides of excavations depends on the ground conditions, including the effects of groundwater, as well as the depth of excavation and the surcharge placed on the sides of the excavation. It is important to note that stability is likely to decrease with time and can reduce very quickly by the introduction of water from precipitation or elsewhere. Unfortunately the number of people killed having been trapped in unsupported excavations remains constant year on year. Regulations do not specify any requirement for the support of excavations, other than that this should be reviewed by risk assessment. However, an assessment of stability should be based on sound engineering design principles and not judgement. Excavation stability must be assessed by a competent person taking into account items such as ground conditions, weather conditions, activities adjacent to the excavation and the movement of groundwater.

Consideration must also be given to access/egress to excavations (for people, plant, equipment and materials), maintaining the safety of the excavation with regards to water ingress, underfoot conditions and the possible build up of gases. Measures must be taken to ensure that people, equipment and/or vehicles do not fall into the excavation.

Care must also be taken to ensure that when an excavation has been completed it is backfilled with appropriate material and compacted where required. This is of particular importance where other activities will be using the area. Plant tracking over a poorly reinstated excavation is at significant risk of overturning. This not only applies to shallow excavations, but also to pile bores and deep urban excavations.

8.3.2 Piling works

Piling works use specialist plant which can weigh up to 140 tonnes, can be up to 30 m high, is front heavy, and the driver usually does not have full visibility around the machine. A safe exclusion zone must be maintained around the piling plant, the public must be protected from risk, and vehicle and plant movement around the piling plant must be properly controlled. Piling rigs require a significant amount of space to operate safely and the specialist contractor should be consulted as early as possible to ensure that the project is sequenced in a way which ensures that this space can be provided and maintained.

Three particular hazards associated with piling works are:

- Design, maintenance and repair of the working platform, see **Figure 8.1** and Chapter 81 *Types of bearing piles*. The platform must be adequate for the full duration of its use. Soft or hard areas beneath the platform, either existing before or created after construction of the platform, can cause rig instability.
- Failure of a pile load test reaction system, see **Figure 8.2**. Significant and real forces are applied through the test pile and

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Figure 8.1 Photograph showing a piling rig which overturned due to the platform not having sufficient capacity to support the bearing pressure of the machine



Figure 8.2 Photograph showing a reaction test set-up which collapsed when a component part failed

reaction system, and each and every component must be designed and constructed rigorously.

- Site personnel getting caught up in rotating parts such as drill strings, augers or other rotating parts of the specialist machinery. The HSE has recently been very active in promoting fixed guarding around rotating parts and reliance on exclusions zones is no longer considered adequate. This has implications for the safe location of piles in restricted areas and when positioned within say 2 m of any physical barrier.

8.3.3 Deep urban excavations

Figure 8.3 shows an example of a congested city centre project which requires the excavation of a deep basement.

The particular hazards here include:

- The effect of works on adjacent structures.



Figure 8.3 A typical congested city centre project

- Sequencing of works is of particular concern where ground movement could occur as excavation of basement structures progresses.
- Protection to members of the public.
- Planning and sequencing the works to ensure that there is sufficient space for each of the contractors to work safely.
- Use of tower cranes. These have a unique set of hazards and must be appropriately planned, designed, registered, maintained and operated by competent persons.

8.3.4 Contaminated ground

Short- and long-term health problems can arise from contact with contaminated ground. Potentially dangerous substances need to be identified, the related risks assessed and safe systems of work put in place in accordance with the Control of Substances Hazardous to Health Regulations (COSHH).

In 1992, the British Drilling Association published *Guidance Notes for the Safe Drilling of Landfills and Contaminated Land*, and these were subsequently adopted by the Institution of Civil Engineers Site Investigation Steering Group and republished in 1993 as Part 4 of the publication *Site Investigation in Construction*. The British Drilling Association is currently producing an updated document called *A Guide to Safe Site Investigation*. Anyone involved with drilling into the ground, whether thought to be contaminated or not, should refer to such specialist documentation.

Guidance on the range of potential contaminants and the means of controlling the related risks is provided in HSG 66 (1991) – *Protection of Workers and the General Public during Development of Contaminated Land*. Specialist advice when dealing with contaminated land must be sought.

8.4 Risk assessment

Risk assessments are used as a means of documenting, defining and managing risk as a response to the Management of Health and Safety at Work Regulations. Risk identification and evaluation is the process of looking at the hazards identified,

calculating the severity of the outcome if their potentials were realised, and checking who might be affected. Risk control is ensuring that the hazard's potential cannot be realised. Therefore the risk assessment process, put simply, is figuring out how to keep people safe and then making a note of what has been decided, and why.

All geotechnical processes will have associated hazards and risks; these are identified and defined in the risk assessment form. Mitigation measures (controls) are introduced to hopefully eliminate the hazard. If complete elimination of the hazard is not possible then measures are introduced to reduce the risk of harm to an acceptable level. What defines an acceptable level of risk depends on a number of factors including the legislative requirements. Competent advice should be sought from geotechnical specialists when reviewing to check that proposed controls measures are adequate.

The best mechanisms are to avoid or eliminate hazards and risks; however if this is not possible then consideration must first be given to 'preventative' control measures. These are generally controls which work for a large number of people at the same time and do not require human input, e.g. using a fixed guard rail around an excavation. If preventative measures cannot be adopted then protective measures should be considered next. Protective measures will generally only work for one person at a time and will require the input of that person – for instance, he may be required to wear a safety harness to prevent a fall into an excavation. This whole process is documented in the risk assessment form. All control measures must be put in place, monitored and maintained, so that the risk to health and safety remains at a low level. Project-specific hazards and risks are dealt with as part of the same process.

The risk assessment process cannot be programmed to be undertaken at a particular point in a project, as the hazards and risks associated with a project will change on a daily basis. The process of identifying hazards, assessing risk and implementing appropriate controls is an ongoing one which should involve everyone who is associated with the project. Many hazards can be effectively eliminated and/or controlled long before the work on site starts, provided the right people, with the right skills, are involved in the planning process.

The other vital part of the risk assessment process is ensuring that the people who are going to do the work have the right information in order to keep themselves safe. Any significant risks on a project must be briefed to the workers. This is usually done in an induction. Inductions do not need to be long and complex; they should simply explain the key site risks and controls, the emergency procedures and the site rules.

Most construction companies have their own processes and procedures in relation to identifying and recording hazards and risks. Specialist geotechnical organisations are the same. These processes are usually based around the principles described in the Management of Health and Safety at Work Regulations and expanded on in the HSE's publication *Five Steps to Risk Assessment* (Health and Safety Executive, 2006) (INDG163).

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8.5.2 Useful websites

Health and Safety Executive; www.hse.gov.uk
Environment Agency; www.environment-agency.gov.uk
British Standards Institution; www.bsigroup.com
Sentencing Guidelines Council; www.sentencing-guidelines.gov.uk
Institute of Occupational Safety and Health; www.iosh.co.uk
Institution of Civil Engineers' Health and Safety website; www.ice.org.uk/topics/healthandsafety/

All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

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Chapter 9

Foundation design decisions

Anthony S. O'Brien Mott MacDonald, Croydon, UK
John B. Burland Imperial College London, UK

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The objective of this chapter is to dispel the widely held view that foundation engineering is simply a matter of designing and then constructing a foundation to carry a given load. In practice, many key decisions have to be made in developing the conceptual design before detailed design calculations can be undertaken. An approach to tackling these key decisions is given. For foundations to be constructed economically and sustainably it is necessary for the geotechnical and structural engineer to work closely together from an early stage. Examples are given of the benefits associated with close cooperation across the design team. Great emphasis is placed on recognising that foundation engineering is a 'process' involving a number of interlinked operations. Success depends on the skill of the engineers and operators and the way these operations are monitored and modified as work progresses. The chapter contains a number of case histories describing problems that emerged during construction and how these were solved. They illustrate how important it is to keep the 'geotechnical triangle' in balance.

9.1 Introduction

In 1951 Professor Karl Terzaghi, in a lecture given at the Building Research Congress in London, said (Terzaghi, 1951):

> On account of the fact that there is no glory attached to the foundations and that the sources of success or failure are hidden deep in the ground, building foundations have always been treated as stepchildren and their acts of revenge for lack of attention can be very embarrassing.

The aim of this chapter is to describe the decisions and procedures that have to be undertaken in arriving at a foundation design and in successfully constructing the foundation and the structure it is intended to support. It is important to understand right from the start that foundation engineering is essentially a process. The process involves a number of interlinked operations, and success depends on the skill of the engineers and operators and the way these operations are monitored and modified as work progresses. Thus the design and construction of a foundation is critically dependent not only on the choice of foundation, but on numerous other factors including the ground and groundwater conditions and the way they are influenced by the construction method, the skill and experience of the contractor, the supervision and quality control of each operation and a clear understanding of the performance requirements of the structure resting on the foundations.

A number of case histories are included which are chosen to emphasise the importance of the geotechnical engineer, the contractor and the structural engineer working closely together to ensure that the process as a whole is successful.

9.2 Foundation selection

Many textbooks cover the topic of foundation design; however, the engineer reading these textbooks could be left with the impression that foundation design is solely concerned with the calculation of bearing capacity and settlement. In practice, many of the key decisions have been made before bearing-capacity and settlement calculations are carried out. A useful mnemonic for the foundation designer is the 'five S's':

- **Site:** What are the nature, topography and size of the site? Are there major constraints, e.g. limited headroom, major underground services? Are there settlement-sensitive structures adjacent to the site? What is the site history, e.g. is it underlain by historic mining, old buried foundations?
- **Soil:** What is the soil (or ground) profile and groundwater regime? This should be understood in the context of the local geology and hydrogeology. Do competent soils exist at shallow depth? Will groundwater seriously affect foundation construction?
- **Structure:** The size of structure, its structural form and layout, the type, magnitude and direction of applied loads, together with the allowable foundation movements, will be major factors in foundation selection.
- **Safety:** Numerous ground-related hazards may affect the site. These may be due to the intrinsic hazards associated with the soil/rock types which underlie the site and the way they may have been affected by post-depositional geological processes (such as landslips), or to human activities, such as historical industrial processes which have left a legacy of ground/groundwater contamination, or hazardous gases, or mining, quarrying or non-engineered materials (dumped fills or landfills) which are potentially unstable. Reference should be made to Chapters 8 *Health and safety in geotechnical engineering*, 40 *The Ground as a hazard* and 41 *Man-made hazards and obstructions* of this manual, in addition to guidance provided by the Health and Safety Executive, e.g. HS(G)66 (Health and Safety Executive, 1991), and CIRIA guides, e.g. CIRIA reports 132 (Steeds *et al.*, 1996) and 552 (Rudland *et al.*, 2001).
- **Sustainability:** This requires a consideration of environmental and social factors and the economic implications of different

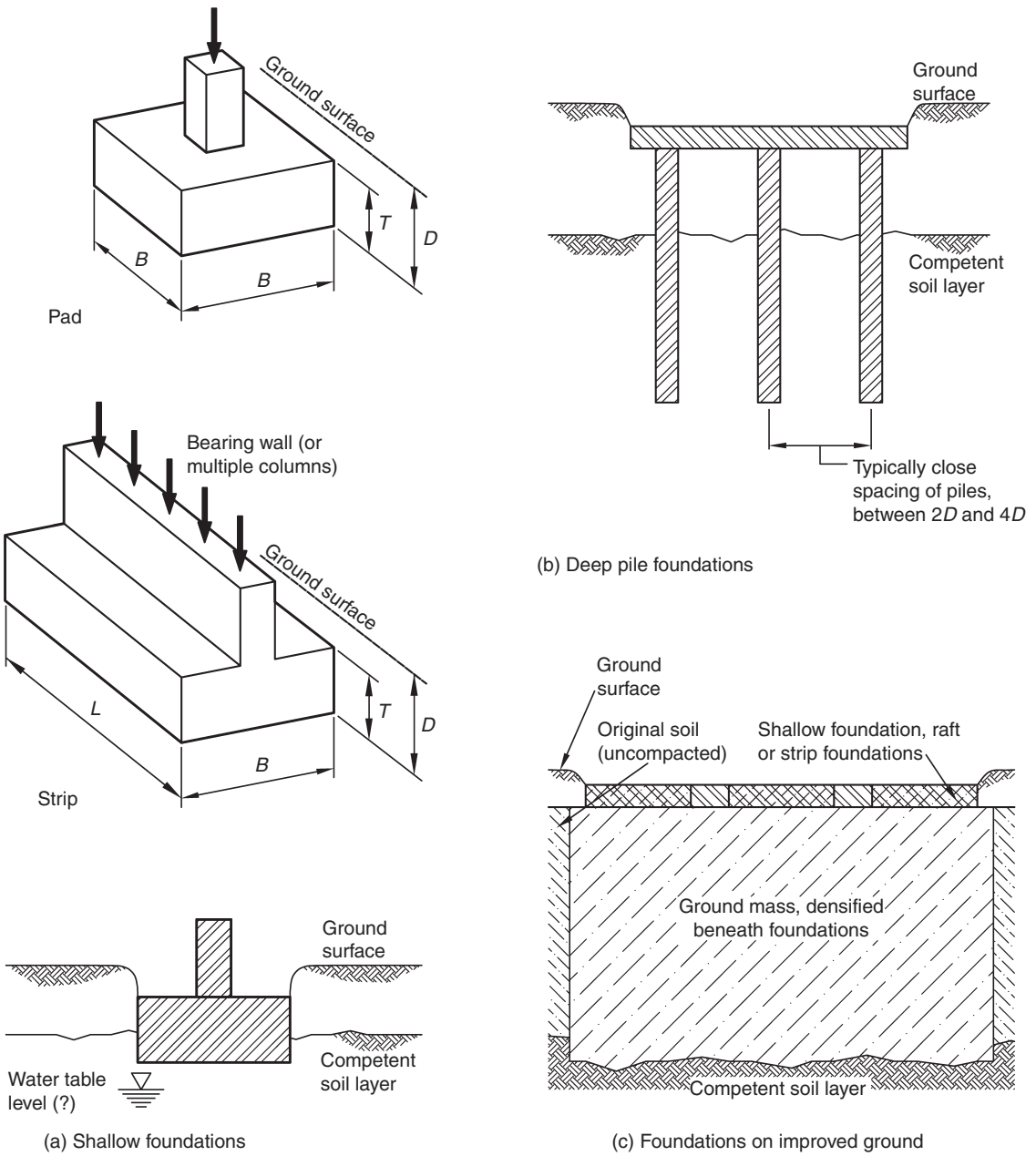


Figure 9.1 Foundation types

foundation options. The aim should be to minimise the environmental and social impact of foundation construction, providing the costs are reasonable. Environmental impacts include an assessment of embodied energy, and the carbon footprint of different options. Social factors would include potential disruptions to adjacent communities from construction traffic, noise, vibration, etc.

The above 'five S's' are discussed in more detail in Chapter 52 *Foundation types and conceptual design principles*. At the end of this chapter a case history is given, which provides a practical example of the factors which will influence foundation selection.

Foundation types are commonly classified into two types (**Figure 9.1**):

(i) *Shallow foundations* – these include: discrete pad footings which usually support a single superstructure column; strip footings which may support a wall; and raft foundations which support the entire superstructure. These are discussed in more detail in Chapter 53 *Shallow foundations*.

(ii) *Deep foundations* – most commonly these include piled foundations, either as single piles below superstructure

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columns/piers (refer to Chapter 54 *Single piles*) or as several relatively closely spaced piles acting together as a pile group to support either the entire superstructure or heavily loaded parts of the structure (refer to Chapter 55 *Pile-group design*). More specialist deep foundations include deep shafts, caissons and barrettes.

It is also useful to consider a third generic foundation type (**Figure 9.1**):

(iii) *Hybrid foundations* – this type comprises elements which are common to both shallow and deep foundations. Examples include: deep ground improvement (to increase the bearing capacity and/or reduce the compressibility of soils below the structure), with shallow foundations constructed on the improved ground; piled rafts where the structure is supported by a raft and a few widely spaced piles. Chapter 59 *Design principles for ground improvement* discusses ground improvement design principles and Chapter 56 *Rafts and piled rafts* discusses piled rafts.

The ground profile and the allowable foundation movement are two key factors in selecting the most appropriate foundation type. If competent founding strata are close to the ground surface then shallow foundations should be considered, since they are usually relatively simple and cheap to construct. However, as foundation loads increase and/or allowable movements are reduced, then the use of piled foundations will become more appropriate. Often, the decision is fairly simple, for example, if there is a deep layer of heterogeneous Made Ground covering the project site, and the foundations are heavily loaded (say for a multi-storey public building or large bridge), then the use of piles will be obvious. However, if the proposed structure applies modest loads and is relatively tolerant of ground movement (say a steel frame single-storey warehouse), then it may be possible to found on the Made Ground (or non-engineered fill), especially if a deep-ground improvement technique can be used to strengthen/stiffen the Made Ground. The important topic of foundation engineering within non-engineered fills is considered in Chapter 58 *Building on fills*.

For some sites the foundation designer may be faced with a difficult decision in deciding upon the most appropriate foundation type. The 'five S's' will then need to be considered in great detail and the designer will have to juggle with a wide range of factors, such as ground strength/compressibility, allowable movement, available space for construction equipment, potential impacts on neighbours and the local community, and the ease, speed and safety of different construction options. Once the advantages/disadvantages are considered and balanced for each option, then a final selection of foundation types, and their dimensions, can be made. Some of the factors that will affect these decisions are subjective, and will be affected by perceptions of how to best minimise risks, especially during construction. This inevitably varies between individuals and organisations and will also reflect local construction practices (which vary markedly around the world). Common factors in properly understanding ground risks are:

(i) Understanding the site history and geology. This requires a comprehensive desk study to be carried out (refer to Chapter 43 *Preliminary studies*).

(ii) Understanding the site's groundwater regime and how it impacts upon the proposed construction and permanent works.

In the vast majority of cases when foundation problems occur it is because either (i) or (ii) above has not been properly understood by the foundation designer. A coherent approach to proper understanding of ground risk is provided by the 'geotechnical triangle' (introduced in Chapter 4 *The geotechnical triangle*), and practical examples are provided in section 9.4 of this chapter.

Figure 9.2 provides an example of the wide range of pad and pile sizes and shapes for the support of a nominal 9000 kN load on a deep layer of stiff overconsolidated clay. Although all the options are 'safe', in the sense that they all have an acceptable factor of safety against bearing-capacity failure, their expected settlement and the issues which need to be considered for construction all vary markedly. For example:

(i) Pad foundation: Is the expected settlement (at 100 mm) too large? If yes, can the pad be made larger, to reduce settlement to a tolerable amount? Is there sufficient space to form the pad? If the pad is excavated, will it undermine adjacent structures or slopes?

(ii) Pier foundations: If deep excavations are required, can an open excavation be formed (at a stable slope angle) or will temporary retaining walls be needed? Or will specialist caisson/shaft sinking methods be more appropriate? Again available space across the site will be a critical consideration. Will excavation need to be carried out below the water table? If the groundwater table is to be lowered to facilitate excavation, what settlement will be induced? What volume of water will be extracted and can it be disposed of?

(iii) Under-reamed piles: Will the ground be stable enough to form an under-ream? This will often depend on the detailed 'fabric' of the ground, together with the groundwater regime. Do silt/sand layers exist within the clay (which may lead to the under-ream becoming unstable prior to concreting), or is the clay uniform, and of low permeability?

(iv) Large diameter (1.2 or 1.5 m diameter) piles: These will require a large heavy piling rig to be mobilised to site, which will be expensive. Will a lot of large diameter piles be needed, or is this just for a few isolated foundations (hence the overall foundation requirements need to be considered for the whole project)? Are there significant weight restrictions for heavy plant, either due to weak surface soils, or weak bridges to gain entry into the site?

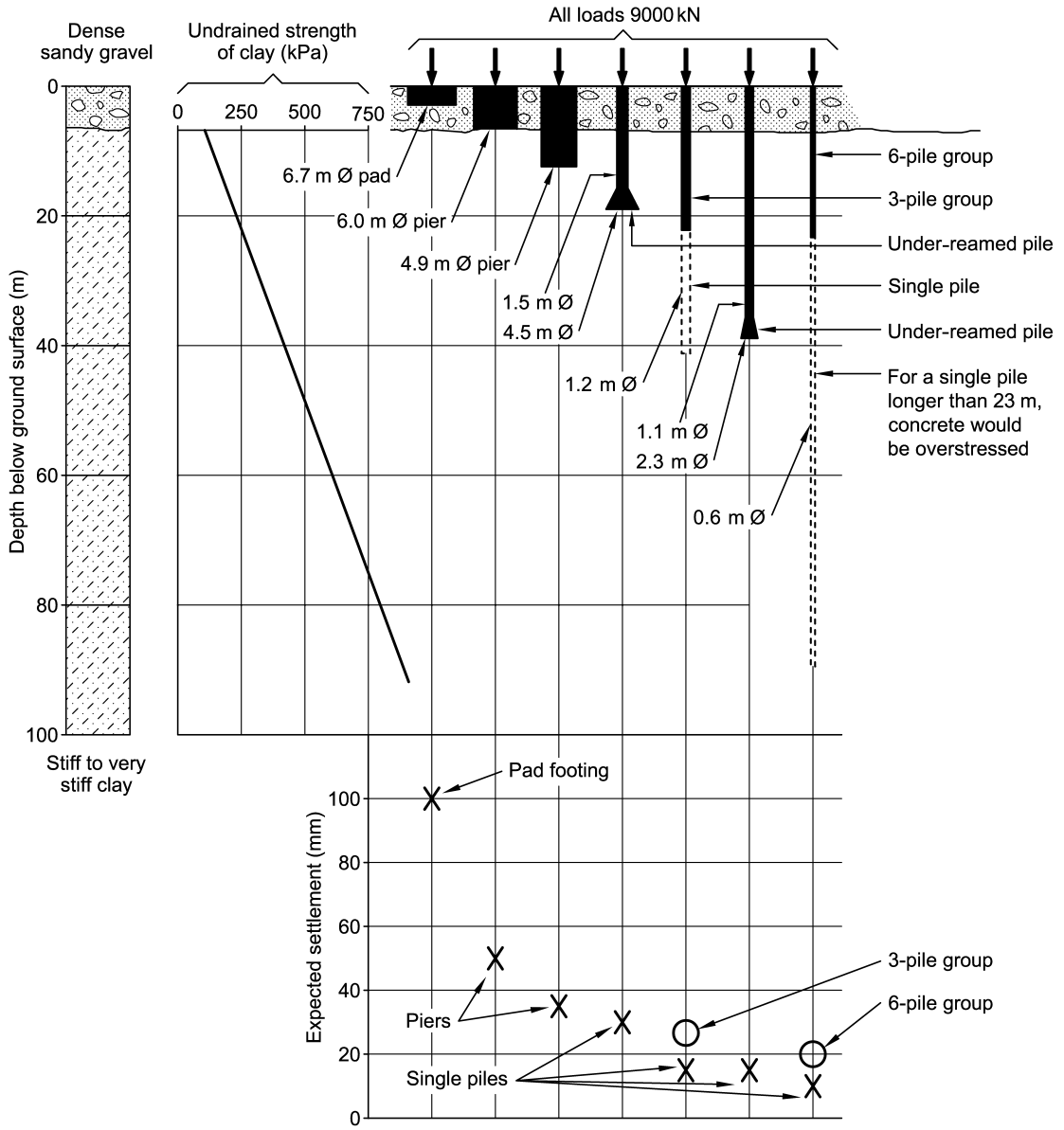


Figure 9.2 Methods of supporting a 9000 kN load
Reproduced from Cole (1988)

(v) Medium diameter (0.6 m diameter) piles: Is there sufficient headroom across the site to construct piles using conventional plant? For bored piles, will any temporary support be needed, e.g. temporary casing through surface gravels or support fluids at greater depth (if the clay is inter-bedded with silts/sands/gravels which lead to pile bore instability, especially if groundwater pressures are high)? Headroom and pile bore stability will also need to be considered for large diameter piles.

For smaller, and less powerful, piling rigs an important consideration will be the risk of hitting obstructions, for example, if old foundations are buried beneath the site, or if hard rock layers need to be penetrated. In these circumstances large diameter piles may be preferred.

Clearly when practical construction issues are being considered, early discussions with specialist contractors will be helpful. However, it is important for the foundation designer to develop a broad knowledge of construction issues, so that discussions with specialist contractors can be meaningful.

The predicted settlements for the different foundation types, shown in **Figure 9.2**, vary by an order of magnitude; hence an understanding of allowable foundation movement is clearly important when selecting the most appropriate and cost-effective foundation type. Often the limits set for allowable

total and differential settlement are arbitrary and overconservative. It is important for the foundation designer to have early discussions with the structural engineer on this matter. Usually total settlement is used as an indicator of what is 'allowable'; however, structural damage will be a function of *differential* rather than total settlement. It is also important to note that it is the differential settlement which occurs *after* the structure is built (and structural connections formed, and sensitive finishes completed) that will lead to damage to the structure. Hence, a considerable percentage of the total settlement that will occur during construction (due to the dead weight of the major structural elements) will be built out and have no impact on the risk of structural damage. This topic is considered in further detail in the context of foundation design, in Chapters 26 *Building response to ground movements* and 52 *Foundation types and conceptual design principles*. Finally, it should be noted that the main opportunity for economic foundation design is to set realistic (rather than overconservative) limits on allowable foundation movement, especially differential movements.

9.3 A holistic approach to foundation engineering

Most textbooks tend to focus on the behaviour of a foundation element (say a strip footing or a pile) acting in isolation. However, practical foundation engineering requires the foundation designer to consider the overall site redevelopment strategy, and in particular how different foundation types may be used in order to ensure the foundations are safe, simple and economic to construct.

Foundations are assemblages of different components, which may include various ancillary processes to facilitate design or construction (such as groundwater control, grouting, ground anchors, etc.) together with the use of retaining walls (either as temporary or permanent works). The interrelationship between foundation components, ancillary processes and assemblages is illustrated in **Figure 9.3**. A simple example would be the construction of a bridge abutment adjacent to a river bank (**Figure 9.4**):

(i) The *bridge abutment* is a composite structure. The stem of the inverted T retaining wall retains the approach embankment fill, and the base of the T acts as a pad footing to support the bridge deck and traffic loads. The settlement of the abutment is affected by the loads and settlement induced by the adjacent embankment, as well as those directly applied from the bridge.

(ii) *Temporary excavations* will be required for construction and may involve a combination of temporary sheet piling and/or groundwater control.

(iii) On the *adjacent river bank* the stability of the slope will need to be checked (under varying river levels), and the river

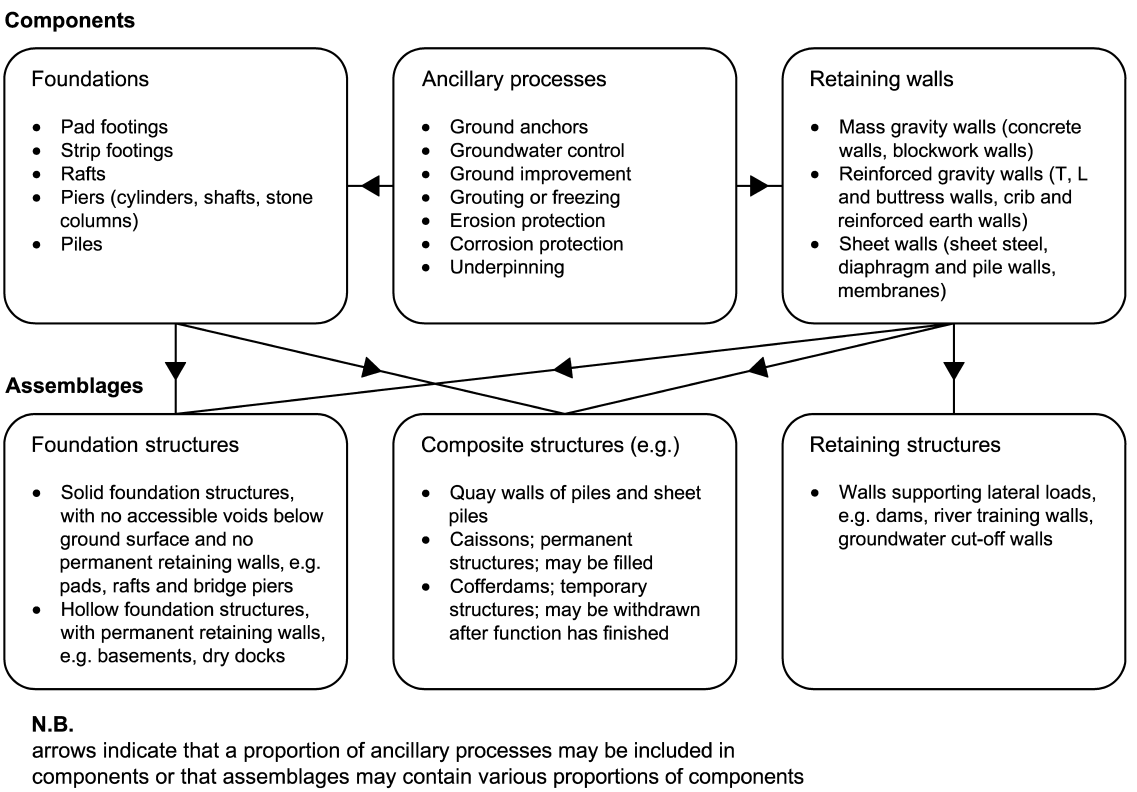


Figure 9.3 Relationships between components, ancillary processes and assemblages
Reproduced from Cole (1988)

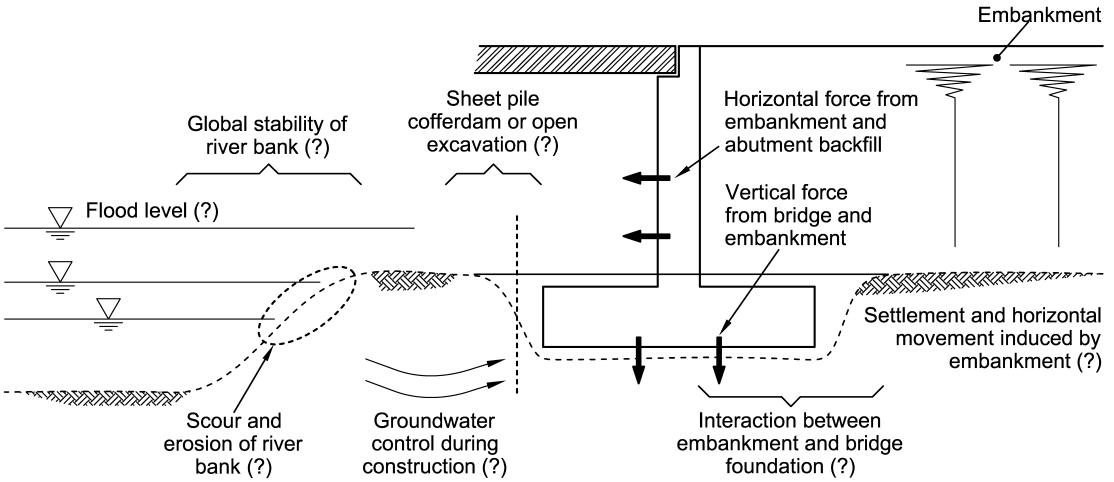


Figure 9.4 Foundation components, ancillary processes and assemblages: a bridge abutment adjacent to a river

bank will need erosion protection, so scour does not lead to the bridge abutment being eroded and undermined.

(iv) The interaction between the above elements also needs to be considered – for example, will the temporary excavation or temporary surcharge loads for the bridge abutment construction destabilise the river bank? Will the ground movements associated with the approach embankment adversely affect the bridge abutment?

Sometimes these different components of a project (say tasks (i), (ii) and (iii) above) are designed by different teams within an organisation, or by different organisations. It is important that potential interactions are considered by a single team and preferably this should be the lead project designer; otherwise serious problems can occur.

9.3.1 Examples of circumstances that influence foundation design

Because of the huge variety of ground and groundwater conditions which may be encountered and the widely different circumstances in which foundations are constructed, foundation design may involve a wide range of components, ancillary processes and assemblages. The main factors which influence those circumstances are: ground shape and location relative to other natural features (rivers, banks, slopes); adjacent site use and structures (surface and underground) and their sensitivity to ground movement/vibration; the site history; proposed superstructure requirements (allowable movements) and loads; the site geology and hydrogeology. A few practical examples are discussed below and illustrated in **Figures 9.5** and **9.6**:

(i) *Ground shape and location*. The overall shape of the ground (its geomorphology) both within and around the project site can have a major influence on foundation design and construction. A walkover survey by an experienced geomorphologist may allow the risk of, say, historic landslides being encountered to be assessed (Phipps, 2003; Fookes *et al*., 2007). Other examples are: sites in mining areas, which may be subject to subsidence and local instability; sites affected by large-scale erosion processes and/or flooding. The overall shape of the site may need to be changed to give a more satisfactory layout for the project development, for example by cutting and/or filling to form terraces, or to increase/decrease the overall site level. Although this may be necessary for the project, it may substantially increase the complexity of the foundation engineering because it will cause significant 'global' ground movements to develop (for example, settlement below deep areas of fill, or heave/swelling beneath deep excavations). If cut/fill is carried out on sites underlain by clays, these global ground movements may take several years to fully develop. Hence, additional forces (both vertical and horizontal) and ground movements would need to be considered in the design of individual foundation components. Chapter 57 *Global ground movements and their effects on piles* considers the effects of global ground movements on pile design.

(ii) *Site history and constraints*. Nowadays 'greenfield' sites, i.e. sites which have had no previous structures or industrial use, are increasingly rare. Obstructions, constraints and various hazards are commonly found on the site of a new project. Some sites may contain remains of archaeological value. Carrying out a comprehensive desk study at an early stage of the project is by far the best way of identifying these potential problems. Guidance on how to discover as much as possible about a site prior to carrying out ground investigation is given in Chapter 43 *Preliminary studies* of this manual. Some sites affected by industrial activities may be underlain by chemically contaminated materials (these may have originated at the site under consideration, or have 'leaked' in from neighbouring sites,

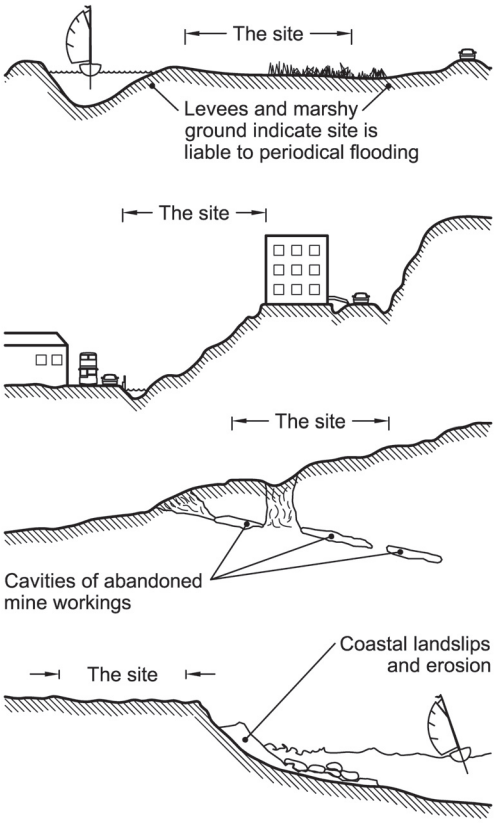


Figure 9.5 Influence of shape of ground on foundation design and construction
Reproduced from Cole (1988)

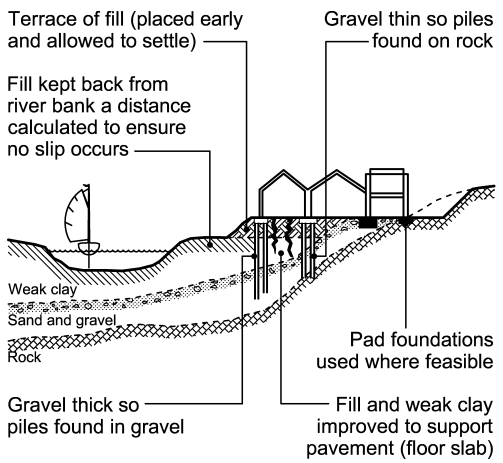


Figure 9.6 Example of how site re-levelling requires several foundation engineering activities
Reproduced from Cole (1988)

or have been spread across an area due to poorly controlled demolition). These contaminated materials may have impacted the soil or groundwater, or be in the form of noxious/explosive gases. If contaminated soils impact a site then the foundation designer will need to work closely with a contaminated land specialist and the relevant regulatory authorities, and it is likely to have a significant impact on foundation design/construction. Contaminated land guidance is given in a wide range of publications including those by the Department of the Environment, Environment Agency, etc. CIRIA reports, such as C552 (Rudland *et al*., 2001) provide a good introduction.

(iii) *Site geology and hydrogeology.* The composition and structure of the ground and the groundwater conditions, both now, during construction, and in the long term, during the design life of the structure, affect the choice of foundations. For example, consider the foundations for a multi-storey building. At one extreme, the site may be underlain by deep deposits of very soft alluvial clays/silts and peat, over an irregularly weathered and fractured rock mass affected by historic coal mining (a relevant case history is discussed in Chapter 52 *Foundation types and conceptual design principles*). The foundations may require numerous deep large diameter piles. Prior to piling, an ancillary process, involving grouting of the mineworkings, will be required to stabilise the site and avoid collapse of the mineworkings in the future. At the other extreme, a massive layer of a strong rock (say granite) with only a few widely spaced discontinuities may be encountered close to the ground surface. This rock could support the superstructure frame (columns/walls, etc.) with little enlargement to form the footings (this rock type would be stronger than the foundation concrete). Between these extremes lie a vast range of different combinations of ground conditions. Perhaps less obvious than the ground types which underlie the site are the potential influence of groundwater conditions on foundation engineering and how they may fluctuate seasonally and in the long term and how these may affect foundation behaviour. **Table 9.1** provides a check list of some groundwater-related factors which may affect foundation design, and an associated commentary. An additional challenge is how rapidly the groundwater regime will lead to changes in ground behaviour, for example softening of clays adjacent to an excavation. This rate of change is affected by the mass (or bulk) permeability of the ground. At one extreme are highly permeable soils, such as 'clean' gravels (i.e. with negligible silt/clay content) and at the other extreme are low permeability homogenous clays. This range covers a change in permeability of about seven orders of magnitude, between 10^{-3} m/s and 10^{-10} m/s. Most natural soils comprise varying amounts of clay, silt and sands; and the 'fabric', i.e. the arrangement of silt/sand layers/lenses within clays, or vice versa, has a major influence on the ground's mass permeability. Laboratory measurements of permeability are usually misleading, and *in situ* permeability testing is preferable (refer to CIRIA report C515 (Preene *et al*.,

Swelling and softening of strata below excavation	For clays, as effective stresses reduce, the strength reduces and compressibility increases. In rocks, joints may open (or joint infill materials may soften), and overall rock mass strength/stiffness is reduced. Repeated wetting/drying of weak rocks may lead to breakdown of rock structure and large loss of strength.
Dewatering during construction and re-establishment after dewatering switched off	In clays/silts: dewatering of adjacent more permeable soils (such as sands/gravel) can lead to a reduction in pore water pressure in the clay/silt, leading to consolidation settlements. After dewatering is switched off, 'rebound' will occur, potentially leading to swelling (depending on foundation bearing pressure and net increase/decrease in effective stresses). In sands/gravel: dewatering may lead to a loss of fines and substantial differential settlement, and ultimately ground instability. Hence, appropriate filters and observations are essential to avoid loss of fines.
Excavation, basal instability	Due to excessive groundwater pressures or hydraulic gradients. Even if ground failure does not occur, there may be substantial loss of ground strength/stiffness.
Erosion and scour	Surface water flows may lead to undermining of shallow foundations, or loss of lateral support to shallow and deep foundations. Erosion protection essential in high risk situations. Silts and fine sands are especially vulnerable.
Frost action	Leads to volumetric expansion, due to ice lense formation and/or freezing of water in cracks. Local codes of practice usually specify minimum foundation depths to avoid adverse effects.
Groundwater level changes (low lying areas may be vulnerable to flooding)	Natural variations in groundwater table level may occur, or man-made changes may occur (e.g. due to burst water main). These may lead to uplift forces on foundations, and a requirement for anchors and/or drains/relief wells to avoid buoyancy failure.
Collapsible soils	These soils are either very loose unsaturated soils, or (more commonly in the UK) loose non-engineered fills. If these soils wet up due to a water table rise, then a large 'collapse'-type settlement may occur.
Expansive soils	Desiccated clays, located above the water table, will be prone to substantial swelling pressures and volumetric expansion (heave or swelling) if the clay wets up. High-plasticity clays are most vulnerable. In the UK, this problem is usually associated with vegetation effects and seasonal wetting/drying (see below).
Vegetation effects and seasonal wetting/drying	Large trees (especially high-water-demand varieties), together with high-plasticity clays, can induce substantial seasonal foundation movements due to winter wetting/summer drying of the clays. Chapter 53 discusses this problem in more detail. If trees have been removed, then the desiccated clay will slowly wet up over several years, as pore water pressures re-equilibrate, leading to large swelling and uplift forces on foundations.

Table 9.1 Groundwater-related issues which may affect foundation design and cause large foundation movements

2000)). It is important to note that the groundwater effects outlined in **Table 9.1** may lead to substantial foundation movements which are independent of the bearing pressures applied by the structure. These effects will often be overlooked by a structural or general civil engineer, and it is important for the geotechnical engineer to carefully consider groundwater-related effects and associated risks.

9.4 Keeping the geotechnical triangle in balance – ground risk management

9.4.1 Background

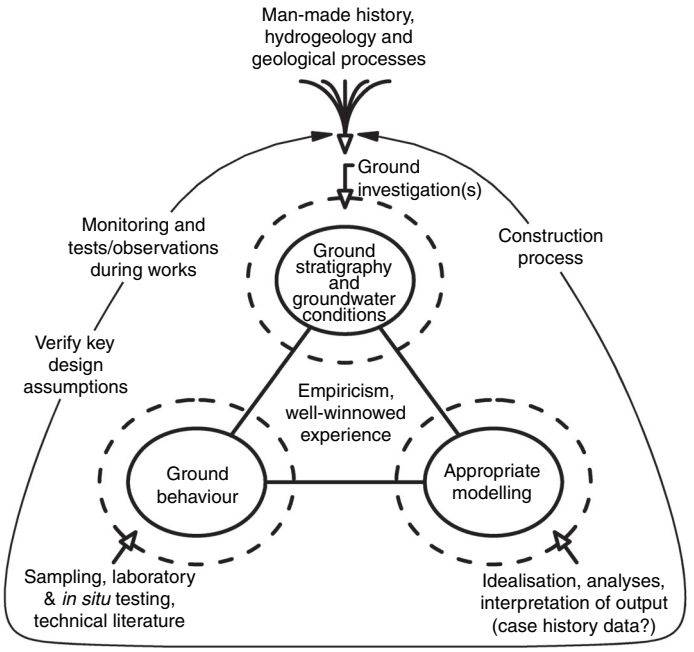
During the last 50 years or so, research developments in soil mechanics and the related engineering sciences have led to immense improvements in knowledge and understanding. Enormous developments in computer technology and numerical modelling enable engineers to analyse highly complex geotechnical problems, which would have been unthinkable a few decades ago. Alongside these theoretical and technical developments have been major improvements in practical construction techniques which enable a much wider range of ground conditions to be safely engineered and extremely large foundation loads to be resisted. Despite all these improvements, major geotechnical difficulties still regularly occur, causing costly delays to projects, and sometimes more severe failures which cause damage to adjacent infrastructure and injury or death to members of the public and construction workers. There are numerous reasons for these failures, including the fragmented nature of the industry, deficiencies in procurement of specialist services, and competitive and time-related pressures. At a technical level a major problem can be the lack of a coherent approach to geotechnical engineering and a systematic approach to ground risk management. In Chapter 4 *The geotechnical triangle* the geotechnical triangle has been described. The four main components of the geotechnical triangle (**Figure 9.7**) are:

(1) *Understanding the ground profile* (and the associated geological processes and man-made activities that formed the ground profile) *including groundwater conditions.*

(2) *The measured or observed mechanical behaviour of the ground* – this includes carrying out laboratory and field (or *in situ*) testing, and careful interpretation of the test data, in the context of the proposed foundation requirements.

(3) *Appropriate modelling* – this is much more than carrying out an analysis. It involves: idealisation, or simplification, of the real soil/structure; identifying and carrying out appropriate analyses; validation and assessment of the

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N.B.
This geotechnical triangle is modified via the outer circle for 'construction process'; this emphasises the importance of construction processes in foundation engineering, in the context of the potential impact of various processes on key design assumptions

Figure 9.7 The geotechnical triangle, foundation engineering applications

output. Several iterations may be needed, and a particularly difficult part of the process is selecting appropriate input parameters (based on (1) and (2)).

(4) *Empirical procedures and 'well winnowed' experience* – empiricism is an inevitable and essential aspect of geotechnical engineering. It is also important to understand the limitations of available empirical procedures. Experience is important in order to make the difficult judgements which are often required; however, the term 'well winnowed' emphasises the point that the experience needs to be founded on a proper understanding of the facts and key behavioural mechanisms associated with past activities and observations.

For practical applications of the 'geotechnical triangle' it has been helpful to consider a fifth component to the triangle (the 'outer loop' shown in **Figure 9.7**):

(5) *Construction processes and design verification* – the information which is available to the foundation designer is seldom perfect (for example, contradictory information about the site's history, variable data from ground investigations, etc.). Hence, several important assumptions often have to be made to develop a design. Therefore, it is important that these assumptions are verified during the construction process. There are a wide range of activities which may have to be carried out, and a few examples are provided in **Table 9.2**. At a basic level, it may be simply a visual check during construction that the geology of the founding stratum for, say, a shallow footing, and its strength, is the same as that assumed for design. For more demanding projects it may involve sophisticated tests of deep piles. Different organisations/design teams may be involved in the foundation engineering activities at different stages of the project. Throughout this process it is important that key assumptions are communicated to other parties, so that ultimately these assumptions can be checked when the foundations are being constructed.

All five components are important, and problems are likely to occur if one component is forgotten or poorly implemented and the triangle goes 'out of balance'. Outlined below are several short case histories, which briefly summarise some foundation engineering problems that have occurred on UK construction projects.

9.4.2 Shallow foundations, adjacent to a landslip

Figure 9.8 provides a plan and cross-section of the site. Shallow foundations, comprising pad and strip footings, were planned for an out-of-town retail development. A ground investigation had been carried out, which included five boreholes and several trial pits. These indicated a thin layer of Made Ground (typically less than 2 m thick) over rockhead which comprised strong limestone. In one corner of the site a shallow retaining wall (about 1.0 to 2.0 m high) was required to create extra space, in order to construct the shallow foundations. The low-height retaining wall was formed from proprietry pre-cast elements with a shallow foundation. Whilst the excavations were being carried out for the retaining wall and shallow foundations at the toe of the slope, a large landslip was triggered. This caused significant delay to the project and public concern, since the landslip caused the closure of a footpath and adjacent public areas. What had gone wrong?

(i) A geological desk study had not been carried out. The positioning of the boreholes and trial pits by the structural engineers had been based mainly on ease of access, rather than technical considerations.

(ii) Inspection of geological maps indicated that a fault passed through the site; unfortunately all the boreholes/trial pits were located on one side of the fault. On the other side of the fault, the ground conditions were quite different with inter-bedded mudstone and siltstone.

(iii) Based on a subsequent geomorphological inspection of the area, it was clear that the area to the west of the fault had suffered from past landslips, and the natural slope was at a factor of safety close to 1.0. Even quite minor excavations were sufficient to reactivate the landslip and cause further instability.

Foundation engineering issue	Possible tests/observations[1]
Confirm founding strata	Inspect excavated surface and compare with design assumptions, simple visual check. For layered strata and/or deep foundations, may need extra boreholes (continuous sampling), if GI boreholes too widely spaced.
Assess variability of strata	Usually simple *in situ* tests (static cone CPT, or dynamic probing) appropriate to check *relative* hardness or density of strata across site, if heterogeneous conditions expected.
Groundwater regime	Important if construction below water table required. Install piezometers in key strata. If dewatering necessary, large-scale pumping tests may be required, prior to works commencing.
Impact on adjacent structures	Precise levelling of affected structures from regular array of survey pins. Subsurface measurements may be needed in some circumstances, especially if lateral movements expected (inclinometers).
Foundation load-deformation	Large-scale load tests on key foundation elements, especially if sensitive to construction/installation, such as deep bored piles. Tests will be more complex if lateral load behaviour needs to be checked.
Deep ground improvement	Verification testing is usually critically important for controlling the effectiveness of the works. Type of test dependent on improvement mechanism, e.g. reinforcement, densification or providing increased cohesion, by addition of cementing agents, or reduction of permeability.

(1) The list is not intended to be comprehensive and merely highlights some common examples.

Table 9.2 Examples of verification tests/observations which may be required during foundation engineering

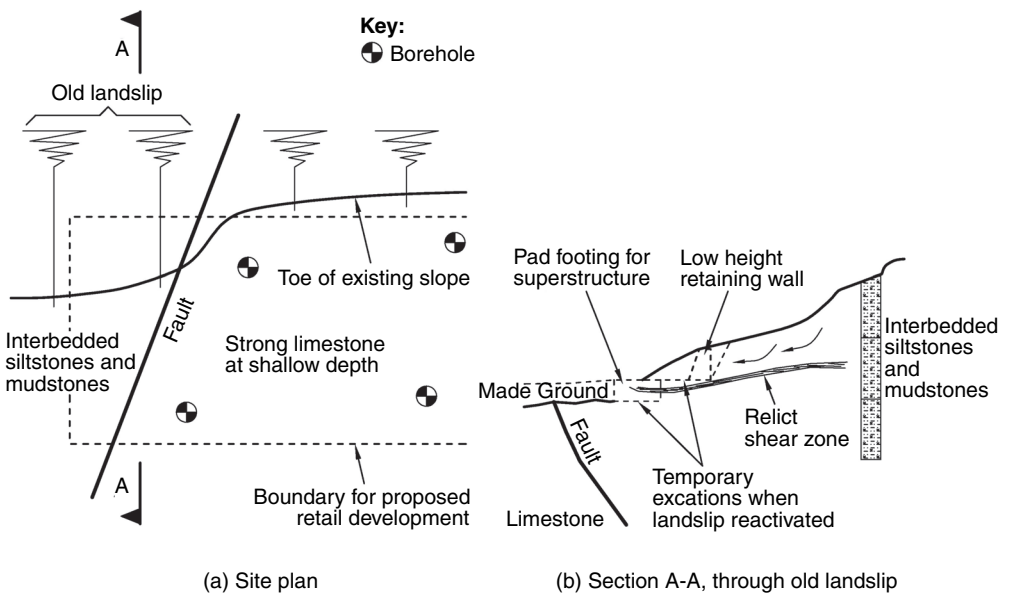


Figure 9.8 Shallow foundation adjacent to a landslip

This is a stark reminder that even simple foundation engineering needs to be done in a careful and systematic way. A desk study and an understanding of the site's geology is a fundamental requirement, and is essential in order to properly plan and design a ground investigation.

At this site, the first construction activity should have been to stabilise the landslip area. The landslip was stabilised by placing a temporary berm (at the landslip toe) and then installing a 'shear key' through the shear zone of the landslip. The shear key involved large diameter heavily reinforced bored piles, constructed as a contiguous embedded retaining wall. Following this the toe of the slope could be safely removed and excavations for the shallow foundations constructed.

9.4.3 Water treatment works, differential swelling

The design of a water treatment works required varying excavation and founding levels across the facility. The operation of the works meant that water levels in the various tanks would fluctuate over time. The construction of the works went well; however, just before the works opened, cracking of floor slabs was observed during commissioning tests. The cracks were monitored, and they became progressively worse over time.

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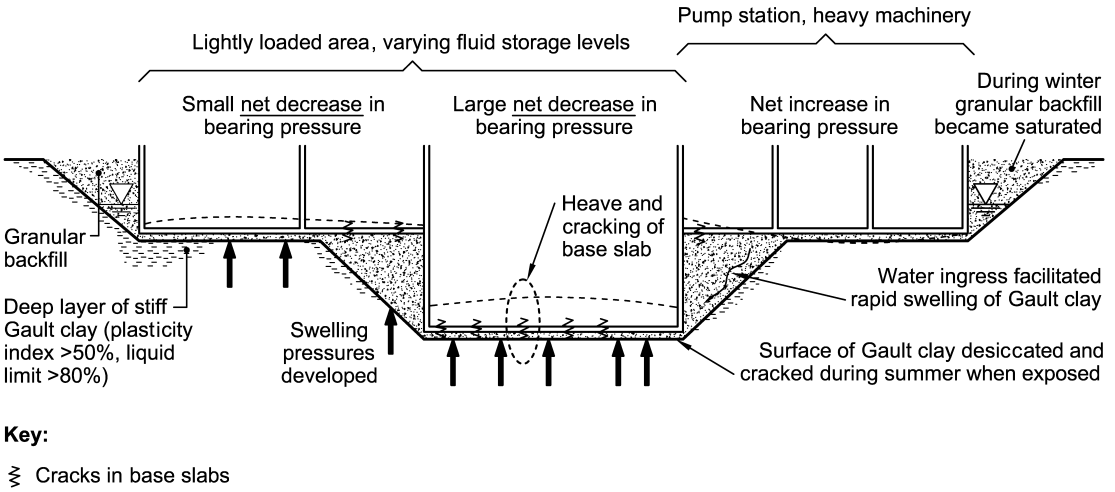


Figure 9.9 Water treatment works, differential swelling

At this stage, many of the areas which had suffered the worst damage were also relatively lightly loaded.

Figure 9.9 shows a cross-section through part of the works. The overall configuration of the sub-structure is quite complex, comprising an irregular array of reinforced concrete boxes of varying size. The site geology is quite simple, comprising a deep layer of heavily overconsolidated Gault clay. The site was in a semi-rural area and was a greenfield site. A simple open non-retained excavation was formed since there were no space constraints. The excavation was carried out during a relatively dry summer and there were no problems with ground or surface water. The concrete sub-structure 'boxes' were then built within the excavation and conventional granular backfill used (**Figure 9.9**).

When the cracking became apparent, survey points were fixed and monitored and these indicated differential vertical movements across the structure, and net upward, swelling, movement beneath the deepest part of the sub-structure. A standpipe piezometer in the backfill indicated about 1 m head of water. Subsequent analysis highlighted the following issues:

(i) Owing to the different founding levels, the different types of equipment, and water levels in certain parts of the works, the *net* change in bearing pressure is extremely variable, from small net *increases* in bearing pressure, to large net *decreases* in bearing pressure.

(ii) Because of the high plasticity index of the Gault clay and its high degree of overconsolidation, it is prone to significant long-term swelling, and if movement is restrained, large swelling pressures are developed against the structure in contact with the Gault clay.

(iii) The trigger for the swelling movements and structural damage was a readily available source of water. Over winter, rainwater seeped into the granular backfill, and along the (relatively poor and cracked) blinding concrete. During the summer period it is likely that desiccation cracking of the exposed Gault clay would have created a relatively high mass permeability, and allowed relatively quick infiltration of water into the clay.

(iv) The fundamental flaw, however, was the lack of awareness on the part of the designers that differential movements would develop, and the structural design was vulnerable to differential movement. The combination of relatively flexible slabs and stiff walls/columns meant that the sub-structure was particularly sensitive to differential movement.

(v) An important cause of the problem was poor communication between the geotechnical and structural engineers. The structural engineer asked the geotechnical engineer to provide 'allowable bearing pressures'. There was no discussion about the nature of the structure, or the variation in applied loads imposed on the foundations. As a result, the structural engineer did not realise there was a risk of swelling or differential movements across the structure. The geotechnical engineer did not realise there would be net unloading across large parts of the structure. This lack of communication led to the design of an intrinsically flawed foundation and structural design.

For this project the mechanical behaviour of the ground was not assessed *in the context* of the proposed foundation requirements. A more appropriate design would have created more uniform loading, or reduced the net unloading beneath the deepest part of the sub-structure, by reconfiguration of the plant layout, or if impractical, provided a structural design which would have minimised differential movement (e.g. a thicker, stronger raft) or allowed differential movement to occur without structural damage (provision of movement joints, rocker slabs, etc.).

9.4.4 Driven piling in mudstone, plugging of open-ended tubular piles

A highway bridge was to be constructed across a small river. The bridge and foundations were designed and let under a conventional ICE contract to a local civil engineering contractor. The foundations were bored pile groups constructed in the Coal Measures rocks, which comprised inter-bedded mudstone and sandstone. A high quality ground investigation was carried out; **Figure 9.10** summarises the ground conditions. The upper Coal Measures rocks were relatively poor quality, and predominantly comprised mudstones with an unconfined compressive strength of about 10 MN/m^2, and RQD values of between 0 and 10%, with frequent reference to open or clay-filled joints in the rock mass. The rock quality improved at depth, with more frequent layers of sandstone with unconfined compressive strengths of about 30 MN/m^2 and RQD values of between 30 and 60%. SPT

Sampling			Strata
Depth (m)	TCR	(RQD)	Description
10.40			
	75%	(0%)	Weak grey slightly carbonaceous SILTSTONE. Locally weathered to slightly sandy fine to coarse angular gravel. Discontinuities horizontal to subhorizontal very closely to closely spaced planar closed to slightly open (<3 mm) clean. At 9.80 m, clay smeared discontinuity. At 10.02 m, 10° clay smeared discontinuity. At 11.10 to 11.20 m, weathered to clayey fine to coarse angular gravel.
11.40			
	93%	(0%)	
12.90			
	93%	(0%)	At 13.10 to 13.70 m, dark grey and carbonaceous, tending to silty MUDSTONE. From 13.70 m, moderately weak to moderately strong sandy siltstone.
14.40			
	100%	(8%)	At 14.40 m and 14.50 m, open (10 mm) slightly sandy clay infilled discontinuities. At 15.27 to 15.30 m, open (30 mm) slightly clayey fine to medium angular gravel infilled discontinuity. At 15.30 to 16.45, subvertical irregular rough closed to slightly open (<3 mm) clean discontinuity.
15.90			
	84%	(40%)	At 16.20 to 16.70 m, dark grey and carbonaceous. At 16.55 m, open (10 mm) gravelly sand infilled discontinuities. At 16.70 to 16.75 m and 16.92 to 16.95 m, open slightly sandy fine to medium angular gravel infilled discontinuities. At 16.92 to 18.30 m, slightly sandy. From 17.00 m, moderately strong to strong. From 17.40 m, discontinuities very closely to medium spaced.
17.40			
	100% (93%)	(40%)	
18.90			At 18.50 to 18.80 m, subvertical irregular smooth closed clean discontiuity.
	100% (93%)	(35%)	

Equipment:	Borehole dia. (mm):	Casing dia. (mm):
Rotary coring, mist flush	92 to 25.40 m depth	120 to 6.00 m depth

Figure 9.10 Extract from borehole log, upper Coal Measures

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'N' values were typically in excess of 100; however, in these ground conditions the SPT was meaningless and of little value.

The contractor offered an alternative foundation design which comprised driven open-ended tubular piles. The driven piles were expected to be quicker to install, and provide equivalent overall performance (capacity and settlement) to the original foundation design. Pile driving proved to be difficult with some piles experiencing premature refusal during driving (probably due to local blocks of sandstone). The project specification required the contractor to carry out pile load tests on selected piles. These tests on working piles were meant to impose loads up to 1.5 times the working load with pile head settlement of less than 10 mm. Unexpectedly, at the working load the pile settlement was excessive and, for practical purposes, the pile had failed. Several other pile load tests were carried out and these also failed. A detailed review was carried out of both the pile installation and the pile design. A number of problems were apparent both in terms of pile construction and design. The pile design had assumed:

(i) The base of the tubular pile would 'plug', i.e. the end-bearing resistance would be mobilised over the full pile diameter. The end-bearing resistance was assumed equal to 4.5 times the unconfined compressive strength of the rock (i.e. equivalent to 9 times the undrained shear strength, as commonly assumed for clay).

(ii) The shaft resistance was based on a mobilised adhesion of twice the SPT 'N' value (i.e. in excess of 200 kN/m^2); the assumption that shaft shear resistance, $\tau = 2$ 'N' is commonly used when designing driven piles in sands and gravels.

Based on assumptions (i) and (ii) the geotechnical capacity of the pile was expected to be extremely high, and the factor of safety was expected to be well in excess of 3.0 at the working load. Unfortunately, as shown by the pile tests, the actual factor of safety was close to 1.0.

The above design assumptions were based on a published design guide for driven piles. However, the guidance and associated equations for calculating pile capacity were actually based on experience of driven piles in soils, not rocks. Hence, the design assumptions were inappropriate. A careful back analysis of the pile test data indicated:

(a) The open-ended tubular piles did not plug, i.e. end-bearing resistance was only mobilised across the annulus of the pile.

(b) Pile resistance was predominantly mobilised by shaft friction, both internally and externally; the average shaft adhesion varied between about 60 and 100 kN/m^2, with a median value of about 85 kN/m^2. The intense remoulding of the mudstone due to pile driving had effectively created a thin zone of very stiff clay along the sides of the steel pile.

The pile tubes had not been strengthened to facilitate hard driving (normally a tubular pile would have a specially designed thick-walled driving shoe, if hard driving was expected). The pile driving hammer was also of inadequate capacity. Hence, driving the piles deeper to the higher quality sandstone at depth was not a viable option. The most effective solution to these design and construction problems was therefore to drill out the materials inside the pile tubes and infill the void with concrete. This enabled the full end-bearing resistance to be mobilised.

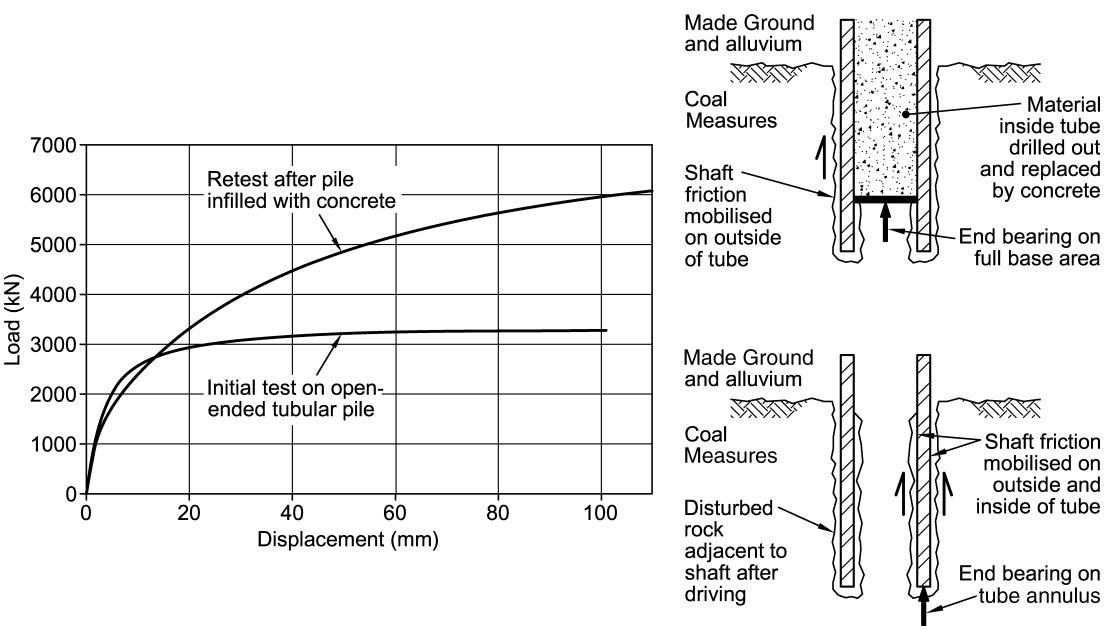


Figure 9.11 Driven tubular piles in mudstone, pile test data. Open-ended pile vs concrete-infilled pile

Subsequent pile load tests indicated that the modified piles had adequate capacity and settlement characteristics (**Figure 9.11**). Because of the fractured nature of the rock mass, the end-bearing resistance was between 1.5 and 2.5 times the unconfined compressive strength (i.e. 3 to 5 times S_u, considerably lower than soil mechanics theory would indicate).

This project experience clearly emphasises the need to understand the ground behaviour in the context of the proposed construction process (in this case the rock mass behaviour during pile driving, which is quite different from that of soil), and understand the limitations of empirically based design guidance. 'Well winnowed' experience was clearly *not* used either in terms of design or construction. When designing open-ended tubular piles, the end-bearing resistance of the pile annulus plus the internal shaft resistance should be compared against the bearing resistance across the full pile area, and the lesser of the two values should be used for design. In addition, the influence of the construction process, in this case the pile driving, on the mobilised shear resistance along the pile shaft was not appreciated. Estimating the capacity of driven tubular piles in weak rocks is actually quite challenging. Even if the design assumptions had been more appropriate, it would have been wise to carry out preliminary pile tests (to failure) on non-working piles, prior to the commencement of the main piling works. This would have significantly reduced the risk of unexpected pile failures, although the cost and duration of the piling works would have been increased.

9.4.5 The geotechnical triangle revisited

Reflecting upon the above case histories, the value of the geotechnical triangle can be appreciated. All aspects of the triangle should be considered and kept in balance, i.e. one element should not be given excessive attention whilst ignoring other aspects. There is a high risk of failures occurring if one or more aspects are forgotten.

Table 9.3 provides a commentary on the problems encountered in the above case histories in the context of the five aspects of the triangle. The errors and omissions which caused the foundation problems may seem obvious; however, all the engineers involved were experienced. In some cases, there was an assumption that because the engineering was perceived as 'simple and routine', short-cuts to normal good practice could be made without risk. Clearly, with the benefit of hindsight, the engineers involved would now have a different opinion!

9.5 Foundation applications

9.5.1 Introduction

In this section two case histories are given which describe the foundation engineering for two very different projects. The first describes the selection of an appropriate foundation type for a multi-span bridge across an important railway line. The site was underlain by estuarine sands of variable density. The second discusses groundwater problems during construction of foundations for a building, which was underlain by chalk. Common to both case histories are the important observations and tests which were carried out during the construction works (i.e. the fifth key component of the geotechnical triangle), and the critical role that these played in successful completion of the works.

9.5.2 Vibrocompaction of estuarine sands

9.5.2.1 Site location and ground conditions

The site is located in North Wales, adjacent to the River Dee estuary. The project involved the construction of an access road into a new business park, which was to be located between an existing main road and the estuary. A bridge was required to carry the new road over a railway line which traverses the centre of the site. This railway line is a strategically important route connecting the UK network to the Irish Sea ferries. The bridge comprised a two-span deck, each span being about 30 m long. To attain adequate clearance to the railway, the approach embankments were relatively high at about 12 m. This case history is described in detail by O'Brien (1997).

The site is located on a flat estuarine plain and the geological sequence is shown in **Figure 9.12**. The groundwater table level was about 2.5–3.0 m beneath ground level, and pore water pressures varied hydrostatically with depth. Of concern, for bridge foundation design, was the variability in relative density of the estuarine sand. Although typically described as loose to medium dense, there were also very loose and dense layers. SPT 'N' values varied between 5 and 35, but the repeatability of the test was poor due to the difficultly of maintaining borehole base stability beneath the water table. CPT q_c values tended to increase with depth, although there was significant variability in the CPT q_c profiles.

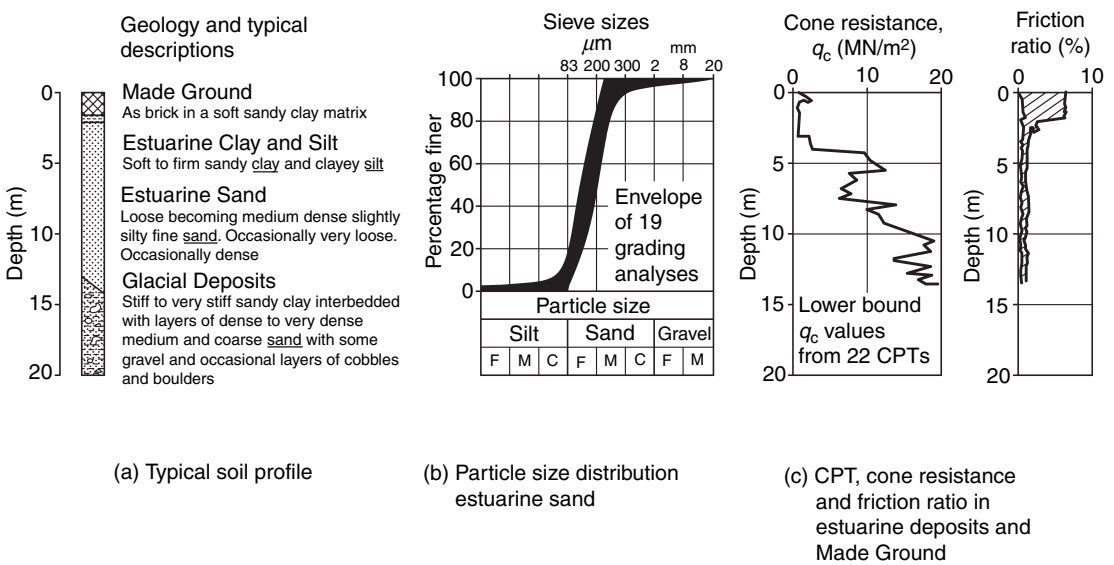
9.5.2.2 Foundation selection

Several different foundation types were considered. The net increase in bearing pressure was about 300 kN/m^2. The bearing capacity of the sands was adequate, and the main concern was ensuring settlement, particularly differential settlement, was sufficiently low. For the multi-span bridge a limit on differential settlement of about 15 mm was targeted. Founding the bridge on shallow foundations, beneath the clay/silt capping layer, would have been feasible if the approach embankments (which imposed the majority of the bearing pressure) could have been constructed before placing the bridge deck. Unfortunately the bridge deck beams had to be placed within a narrow timeframe, when a track possession (for construction over the railway line) had been granted. This possession period was fixed and critical to the whole project, and all other activities had to be as flexible as possible. If the deck was placed before construction of the approach embankments, then differential settlement was expected to be higher than the 15 mm allowable (perhaps double this value, due to the variable density of the estuarine sands). The main foundation options are

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Case history	Ground profile and groundwater conditions	Ground behaviour	Appropriate modelling	Empiricism, case records, well-winnowed experience	Construction process test/observations and verifications
Shallow foundations, adjacent to landslip	No desk study. Geology *not* understood. Poor positioning of boreholes, relative to fault.	Since geology not understood, no awareness of adjacent landslip.	Global stability of site not assessed, only local stability of retaining walls/ foundations considered.	Geological maps and memoirs highlighted the risk of landslips in this area (but not reviewed by designers).	Because hazard not identified, construction process was inappropriate. Should have stabilised the slope ***prior*** to excavations.
Water treatment works, differential swelling	Geology and ground profile understood. Deep layer of Gault clay.	Poor communication between geotechnical and structural engineers. Hence, net unloading of O/C plastic clay not identified.	Only bearing capacity and settlement checks for net increases in bearing pressure. Consequences of swelling not assessed.	The swelling potential of high-plasticity overconsolidated clays, such as Gault clay, is well known. But poor communication meant that potential risk for this structure was not identified.	Since design was flawed, construction process, tests, etc. would not have mitigated the risk.
Driven piling in mudstone, plugging of tubular piles	Good desk study and high quality ground investigation carried out.	Appropriate testing and observations; ground behaviour was well established. But not understood by designer/contractor in the context of proposed construction, i.e. driven piling.	Poor modelling. Key assumptions were incorrect. Open-ended tubular piles were unlikely to plug.	Inappropriate design methods used (not relevant for weak rock). Poor construction methods (piles not strengthened for hard driving, underpowered piling hammer), so unable to drive to competent strata.	Pile tests on working piles identified problem, but too late, so overall project was delayed. Tests on preliminary piles would (technically) be preferable but cost/ time penalty.

Table 9.3 Keeping the geotechnical triangle in balance: case history examples



(a) Typical soil profile

(b) Particle size distribution estuarine sand

(c) CPT, cone resistance and friction ratio in estuarine deposits and Made Ground

Figure 9.12 Vibrocompaction of estuarine sands, ground conditions

summarised in **Table 9.4**. Pile integrity and durability was a particular concern, since previously the site had been used as a chemical works, and the ground conditions were aggressive to reinforced concrete. At the time the works were implemented (mid-1990s), CFA piling rigs were much less powerful than today. Large diameter (> 1.0 m) CFA rigs were not available, and installing heavy pile reinforcement, especially to accurate verticality tolerances, was judged to be impractical. Heavy pile reinforcement would have been required because the bridge abutment and embankment backfill imposed significant lateral loads onto the piles. Because of the reinforced concrete durability concerns, it would have been important to ensure the reinforcement had adequate concrete cover; hence accurate reinforcement installation was necessary. Driven piles would probably have caused damaging vibrations and settlement to the adjacent railway. Large diameter bored piles (due to the need to use bentonite slurry support) were expected to be very expensive. Vibrocompaction was believed to be the most appropriate foundation engineering option compared with the other available options. Deep ground improvement, using vibrocompaction, would increase the relative density of the sands, and in particular, eliminate the zones of loose sand that existed. This would enable simple foundations to be constructed on the improved ground, and reduce the risk of differential settlement. Vibrocompaction is discussed in more detail in Chapter 59 *Design principles for ground improvement*. The vibrocompaction method used at this site used a 'wet top feed' technique, which involves using a vibroflot (a large vibrating poker) to compact the ground. Water is used as a jetting medium to facilitate insertion of the vibroflot, and gravel is fed into the hole around the vibroflot from the ground surface and compacted by the vibroflot.

The ground conditions (fine estuarine sands, with a low silt/clay content) were believed to be ideal for vibrocompaction. Discussions with local contractors were held, but locally vibrocompaction was generally only carried out for lightly loaded industrial buildings founded on Made Ground. Typically, the performance criteria were less onerous, with net bearing pressures being of the order of 100–150 kN/m^2. Nevertheless, a review of the technical literature and discussions with specialist contractors indicated that vibrocompaction would be feasible, although possibly challenging. The specification was developed carefully, in order to control the anticipated project risks:

(i) A preliminary trial was required, using different compaction spacings, and compaction effectiveness was checked by carrying out CPT tests before and after compaction (during this trial, remote from the railway line, ground vibration and settlement were also monitored).

(ii) The specification was an 'end product' type: a target CPT cone resistance profile had to be met by the specialist ground improvement contractor (**Figure 9.13**).

(iii) A limit on allowable ground vibration and settlement at the railway line (based on discussions with the railway authorities) was stated. It was recognised that this constraint would prevent the use of the most powerful rigs for vibrocompaction, but lower-powered rigs could be used, and would still be able to densify the sands.

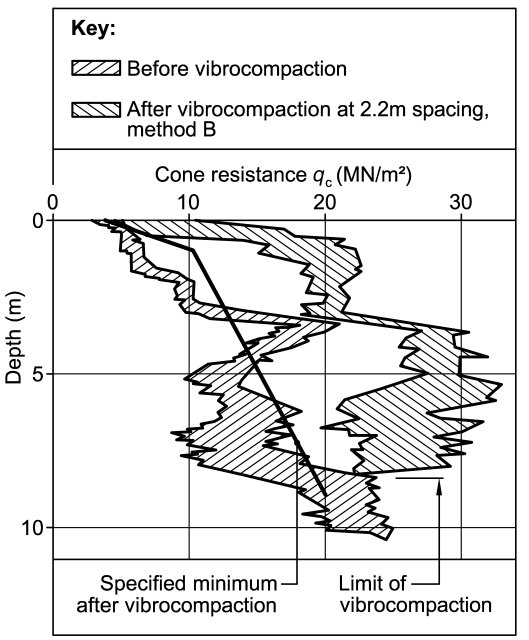


Figure 9.13 Vibrocompaction preliminary trial–influence of different compaction methods (method A vs method B)

Option	Potential advantages	Potential disadvantages
1. Driven piles	Suited to ground and groundwater conditions. Good vertical capacity.	Difficult driving due to dense sand layers. Large vibrations and damage to railway. Limited lateral/bending capacity unless raking.
2. CFA Piles	Low vibrations. Relatively inexpensive.	Inadequate lateral/bending capacity. Poor integrity/durability.
3. Large diameter bored piles	Good lateral/bending capacity. Low vibrations.	Need temporary casing and slurry support. Very expensive.
4. Vibrocompaction	Quick and inexpensive (less than 50% of 3). Suited to ground and groundwater conditions. Allows simple sub-structure.	Vibrations and possible damage to railway. Inadequate degree of improvement. Lack of precedent in UK for heavily loaded bridges.

Table 9.4 Case history: vibrocompaction of estuarine sands, advantages/disadvantages of different foundation options

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9.5.2.3 Observations and tests during the works

The successful tenderer used a vibroflot with a power rating of 50–80 kW, and medium/coarse gravel was used as backfill at each compaction point. The initial trial using a compaction spacing of 2.6 m was very disappointing (refer to **Figure 9.13** (method A)), since the post-compaction cone resistance profile failed to meet the specification requirement. There were a range of possible reasons discussed by the designer and contractor (ground conditions different from expected? specified target cone resistance profile reasonable?), although the discussions quickly focused on how the vibroflot was being used by the rig operator. Method A involved: withdrawal of the vibroflot in 0.6–0.9 m depth increments, compaction duration per increment of 20–30 seconds, high water flow rates from side jets during withdrawal. This method had been used successfully to treat Made Ground, when the improvement mechanism was mainly 'reinforcement' by construction of relatively large diameter stone columns. In contrast, the prime requirement for this project was to densify the sands. An alternative construction method, 'method B', was proposed, which involved: withdrawal in 0.3 m depth increments, compaction continuing for a longer period until the power consumption peaked, and water flow during withdrawal kept to a minimum (to avoid washing out excessive quantities of sand). From **Figure 9.13** it can be seen that post-compaction CPT q_c tests showed that method B was far more effective than method A in terms of compacting the sands. Method B was used for the remainder of the trial (to check the influence of different compaction spacings on CPT cone resistance) and for the main works.

The measured ground vibrations were towards the lower bound of those reported in the literature (a resultant peak particle velocity of less than 15 mm/s) and ground surface settlements between 3 mm and 7 mm. Following completion of the trial the railway authorities were satisfied that vibrocompaction could take place up to 2 m from the railway.

For the main works a compaction spacing of 2.2 m was selected since at this spacing there was a negligible risk of the post-compaction CPT q_c values falling below the target profile. The plan area of compaction extended about 3 m beyond the bridge pad footings, with a minimum of one compaction point beyond the footing. The pad foundations were founded at about 2.5 m below ground level, to ensure the foundations were below a surface layer of silt and clay, and above the groundwater table level. Pre- and post-compaction CPT tests were carried out to verify that the vibrocompaction was adequate. CPT profiles for one of the bridge abutments are shown in **Figure 9.14**. The post-compaction CPT q_c values exceed the target profile by a comfortable margin, and are typically about two to three times higher than the pre-compaction values across the main depth of interest. The subsequent performance of the bridge foundations has been satisfactory. Based on the post-compaction CPT profiles, routine calculations indicated a settlement of about 15 mm, whilst more sophisticated analyses indicated a settlement of about 10 mm. Measured settlements

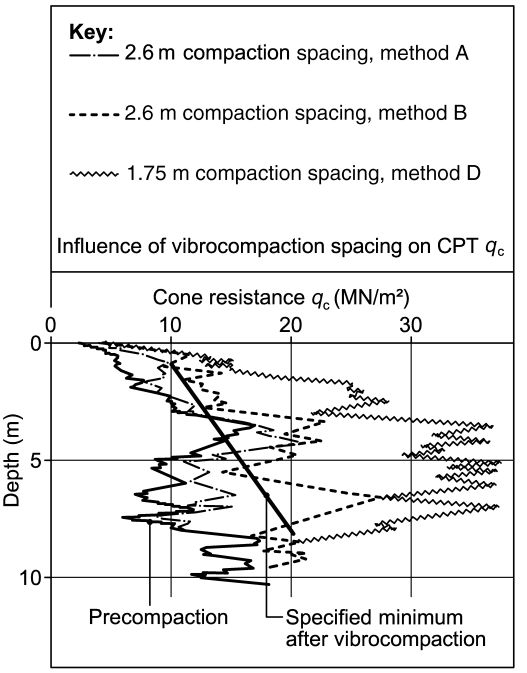


Figure 9.14 Vibrocompaction, before and after CPT tests, 2.2 m spacing, method B

were between about 5 and 10 mm, with differential settlements of less than 5 mm. These were comfortably within the allowable limits for the bridge deck.

9.5.2.4 Concluding remarks

This case history describes some of the factors which need to be considered when deciding upon an appropriate type of foundation. The foundation type, deep ground improvement with shallow foundations, performed extremely well. However, successful use of the technique drew heavily on a balanced application of all five components of the geotechnical triangle. In particular, the fifth component, 'understanding the construction process and verifying the design assumptions', was important. The initial vibrocompaction method (method A) was inappropriate; observations and tests during a preliminary trial identified the problem and enabled a more effective construction method to be identified. This was critical for the successful completion of these works.

9.5.3 Shallow foundation on soft chalk

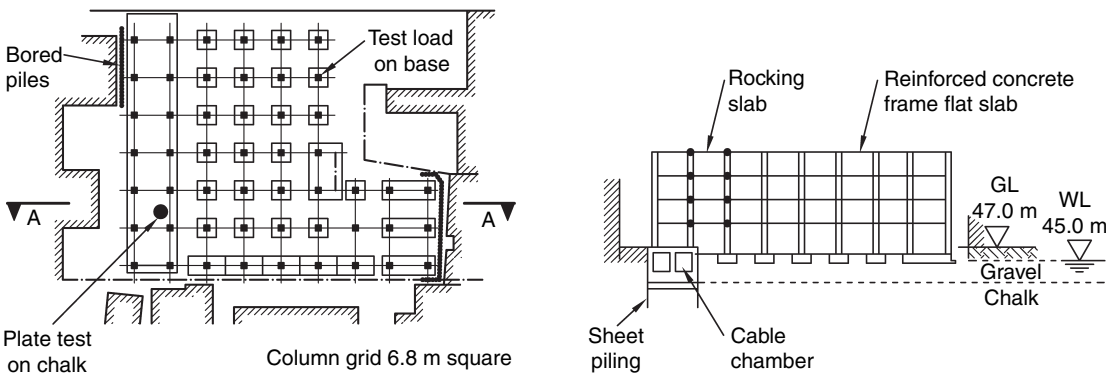
9.5.3.1 The ground conditions and construction problems

The construction of a telephone exchange in the centre of Salisbury came to an abrupt halt when unexpected ground conditions were encountered. Work was not to resume for some six months. The geological map shows the site to be covered by valley gravel overlying Upper Chalk. Four boreholes were sunk on the site using a shell and auger rig. The borehole logs showed the profile to consist of 0.9–1.4 m of Made Ground overlying 4.9–6.4 m of medium dense flint gravel with layers

of silty clay. All the boreholes were reported as entering hard fissured chalk below the flint gravel. The water table was about 2 m below ground level. This case history is discussed in detail by Burland *et al*. (1983).

The telephone exchange is a four-storey building of reinforced concrete flat slab construction. **Figure 9.15** shows a plan and cross-section through the building. Most of the structure to the east is founded on pad footings bearing on gravel with a pressure of about 200 kN/m^2. A notable feature is a chamber running along the western edge of the building. The base of the cable chamber was to be founded in chalk some 6.6 m below original ground surface at a gross bearing pressure of approximately 185 kN/m^2. The cable chamber was constructed in a coffer-dam 9 m wide and 47 m long. The coffer-dam was constructed mainly of sheet piles but in the north-west corner a contiguous bored pile retaining wall was used to minimise vibration alongside an existing building. **Figure 9.16** is a cross-section through the coffer-dam.

Work progressed smoothly until excavation within the coffer-dam penetrated the chalk. Thereafter progress was difficult. A close visual examination of the chalk in the base of the excavation revealed that, far from being hard and fissured, it was very soft, consisting of small angular pieces of chalk in a silt-like putty chalk matrix. At this stage two-thirds of the pad foundations had been completed. The coffer-dam area had been partially excavated. At the south end in the base of the excavation



(a) Site plan

(b) Section A-A

Figure 9.15 Shallow foundations on soft chalk; plan and cross-section through building

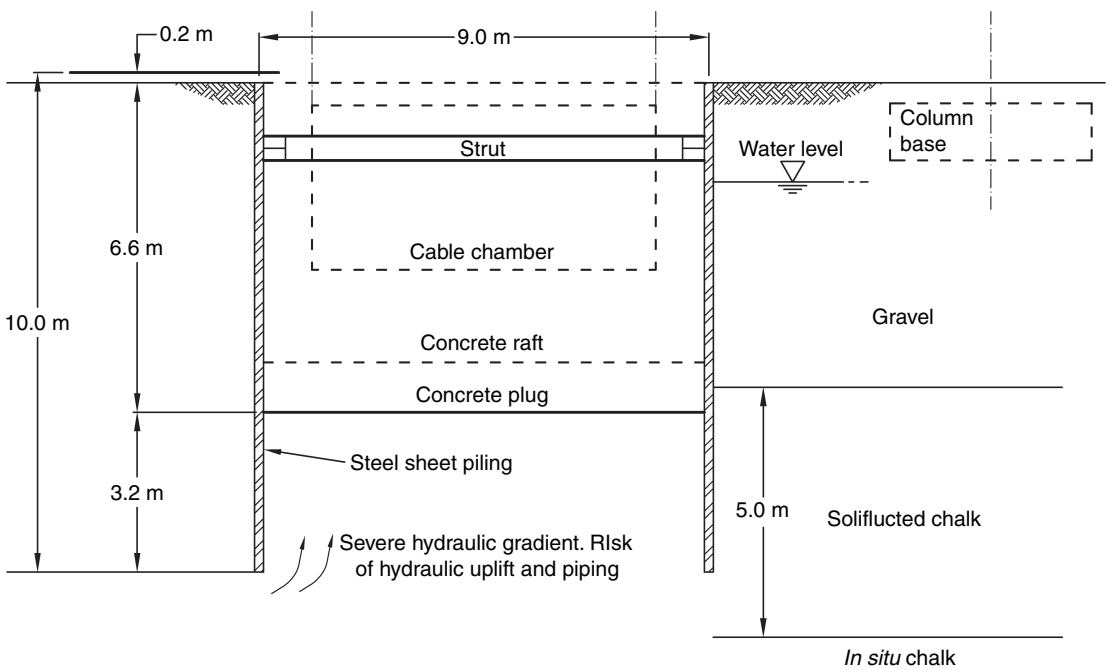


Figure 9.16 Shallow foundations on soft chalk; cross-section through coffer-dam

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the chalk had turned into a slurry, making it impossible to obtain a firm bottom. The coffer-dam was being kept partly free of water by pumping from two sumps, and a considerable quantity of water was flowing along the bottom of the coffer-dam.

Since the ground conditions within the excavation had proved to be so different from those assumed, the whole basis of the foundation design had to be questioned. In particular the use of pad footings and a raft foundation beneath the cable chamber were thrown into doubt. Before reaching any conclusions it was felt that a further site investigation should be carried out. The soil strata, as they affect the foundations, may be divided into three categories:

Medium dense sandy gravel – soil type G

This flint gravel contains very sandy pockets and also pockets or lenses of a more clayey nature. Near the bottom of the stratum there is an increasing proportion of cobble-size flints. At the base of the stratum is a layer of about 0.3 m thickness of gravel and sand-sized rounded particles of chalk.

Soliflucted chalk – soil type Ch 1

This stratum consists of fragments of hard white chalk in a soft matrix of remoulded chalk with flints. The matrix is mostly silt size but the materials exhibit cohesion and plasticity. Samples of the material broke open easily and were similar to soft or firm clayey silts in appearance and feel. During drilling the chalk was found to flow up into the borehole and the casing sank under its own weight. It appears therefore that the material is very sensitive and loses much of its strength when it is remoulded without drainage. The SPT values for this material are very low, ranging from 1 to 10 with an average of about 7.

In situ chalk – soil type Ch 2

It proved difficult to accurately define the changeover between Ch 1 and Ch 2 chalk by examination of samples. There is, however, a noticeable increase in the SPT values when moving from Ch 1 into Ch 2 material. Moreover, on reaching the *in situ* chalk, the borehole casing stopped sinking. As a rough guide a level of about 35 m OD was taken as the change from Ch1 to Ch 2.

The soliflucted Ch 1 material had probably been redeposited, and it was felt that it would behave like a silt. Four 100-mm-diameter U100 samples of soliflucted chalk were selected for consolidated drained triaxial testing. The results of the tests gave $\Phi' = 40°$ and $c' = 0$. Values of m_v, measured during oedometer tests ranged from 0.04 to 0.11 m^2/MN. The measured permeability was about 4×10^{-8} m/s, which is consistent with a silt. Pumping tests in the underlying Ch 2 chalk gave a permeability of about 2×10^{-5} m/s, reflecting the fissured nature of the chalk. Hence, the natural chalk has a permeability which is about a thousand times higher than the overlying soliflucted chalk.

A simple calculation of vertical equilibrium of the chalk in the base of the coffer-dam (during the excavation stage) showed that the total vertical stress due to the chalk (at the sheet pile toe level) was about 69 kN/m^2, whereas the groundwater pressure at that level was 76 kN/m^2. There was therefore a high risk of failure by hydraulic uplift and piping in the base of the coffer-dam.

9.5.3.2 The pad footings

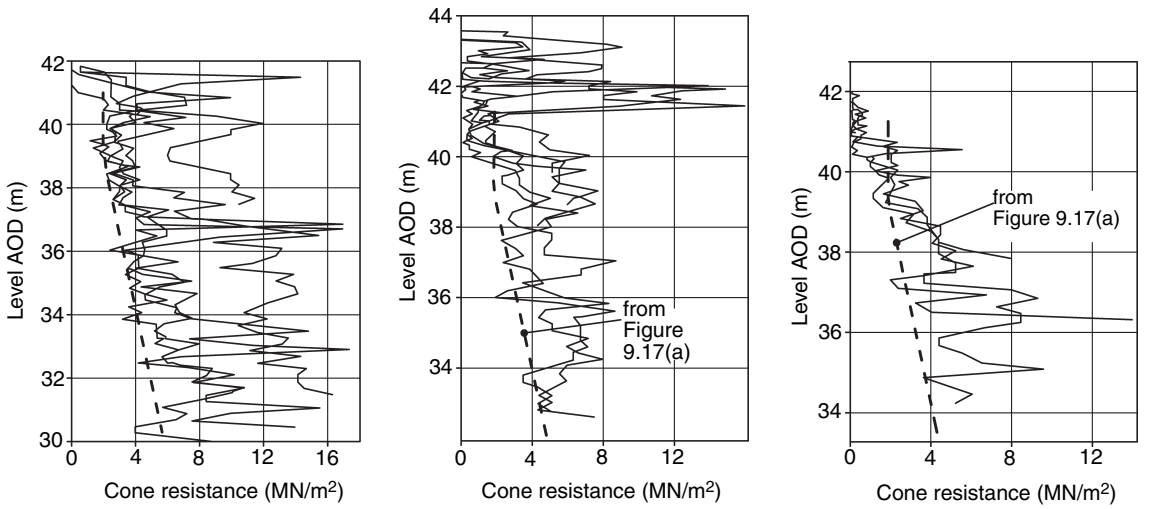
The first problem to deal with was the adequacy of the existing pad footings. Based on the ground investigation data for the soliflucted chalk, preliminary calculations indicated settlements of between 75 and 175 mm, which were deemed to be too large. An expensive solution was removal of the pads and installation of piles. Before taking this drastic action, an alternative means of assessing the pad footing settlement was required. Laboratory testing was believed to be unrealistic due to disturbance when sampling the chalk. *In situ* plate load tests would have been difficult below the water table. The simple and direct approach which was chosen was to load two of the existing pad footings up to the working load of 4000 kN (i.e. 400 tonnes) and measure the settlement. Although expensive, it was relatively quick, and all uncertainties (in terms of interpretation of alternative indirect methods) were eliminated.

The footings are 4.2 m square, 0.9 m thick, and founded at about 44.5 m OD. In order to measure soil deformation below the footing, holes were drilled through the pad: one hole to the gravel–chalk interface (about 3.5 m below founding level) and the other extended 8 m into the chalk. The holes were sleeved and rods embedded in mortar at the base of the hole. Precision levelling was used to measure the settlement of the rods and the four corners of the pad. The footing was loaded in 25 tonne increments up to 200 tonnes, and left for 42 hours. It was then loaded to 400 tonnes and left for 70 hours. Settlement was measured frequently throughout the test. Under full load the settlement was only 2.5 mm, with about one-third of this in the chalk. The time-dependent settlement was negligible. Because of these very small settlements, a second test was unnecessary. The full-scale test demonstrated unambiguously that the pad footings were more than adequate.

9.5.3.3 The cable chamber

In view of the very soft nature of the chalk it was felt that the stability of the sheet pile walls was marginal. Pumping was therefore stopped and the coffer-dam was allowed to flood. The next task was to investigate the chalk at the base of the coffer-dam and assess its degree of disturbance. It was felt that a simple method of doing this was to use a cone penetrometer. A number of tests carried out within the coffer-dam and outside it should provide a means of assessing the variability of conditions within the coffer-dam and the degree of disturbance in relation to the surrounding undisturbed chalk.

Eleven tests were carried out alongside the existing pad footings. The tests were made from the bottom of new boreholes drilled by shell and auger methods through the gravel and lined with rigid 74-mm-diameter UPVC pipe. The results of five



(a) Results of cone penetration tests alongside existing pad footings

(b) Results of cone penetration tests at north end of coffer-dam

(c) Results of cone penetration tests at south end of coffer-dam

Figure 9.17 Shallow foundations on soft chalk; cone penetration tests

typical tests are given in **Figure 9.17(a)**. As is to be expected the results show considerable scatter but there is a well-defined lower limit as shown by the broken line. Below a level of 39 m OD the penetration resistance increases steadily with depth.

Seventeen tests were carried out in the base of the coffer-dam from a rigid working platform spanning the excavation. At the north end of the coffer-dam the depth of excavation was approximately 3.5 m below original ground level and was still in the gravel. The results of five cone penetration tests are shown in **Figure 9.17(b)**. The gravel extends to approximately 41.5 m OD, which is confirmed by an adjacent borehole log. For comparison the broken line is taken from **Figure 9.17(a)**. It can be seen that the cone resistances lie well below the broken line down to about 40 m OD, below which there is reasonable agreement.

At the southern end of the coffer-dam, excavation was approximately 5 m below original ground level. **Figure 9.17(c)** shows the results of three cone penetration tests, and it can be seen that very low penetration resistances were encountered down to approximately 40.5 m OD. Below a level of about 38 m OD the results are in good agreement with those given in **Figure 9.17(a)**. The results of the cone penetration tests point strongly to the fact that the soliflucted chalk in the base of the coffer-dam had become loosened and disturbed by the upward flow of water causing piping. On the basis of the cone test results it was felt that it would be unwise to found the raft for the cable chamber above a level of 40.4 m OD.

Before reaching a final decision to found the cable chamber on a raft it was necessary to make an estimate of the settlement. A simple large-scale *in situ* loading test was devised. The aim of the test was to reproduce as realistically as possible the actual construction conditions. The test was sited at the southern end of the coffer-dam where the cone resistances were least. A 2 m square pit was excavated underwater to a level of 40.4 m OD (i.e. approximately 1.5 m below the existing bottom). A diver was used to guide and control the excavation. A 1.5-m-diameter concrete cylinder was then constructed and loaded by kentledge. There were a number of practical problems with carrying out this test; in particular, the cylinder was not vertical and tilted during loading. Nevertheless, load settlement measurements were made up to a load of 51 tonnes, and the test data showed two important features:

(i) up to working load, settlement was approximately proportional to load;

(ii) time-dependent settlement under constant load was very small.

Back analysis of the test, using linear elastic theory, allowed the settlement of the proposed cable chamber raft to be predicted; the results of these predictions are shown in **Table 9.5** for various construction stages. Construction up to ground floor level was to be carried out with groundwater pumping from wells in the natural (Ch 2) chalk. Dewatering would then be stopped leading to a decrease in *net* bearing pressure (and during the net decrease and subsequent increase in pressure, settlement was assumed to be zero until the previous maximum net pressure was exceeded). The total predicted measurable settlement is 62 mm. About half of this takes place during construction up to ground floor level. During superstructure construction the calculated settlement is 26 mm, with a further 10 mm due to live load. The large-scale test indicated that most of the settlement would be immediate. Hence, most of the settlement during superstructure construction would be 'built out'. Provision

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Construction stage	Bearing pressure (kN/m²)		Foundation settlement (mm)	
	Incremental	Net increase	Incremental	Total
Concrete plug	22	22	11*	–
Construction up to ground floor	52	74	26	26
Cease dewatering	–40	34	0	26
Construction to roof level	60	94	10	36
Internal walls and finishes	32	126	16	52
Live load	20	146	10	62

Note: * Not measurable

Table 9.5 Case history: Shallow foundations on soft chalk. Cable chamber, predicted settlements

was made for residual differential settlement between the cable chamber raft foundation and adjacent pad footings by incorporating 'rocker' slabs. These were omitted until *after* the superstructure was completed.

9.5.3.4 Cable chamber construction and foundation settlement

Concreting of the concrete plug in the bottom of the excavation was carried out under water in a series of transverse bays so as not to impair the stability of the coffer-dam. Before the concrete was placed, a layer of filter fabric was placed on the chalk type Ch 1 and covered with a layer, 100 mm thick, of coarse granular fill to form a drainage layer. Three 450-mm-diameter sleeves were cast in the plug through which sleeved wells were installed to a depth of 8 m below the excavation level and into chalk Ch 2. In addition two 75-mm-diameter pressure relief wells were installed through the plug in each bay and down into the chalk Ch 2. Pumping from the 8-m-deep wells using a 150-mm-diameter pump in each well lowered the water level to the level of the drainage layer beneath the concrete plug. Pumping continued until the raft and basement walls were complete, when the structure had sufficient weight to overcome buoyancy. Concrete for the complete raft was placed in a day.

The maximum settlement recorded for any of the pad footings was 5 mm and the average settlement was 2–3 mm. The time-dependent settlement was negligible. These results are in excellent agreement with the results of the full-scale loading test on one of the pad footings. The maximum settlement of the cable chamber near the south end was 15 mm, while near the north end it was 8 mm. Most of the settlement took place during the construction of the raft. Thereafter the settlements were small and very uniform, ranging from 3 to 5 mm.

9.5.3.5 Concluding remarks

It is evident from the measured settlements that the pad and raft foundations adopted for the telephone exchange were entirely satisfactory. Yet the decision to continue with these foundations was not an easy one. The top 5 m of chalk was very soft and all the available evidence pointed towards large settlements. It is very doubtful whether a traditional site investigation with sampling and laboratory testing would have resolved the issue because of the difficulty of obtaining representative undisturbed samples. In this case a direct approach to the problem was adopted by carrying out large-scale loading tests. The full-scale loading test on the pad footing was carried out quickly and simply and provided conclusive evidence of the adequacy of the pad foundations. The loading test in the coffer-dam was less satisfactory in that the results led to a significant overestimate of the settlements (62 mm predicted and 8–15 mm measured). Nevertheless, the large-scale test played a very important part in arriving at the final foundation solution.

This case history highlights the problems that can occur during construction, especially when the groundwater regime and groundwater effects during construction are not understood. Careful observations and tests during the works were of vital importance, in order to successfully complete the works. Construction of the cable chamber below the water table required a proper dewatering and depressurisation system to be installed. This was only switched off once the weight of the permanent works was sufficient to avoid buoyancy.

9.6 Conclusions

- A useful mnemonic for foundation design is the 'five S's': site, soil, structure, safety and sustainability.
- The ground profile and the allowable foundation movement are two key factors in selecting the most appropriate foundation type.
- Common factors in properly understanding ground risks are:
  - **(i)** understanding the site history and geology;
  - **(ii)** understanding the site's groundwater regime and how it impacts upon the proposed construction and permanent works.

In the vast majority of cases when foundation problems occur it is because either (i) or (ii) above has not been properly understood by the foundation designer. A coherent approach to proper understanding of ground risk is provided by the geotechnical triangle (introduced in Chapter 4 *The geotechnical triangle*).

- Often the limits set for allowable total and differential settlement are arbitrary and overconservative. It is important for the foundation designer to have early discussions with the structural engineer on this matter. Note that it is the *differential* settlement which occurs *after* the structure is built and finishes applied that has the potential to cause damage. The main opportunity for economic foundation design is to set realistic limits on allowable foundation movement, especially differential movements.
- The foundation engineering associated with a project can involve the assemblage of a number of components and a variety of processes. It is important that a holistic approach be adopted. When different components of a project are designed by different teams the potential interactions should be considered by a single team – preferably the lead project designer.
- Examples are given in the chapter illustrating the importance of keeping the geotechnical triangle 'in balance'. In addition to the four activities included in the triangle, it has been found helpful to consider a fifth aspect which involves *construction processes and design verification*. This aspect recognises the fact that foundation engineering is essentially a process requiring monitoring and verification of key design assumptions.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

Chapter 10

Codes and standards and their relevance

Trevor Orr Trinity College, Dublin, Republic of Ireland

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Geotechnical codes and standards represent standards of good practice. They serve as a means to achieve the safety levels required by society for geotechnical structures and as a means of communication, providing a common understanding between all those involved in a geotechnical project. Geotechnical codes and standards have moved from being prepared by committees of the engineering institutions to being prepared by technical committees of the European Committee for Standardisation (CEN, an acronym for Comité Européen de Normalisation) and from being based on the permissible stress method with overall factors of safety, to being based on the limit state design method with partial factors. The main components of a geotechnical design are the geometric data, the loads and geotechnical parameters, the verification method and the complex nature of geotechnical design situations. These design components are presented in the form of a geotechnical design triangle, the components of which are related to the aspects of the geotechnical triangle.

10.1 Introduction

As society grows more complex, with additional laws and regulations, and as people become more litigious, those involved in geotechnical engineering need to be confident that their designs are reliable and have the level of safety required by society. Reliable and safe geotechnical designs may be achieved through possessing a good understanding of soil behaviour, through adopting appropriate models for geotechnical design situations and, in view of the complex nature of soil, through taking appropriate account of previous comparable experience. Design codes of practice and standards aim to provide recommendations for good design practice, setting out the processes and procedures to achieve geotechnical designs that meet the requirements of society with regard to health and safety. In addition, they aim to provide designs that meet the requirements of the owners and users with regard to serviceability, i.e. with regard to acceptable settlements and deformations, and with regard to economy. As Simpson *et al*. (2009) noted, codes have a responsibility to balance economy with safety and serviceability in a world in which limitations of resources are increasingly recognised.

10.2 Statutory framework, objectives and status of codes and standards

The Building Regulations 2010, published by the UK Department for Communities and Local Government and made under the powers of the Building Act 1984, provide the statutory framework for buildings projects by setting out the functional requirements to ensure the health and safety of people in and around all types of buildings, as well as the requirements for energy conservation and the welfare and convenience of disabled persons. Practical guidance on ways to comply with the functional requirements of the Building Regulations is contained in Approved Documents, which are published in a number of parts by the National Building Specification (NBS) for the Department for Communities and Local Government. For example, Approved Document A (NBS 2006), which is concerned with structural safety, refers to two geotechnical British Standards, both published by the British Standards Institution, London (as are all the codes and standards referred to in this chapter and listed in the tables, except for the first codes of practice, CP 1, CP 2 and CP 4, as noted in section 10.4): BS 8002 *Code of Practice for Earth Retaining Structures* and BS 8004 *Code of Practice for Foundations*. These provide practical guidance for meeting the structural safety requirements of the Building Regulations. However, since the Eurocodes superseded these standards on 31 March 2010, prior to Approved Document A being revised, reference should now be made to BS EN 1997-1:2007 rather than to BS 8002 and BS 8004 as providing practical guidance on meeting the structural safety requirements of the Building Regulations. The prefix EN indicates that it is a European norm or standard.

It should be noted that the guidance given in the Approved Documents is not a set of statutory requirements and does not have to be followed. Compliance with the codes and standards referred to in the Approved Documents is just one way of fulfilling (i.e. being deemed to satisfy) the functional requirements in the Building Regulations. Hence, the referenced codes and

standards are not mandatory and alternative methods may be used, although the designer may need to prove that the alternative methods satisfy all the relevant functional requirements and that the resulting design still complies with the Building Regulations. There is a legal presumption that if a designer has followed the guidance in an Approved Document, this is evidence that the design has complied with the Building Regulations. The foreword to BS 5930:1981 *Code of Practice for Site Investigations* has the statement that '*compliance with this code of practice does not confer immunity from the relevant statutory and legal requirements*' and this statement is valid in the case of all the codes of practice referred to in the Approved Documents.

While the existing British codes of practice have been published with the objective (as noted in BS 5930:1981) to represent standards of good practice and have taken the form of recommendations, the objectives of the Eurocodes – including Eurocode 7 for geotechnical design (BS EN 1997-1:2007) – are broader. It is stated in the forewords to the Eurocodes that they are to serve as reference documents for the following purposes:

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – mechanical resistance and stability – and Essential Requirement N°2 – safety in case of fire;
- as a basis for specifying contracts for construction works and related engineering services;
- as a framework for drawing up harmonised technical specifications for construction.

With regard to those who use codes and standards in geotechnical engineering, Eurocode 7 makes the following important assumptions, which should not be overlooked:

(i) data required for design are collected, recorded and interpreted by appropriately qualified personnel;

(ii) structures are designed by appropriately qualified and experienced personnel;

(iii) adequate continuity and communication exist between the personnel involved in data collection, design and construction;

(iv) adequate supervision and quality control are provided in factories, in plants, and on site;

(v) execution is carried out according to the relevant standards and specifications by personnel having the appropriate skill and experience;

(vi) construction materials and products are used as specified in Eurocode 7, or in the relevant material or product specifications;

(vii) the structure will be adequately maintained to ensure its safety and serviceability for the designed service life;

(viii) the structure will be used for the purpose defined for the design.

Ensuring compliance with all of these assumptions is more important for ensuring the safety of geotechnical structures and reducing the risk of failure than is precision in the design calculations, or the values adopted for the safety factors. Regarding compliance with (i), (ii) and (v) above, Eurocode 7 does not define what is meant by people having appropriate qualifications, skill and experience. This will depend on the nature of the project and the ground conditions and has been left to national determination. A formal measure to address this issue has been undertaken by the Institution of Civil Engineers together with the Geological Society and the Institute of Materials, Minerals and Mining with the establishment in June 2011 of the UK Register of Ground Engineering Professionals (UK RoGEP). Details about the Register and how to become a registered ground engineering professional are given in document 3009(4) (ICE, 2011) prepared by the British Geotechnical Association under the auspices of the Ground Forum. The UK RoGEP provides external stakeholders, including clients and other professionals, with a means to identify individuals who are suitably qualified and competent in ground engineering.

10.3 Benefits of codes and standards

According to the British Standards Institution website (www.bsigroup.com), standards help to make life simpler and increase the reliability and effectiveness of the goods and services we use. Hence, in the case of geotechnical design, their benefits are that they establish the procedures and requirements, including the safety elements, for the design process to achieve the levels of safety, serviceability and economy required by society. Codes and standards are created by those involved in the industry and hence they bring together important collective understanding and relevant experience. Another benefit is that they provide a common reference and means of communication for all those involved in geotechnical projects: geotechnical designers, ground investigators, those working in geotechnical laboratories, structural engineers, contractors, clients, users, public authorities and regulators. Simpson *et al.* (2009) have elaborated on the communication aspect of codes and how they provide (i) a link between analysis and design, where analysis is a tool in the design process; and (ii) a link between the activities of the designer and the requirements of society regarding safety and serviceability.

In the case of the Eurocodes, the European Commission has published a document called Guidance Paper L (EC, 2003) in which the intended benefits and opportunities of the Eurocodes are listed as being to:

- provide common design criteria and methods to fulfil the specified requirements for mechanical resistance, stability and resistance to fire, including aspects of durability and economy;
- provide a common understanding regarding the design of structures between owners, operators and users, designers, contractors and manufacturers of construction products;
- facilitate the exchange of construction services between member states;

- facilitate the marketing and use of structural components and kits in member states;
- facilitate the marketing and use of materials and constituent products, the properties of which enter into design calculations, in member states;
- be a common basis for research and development in the construction sector;
- allow the preparation of common design aids and software;
- increase the competitiveness of the European civil engineering firms, contractors, designers and product manufacturers in their world-wide activities.

10.4 Development of codes and standards for geotechnical engineering

Codes of practice and standards are essentially two different types of document. A code of practice (CP) is defined by the British Standards Institution (BSI) in BS 0–2:2005: *A standard for standards* as a document which provides:

> recommendations for accepted good practice as followed by competent and conscientious practitioners, and which brings together the results of practical experience and acquired knowledge for ease of access and use of the information

while a standard is defined in BS 0–1:2005 as:

> a document, established by consensus and approved by a recognized body, that provides, for common and repeated use, rules, guidelines or characteristics for activities or their results, aimed at the achievement of the optimum degree of order in a given context.

Thus a code of practice is used for design as it aims to reach an acceptable technical level of quality and is expressed as functional requirements based on scientific/technical principles. It normally avoids standardising particular design methods or detailed construction procedures, leaving these choices to the designer (Krebs Ovesen and Orr, 1991). A standard, on the other hand, aims to achieve a certain degree of order in a given context through specifying particular methods or procedures to be used.

Examples of codes of practice in geotechnical engineering, as defined above, are the former BS codes for site investigations, earth-retaining structures and foundations, while examples of standards are the BS soil test standards which provide the specifications, i.e. the requirements and procedures, for the different tests for soils. Eurocode 7 is, by definition, a code of practice, since it provides recommendations for good practice as followed by competent practitioners. However, it is also a standard (according to the above definition) since it provides agreed rules aimed to achieve the optimum degree of order in geotechnical design expressed as a particular level of safety.

In recent years, since BSI has been publishing different types of document, including specifications, methods, guides, vocabularies and codes of practice, all with the same status and authority, it was decided to publish them all as standards. Hence, since the 1980s, BSI has published codes of practice as standards with the prefix BS for British Standard, although the term ‘code of practice’ has usually been retained in the title of the standard.

The first codes and standards for geotechnical engineering in the UK were prepared in the 1940s and 1950s by committees convened by the engineering institutions on behalf of the Codes and Practice Committee for Civil Engineering, Public Works, Building and Construction Work, under the aegis of the former Ministry of Works, for publication in the Civil Engineering and Public Works Series. These design codes of practice were based on the permissible-stress method with overall factors of safety. More details about the evolution of British codes of practice and standards have been provided by McWilliam (2002). He has reported on the issues that arose during the development of the codes of practice in relation to the traditional role of professional engineers, who regarded design rules as the individual choice for the engineer, and their developing relationships with government agencies and international trade.

Work on *Code of Practice no. 1 (CP 1): Site investigations* started in 1943 and was published by the Institution of Civil Engineers in 1950; *Code of Practice no. 2 (CP 2): Earth retaining structures* was published by the Institution of Structural Engineers in 1951; and *Code of Practice no. 4 (CP 4): Foundations* was published by the Ministry of Works in the late 1940s. The first standard for the testing of soils was published by BSI in 1948 as BS 1377: *Method of test for soil description and compaction*, which consisted of just one part when first published, but has since increased to nine parts. In 1949 the responsibility for the preparation and issue of the codes of practice of the Civil Engineering and Public Works Series in the UK was handed over to the following four professional engineering institutions: the Institution of Civil Engineers, the Institution of Municipal Engineers, the Institution of Water Engineers and the Institution of Structural Engineers.

In 1954, while the drafting of codes of practice remained with the professional engineering institutions, the responsibility for publishing the codes of practice was transferred from the engineering institutions to the British Standards Institution, where it has remained. Revised versions of CP 1 and CP 4 were published by BSI as CP 2001 in 1957 and CP 2004 in 1972, respectively. While they still had the phrase ‘code of practice’ in their titles, subsequent revisions of these codes of practice were published during the 1980s as British Standards with the prefix BS rather than CP. For example, the revised version of CP 2001 was published as BS 5930 in 1981 and the revised version of CP 2004 was published as BS 8004 in 1986. The first three revisions of BS 1377, in 1961, 1967 and 1975, were published as single documents. In the 1961 version, the title was changed to *Methods of Testing Soil for Civil Engineering*. The 1975 version was in metric form and included procedures for a greater range of soil tests. Subsequently, in 1990, the next revision and current version was published in nine separate parts with the slightly modified umbrella title *Method of Test for Soils for Civil Engineering Purposes*.

With the move towards harmonisation in Europe through the formation of the European Economic Community and then the European Union, the European Commission (EC) in 1975 initiated a programme to prepare the set of harmonised standards for the design of buildings and civil engineering works, known as the Structural Eurocodes. These standards are all based on the same limit state design method with partial factors. Thus they harmonise geotechnical design with structural design. In the case of geotechnical design, they have the additional advantage that, being based on the limit state design method, they provide a 'common language' for geotechnical design in the different countries in Europe. Until the publication of Eurocode 7, design codes and standards in Europe varied greatly. European harmonised standards are standards that are officially recognised by reference in the Official Journal of the European Union as meeting the essential requirements of a relevant European directive. In the case of the Eurocodes, the relevant directive is the Construction Products Directive 89/106/EEC (EC, 1989) of 21 December 1988 on the approximation of laws, regulations and administrative provisions of the member states relating to construction products. After 15 years' work on the Eurocodes, it was decided in 1990 to transfer this work from the EC to the CEN. The national standards organisations in Europe are members of CEN and hence BSI is the UK member of CEN. CEN has published Eurocode 7 as EN 1997.

Since the responsibility for establishing the required level of safety for structures in a particular country is a national responsibility; Eurocode 7 and each of the other Eurocodes is published with a National Annex. The National Annexes provide the values of the partial factors, referred to as Nationally Determined Parameters (NDPs), and other safety elements to be used with a particular Eurocode. The National Annexes also provide references to non-conflicting complementary information (NCCI) that may be used with the Eurocodes.

Each Eurocode has been prepared by a technical committee (TC) of CEN, TC 250 *Structural Eurocodes*. As well as preparing the Eurocodes for structural and geotechnical design, CEN has also in recent years been preparing, through its TCs, a large number of standards relating to other aspects of geotechnical engineering and many of these have now been published by CEN as ENs. Some standards relating to geotechnical engineering have also been prepared in recent years by technical committees of the International Standards Organization (ISO) and published as ISO standards, with the prefix ISO. In order to avoid duplication of work, it was decided in 1991, under the Vienna Agreement, that when CEN and ISO both required a standard in a particular area, they would cooperate in the preparation of the standard and exchange technical information. In practice, this has meant that there has been collaboration between CEN and ISO in the preparation of standards for geotechnical investigation and testing, most of these being prepared by CEN TC 341 *Geotechnical Investigation and Testing*, but with the standards for the identification and classification of soil and rock being prepared by ISO TC 182 *Geotechnics*.

As a member of CEN, BSI is obliged to adopt identical versions of all European Standards as British Standards, and to withdraw any national standards that might conflict with them. Hence, since the Eurocodes and other European standards have been approved and published by CEN as European Standards with the prefix EN, they have been adopted and published by BSI as British Standards with the prefix BS EN and the conflicting BS standards have been superseded. However, if BSI decides to keep an existing conflicting standard as a current document rather than withdraw it, this is achieved by changing its status from a standard, with the prefix BS, to a published document, with the prefix PD. This is a document that does not have the status of a standard but contains information that is useful for the industry. If ISO standards are adopted as British Standards, they are published by BSI with prefix BS ISO, unless they have been prepared by an ISO or CEN committee working under the Vienna Agreement, in which case they are published with the prefix BS EN ISO.

10.5 Why geotechnical and structural codes and standards differ

Geotechnical design codes and standards, such as Eurocode 7, differ in both structure and content from structural design codes. This is because soil has a number of special features which the materials used for structural design (e.g. steel) either do not have, or have only to a much lesser extent. The consequences for geotechnical design and Eurocode 7 of each of these special features of soil is explained in the following paragraphs and summarised in **Table 10.1**.

(a) *Soil as a natural material* The principal feature of soil which affects geotechnical design and causes geotechnical codes to differ from structural codes is that it is a natural material, unlike the materials used in structural design, such as concrete and steel, which are manufactured. The consequence of this is that the properties of the soil need to be determined as part of the geotechnical design process rather than being specified by the designer, as in the case of structural design. It is for this reason that Eurocode 7 has two parts, which cover all the components of geotechnical design. These are Part 1: *General rules*, with the principles and requirements for the geotechnical aspects of the design of buildings and civil engineering works, and Part 2: *Ground investigation and testing*, with the design requirements relating to the planning and reporting of ground investigations, the general requirements for a number of commonly used laboratory and field tests for soil, the interpretation of test results, and the derivation of geotechnical parameters and coefficients.

(b) *The two- or three-phase nature of soil* Soil consists of mineral particles which have shear strength, and water and air which have no shear strength. As a result, the principle

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Soil	Steel	Consequences for geotechnical design
(a) Natural material	Manufactured	Properties are determined, not specified, so ground investigation and testing are part of design process
(b) Two or three phases	Single phase	Need to consider water and water pressures as well as soil properties, and also short- and long-term behaviour
(c) Non-homogeneous and highly variable	Homogeneous	Characteristic value is not 5% fractile of test results and need to use comparable experience when selecting characteristic values. Also for redundant structures, stronger ground attracts load while weaker ground sheds load, so partial load factors close to unity are appropriate
(d) Frictional	Non-frictional	Loads affect resistances so need to take care factoring permanent loads, and hence partial factor of unity usually appropriate
(e) Ductile	Most manufactured materials are not as ductile as soil	Enables load redistribution in the ground and redundant structures, so that partial factors close to unity may be appropriate for structural loads
(f) Dilatant	Non-dilatant	Dilation on shearing and the magnitude of the shear strains need to be considered when deciding whether the peak, critical state or residual strength parameters are appropriate
(g) Compressible and with a low shear stiffness	Non-compressible	Design is often controlled by serviceability limit state, not by ultimate limit state
(h) Nonlinear with a complex stress–strain behaviour	Linear and simple	Serviceability limit state calculations usually difficult, so for simple situations, design may be carried out using ultimate limit state calculations

Table 10.1 Special features of soil compared with steel and consequences for geotechnical design

of effective stress needs to be taken into account in geotechnical design. This principle states that all effects of a change of stress on soil are due to changes in effective stress, where the effective stress is the total stress minus the pore water pressure – i.e. it implies that all changes of soil strength, compression and distortion are due to changes in the stresses acting on the soil particles, not due to changes in the pore water pressure. It is for this reason that the principle of effective stress is so fundamental to geotechnical design and why it is normally necessary in geotechnical design to determine the pore water pressures and understand how the pore water is behaving. Eurocode 7 takes account of this by constantly referring to the need in geotechnical design to consider pore water pressures and the effects of changes in pore water pressures, e.g. Paragraph 2.4.2(9)P. Furthermore, since changes in pore water pressure take time to occur in fine-grained, low permeability soils, and since the properties and behaviour of soils change with time as the pore water pressures dissipate, it is necessary in geotechnical design, as stated in Paragraph 2.2(1)P, to consider both short- and long-term design situations. This time-dependent behaviour is a unique feature of geotechnical engineering that does not occur in structural design. Further discussion of this behaviour is given in Chapter 17 *Strength and deformation behaviour of soils.*

(c) *The non-homogeneous and variable nature of soil* Soil is both non-homogeneous and variable and the volume of soil involved in any geotechnical design situation is usually very large compared with that which is sampled and tested. For these reasons, it is normally not appropriate to select the 5% fractile of the test results as the characteristic value for use in geotechnical designs (unlike when selecting material parameters for use in structural designs). According to Paragraph 2.5.4.2(2)P of Eurocode 7, the characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state, which is often the mean of a range of values covering a large surface or volume of the ground. Although it is not anticipated that statistics will normally be employed in the selection of the characteristic value, Eurocode 7 states that a cautious estimate of the mean value is a selection of the mean value of the limited set of geotechnical parameter values, with a confidence level of 95%. It is important that the characteristic value should be selected not only on the basis of soil tests results, but also on the basis of experience of similar soil conditions and the behaviour of similar structures in such soil.

The greater inhomogeneity and variability of soil compared with manufactured materials not only affects the selection of characteristic values in geotechnical design, but also the values chosen for the partial material and load factors. Due to the greater variability of soil and the greater uncertainty concerning the soil parameter values compared with structural parameter values, larger partial material factor values are generally chosen for geotechnical design than for structural design, e.g. 1.25 on $\tan\phi'$ compared with 1.15 on the strength of steel, where ϕ' is the effective angle of friction. As a result of the greater variability of soil, in geotechnical design, there is often less uncertainty in the loads compared to the soil parameter

values, particularly when the loads are due to the self-weight of the soil, or compared with the uncertainty in the loads in structural design; for this reason, lower partial load factors are often chosen for geotechnical design. For example, 1.0 on permanent loads and 1.3 on variable loads compared wit the partial load factors of 1.35 and 1.5, respectively, chosen for structural design.

Another consequence of the non-homogeneous and variable nature of soil is that, in the case of redundant structures on soil, the loads from the structure will be redistributed from areas where the soil is weaker (hence less able to support the loads) to areas where the soil is stronger (better able to support the loads). This unplanned redistribution process is favourable with regard to the geotechnical design and hence justifies the choice of low partial load factors. However, in the case of structural design, the situation is different because manufactured materials are not normally non-homogeneous and variable, with weak and strong zones. Unplanned load redistribution in redundant structures is usually unfavourable rather than favourable and thus necessitates higher partial load factors than in geotechnical design.

(d) *The frictional nature of soil* Since soil is frictional, its strength is a function of the normal effective stress as well as the angle of friction. Thus it is necessary to be careful when factoring permanent loads in geotechnical design calculations, and to ensure that normal loads which are unfavourable (i.e. those tending to cause failure) but which are also contributing to the soil strength and resistance , through providing the normal stress on the failure surface, are not increased. It is for this reason that the partial factor of 1.0 mentioned in (c) above is normally used on permanent loads, including the self-weight of soil, in geotechnical design. Further discussion of the frictional nature of soil is given in Chapter 17 *Strength and deformation behaviour of soils.*

(e) *The ductile nature of soil* Soil is usually very ductile – it can undergo large deformations when sheared and still retain significant shear strength, even if, on shearing, it first reaches a peak strength and then reduces to a residual strength. Although some structural materials, such as steel, are ductile, the extent of their ductility is normally less than for soil and hence, unless special measures are included to improve the ductility, structures are not able to deform as much as soil before failing. A consequence of the ductile behaviour of soil is that when homogeneous soil supports a redundant structure, those areas that are more heavily loaded will yield, allowing load to be transferred through the structure from the more heavily loaded to the less heavily loaded areas. Hence, this ductile nature is beneficial for the geotechnical design. This is another reason why the partial factors on permanent and variable loads in geotechnical design are often less than those in structural design.

(f) *The dilatant behaviour of soil* Some soils, for example dense coarse-grained soils, dilate when sheared, exhibiting a peak angle of friction, ϕ'_p. After sufficient shearing, they reach the critical state or constant-volume angle of friction, ϕ'_{cv} when the soil is completely remoulded. In the case of other soils, for example clays, the soil particles may become aligned along the failure surface after large strains, with the result that a residual angle of friction is reached which is less than the critical state angle. Generally what is important in geotechnical design is the soil strength that is available to prevent failure occurring, rather than the strength after failure. Eurocode 7 states in Paragraph 2.4.3(2)P that the geotechnical parameter 'values obtained from test results and other data shall be interpreted appropriately for the limit state considered' and in Paragraph 2.4.3(4)P notes that 'account shall be taken of the possible differences between the ground properties and geotechnical parameters obtained from test results and those governing the behaviour of the geotechnical structure', giving soil and rock structure and brittleness as two of the factors that may cause these differences. Further discussion of the dilatant behaviour of soil is given in Chapter 17 *Strength and deformation behaviour of soils.*

Eurocode 7 generally does not state if the soil shear strength to be used in design calculations is the peak, critical state, or residual value. Paragraph 3.3.6(1)P notes that, when assessing the shear strength, the influence of features including the stress level, anisotropy of strength, strain rate effects, very large strains where these may occur in a design situation, pre-formed slip surfaces and degree of saturation, shall be considered. Since structures will generally be considered to have failed well before the deformations required to get to the critical state have been reached (geotechnical designs have traditionally been carried out using the peak strength), and since most field tests are correlated to the peak strength, this is the strength that is normally used in designs to Eurocode 7. This is indicated by Paragraph 6.5.3(5), which, referring to the strength used to determine the resistance to sliding, states that 'for large movements, the possible relevance of post-peak behaviour should be considered'. The only paragraphs in Eurocode 7 that refer to the use of a specific angle of friction are Paragraphs 6.5.3(10) and 9.5.1(3). Paragraph 6.5.3(10) states that 'the design (structure-ground interface) friction angle δ_d may be assumed equal to the design value of the effective critical state angle of shearing resistance, $\phi'_{cv,d}$, for cast *in situ* concrete foundations and equal to 2/3 $\phi'_{cv,d}$ for smooth pre-cast foundations'. Paragraph 9.5.1(3) states that 'A concrete wall or sheet pile wall supporting sand or gravel may be assumed to have a design wall–ground

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interface parameter $\delta_d = k\phi_{cv,d}$. k, the wall-ground interface friction angle factor, should not exceed 2/3 for precast concrete or sheet piling'.

(g) *The compressible nature and low shear stiffness of soil* Since soil, particularly fine-grained soil, is compressible with a low shear stiffness, and is normally very much more compressible and less stiff than structural materials, it can undergo large deformations exceeding the limiting values for the structure. Hence the controlling criterion in many geotechnical design situations is limiting the deformations rather than preventing failure. Geotechnical designs are therefore often controlled by serviceability limit state considerations rather than by ultimate limit state requirements.

(h) *The nonlinear and complex stress strain behaviour of soil* The nonlinear and complex stress–strain behaviour of soil, and the difficulties in determining appropriate parameter values and modelling soil deformations, sometimes make it difficult to obtain reliable predictions of the ground deformations. Consequently for simple design situations, geotechnical designs may be carried out using ultimate limit state calculations with appropriate partial or overall safety factor values, chosen so as to limit the mobilised shear strength and hence the ground deformations. An example of this in Eurocode 7 is the application rule in Paragraph 6.6.2(16), which states that 'For conventional structures founded on clays, the ratio of the bearing capacity of the ground, at its initial undrained shear strength, to the applied serviceability loading should be calculated. If this ratio is less than 3, calculations of settlements should always be undertaken. If the ratio is less than 2, the calculations should take account of nonlinear stiffness effects in the ground'.

10.6 The geotechnical design triangle

Burland (1987) proposed that the teaching of soil mechanics should focus on four distinct aspects. He originally presented these four aspects as the soil mechanics triangle, a concept which he has developed and expanded into the geotechnical triangle shown in **Figure 10.1** and discussed in Chapter 4 *The geotechnical triangle*. The geotechnical triangle has the first three aspects at its apexes: the ground profile, the soil behaviour and an appropriate model. The fourth aspect: precedent, empiricism and well-winnowed experience, lies at its centre and is linked to the other three.

In a similar way, Orr (2008) has shown that the different components of geotechnical design can be formed into the geotechnical design triangle shown in **Figure 10.2**. The components of geotechnical design that form the apexes of the geotechnical design triangle are the geometrical data, the loads and geotechnical parameters, and the verification method. Linking these three components is the fourth component, the design

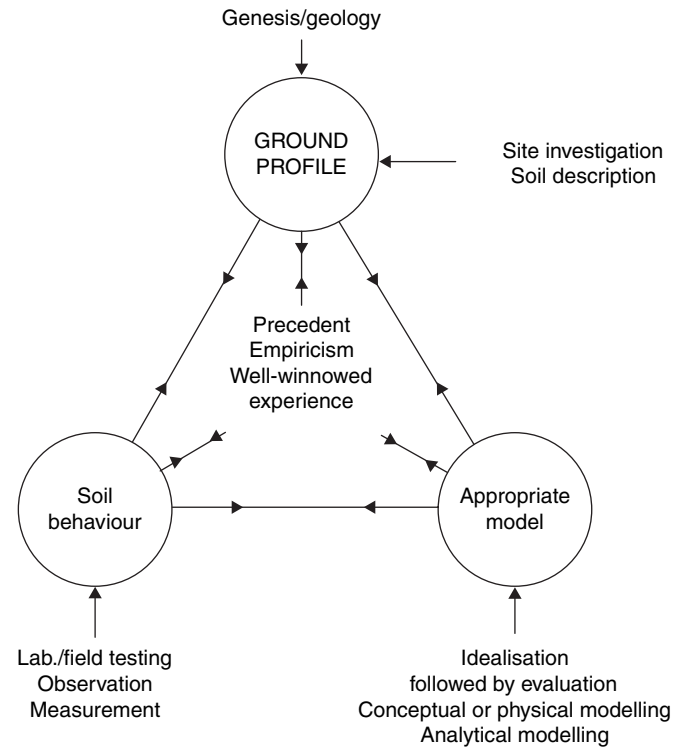


Figure 10.1 Geotechnical triangle

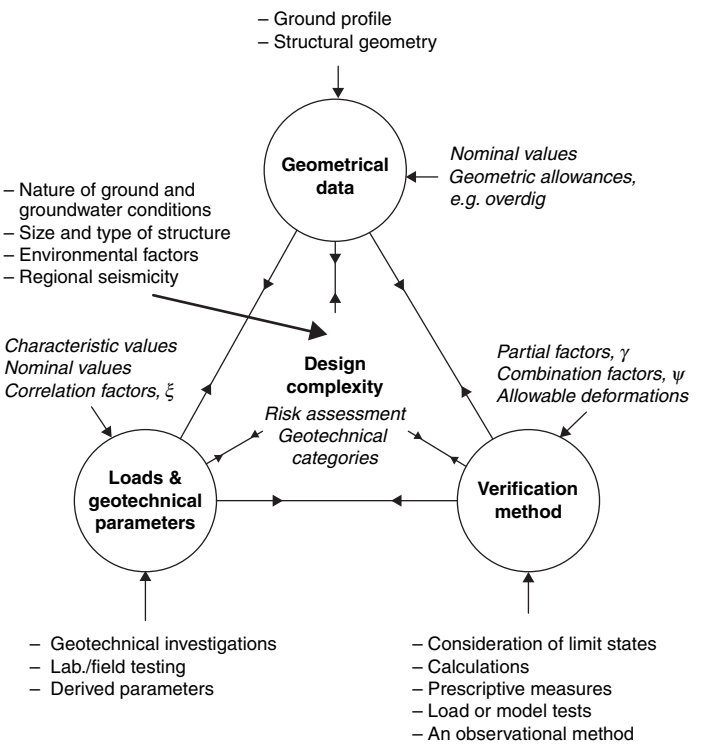


Figure 10.2 Geotechnical design triangle

complexity. How the design complexity affects the other geotechnical design components is an important design consideration and hence design complexity has been placed at the centre of the geotechnical design triangle, linked to each of the other three design components in the same way as precedent, empiricism and well-winnowed experience are linked to the components at the apexes of the geotechnical triangle.

The factors and design procedures involved in the four geotechnical design components are listed as bullet points in the geotechnical design triangle in **Figure 10.2**. The factors involved in the first component, the data describing the geometry of the design situation, include the level and slope of the ground surface, water levels, levels of interfaces between strata, excavation levels and the dimensions of the geotechnical structure. The loads involved in geotechnical designs are usually either provided by the structural designer or determined from the weight density (unit weight) of the soil, while the geotechnical parameters, such as the soil strength and stiffness, are determined as derived values from geotechnical investigations involving field and laboratory tests.

Regarding the verification method, with the introduction of Eurocode 7, the limit state design method has replaced the permissible-stress method for geotechnical design. This design method requires that all the relevant limit states are considered and shown to be sufficiently improbable. This should be achieved by adopting one or more of the following: the use of calculations, the adoption of prescriptive measures, loads or model tests, and an observational method. The factors that make up the fourth component, the design complexity, are the nature of the ground and the groundwater conditions, the nature of the structure, the nature of the surroundings, the environmental factors and, where relevant, the regional seismicity. Each of these factors and their level of complexity needs to be considered at the start of a geotechnical design as they will affect how each of the other three components is treated.

10.7 Safety elements adopted in Eurocode 7

In geotechnical designs to Eurocode 7, each component of the geotechnical design triangle is treated in an appropriate manner or with an appropriate safety element to ensure that the required level of safety is achieved. The treatment manner or safety elements adopted in the case of each component are explained in the following paragraphs and are shown in italics in **Figure 10.2**.

In the case of the geometrical data, the values used for design are measured or chosen nominal values. Where appropriate, geometrical allowances are included as the measures to ensure safety; examples of such allowances in BS EN 1997–1 are the overdig allowance in the design of retaining structures (Paragraph 9.3.2.2(2)) and the additional constructional tolerance on the foundation width in the case of spread foundations with highly eccentric loads (Paragraph 6.5.4(2)).

In the case of the loads and geotechnical parameters, the uncertainty and variability in these are taken into account and safety is achieved through the selection of suitably cautious values, which are known as characteristic values. Where sufficient statistical data are available, the characteristic value of a load is defined as the 95% fractile value of the load distribution, while in the case of a material parameter, it is defined as the 5% fractile value. However, often sufficient data are not available to obtain the characteristic value of a load statistically and hence a 'nominal' value is used instead. When combinations of loads are involved in a design situation, combination factors are applied to the characteristic loads to obtain 'representative' values of loads. These are the values to which the partial action (load) factors are applied to obtain the design values that are used in design calculations to Eurocode 7. Regarding the characteristic value of a geotechnical parameter, Paragraph 2.5.4.2(2)P of EN 1997–1 states that it 'shall be selected as a cautious estimate of the value affecting the occurrence of the limit state'. When the pile resistance is determined from pile load tests or profiles of field tests, BS EN 1997–1 provides correlation factors, ξ, that are applied to the test results or profiles to determine the characteristic pile resistance. The ξ values used to determine the characteristic pile resistance depend on the number of pile load tests or test profiles.

With regard to the verification method, the required level of safety when using the limit state design method in BS EN 1997–1 is achieved by applying appropriate partial factors to the loads and/or the geotechnical parameters values or resistances in the ultimate or serviceability limit state calculation models. Partial model factors may also be introduced to account for uncertainty in the calculation model, though this uncertainty is normally accounted for in the values chosen for the partial load and material factors. The values of the partial factors provided in the UK National Annex to EN 1997–1 are all for ultimate limit states, since the partial factors for serviceability limit states are all equal to unity.

The way that safety is introduced in BS EN 1997–1 with regard to the complexity of a design situation is by the requirement in Paragraph 2.1(8)P: that the complexity of each design situation shall be identified together with the associated risks so that minimum requirements for the extent and content of the geotechnical investigations, calculations and construction control checks can be established. When identifying and addressing the complexity of a design situation, it is important to take account of comparable experience, which is defined in Paragraph 1.5.2.2 as 'documented or other clearly established information related to the ground being considered in design, involving the same types of soil and rock and for which similar geotechnical behaviour is expected, and involving similar structures. Information gained locally is considered to be particularly relevant'. An example of a risk assessment procedure in BS EN 1997–1 to address the design complexity is the geotechnical categories given in Paragraph 2.1(10). These geotechnical categories are presented as an application rule, not as a principle, in BS EN 1997–1, and hence their use is optional for geotechnical designs carried out in the UK.

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Soil mechanics aspects	Geotechnical design components	Treatment manner or safety elements
Ground profile	Geometrical data	Nominal values
– Genesis/geology	– Level and slope of ground surface	Foundation tolerance
– Site investigation	– Water levels	Overdig allowance
– Soil description	– Levels of strata interfaces	
	– Excavation levels	
	– Structural dimensions	
Soil behaviour	Loads and geotechnical parameters	Characteristic values
– Laboratory/field tests	– Geotechnical investigations	Nominal values
– Observation	– Field testing	Correlation factors, ξ
– Measurement	– Laboratory testing	
	– Derived parameters	
Appropriate model	Verification method	Partial factors, γ
– Idealisation followed by evaluation	– Consideration of all relevant limit states	Combination factors ,ψ
– Conceptual or physical modelling	Verification by	Design values
– Analytical modelling	– Calculations	Limiting movements
	– Prescriptive measures	
	– Load or model tests	
	– An observational method	
– Precedent	Design complexity	Risk assessment
– Empiricism	– Soil conditions	Geotechnical categories
– Well-winnowed experience	– Groundwater situation	Comparable experience
	– Influence of the environment	
	– Regional seismicity	
	– Nature of structure	
	– Nature of surroundings	

Table 10.2 Relationship between the geotechnical design components and soil mechanics aspects (continued overleaf)

10.8 Relationship between the geotechnical design triangle and the geotechnical triangle

The relationship between the four components of the geotechnical design triangle and the aspects of the geotechnical triangle is shown in **Table 10.2**. This table shows that the components of geotechnical design are related to and cover the same areas as Burland's aspects of soil mechanics. However, the safety elements of the geotechnical design components, which are a fundamental feature of geotechnical design, are not included in the four aspects of soil mechanics, which were identified as being essential for the understanding of soil behaviour.

With regard to complexity, Burland (2006) stated that he considered the main difficulty in soil mechanics was not so much in the complexity of the material as such, but in the fact that, all too often, the boundaries between the aspects become confused. He also noted the need for precedent, empiricism and well-winnowed experience to moderate and account for uncertainties in the aspects of soil mechanics. A similar situation exists in geotechnical design whereby it is not so much the complexity of the situation that affects the safety of a design, but the need to identify the uncertainties involved in each of the design components and adopt appropriate procedures for each component. This involves using comparable experience as defined in EN 1997–1. An important statement in BS EN 1997–1 related to ensuring safety in complex geotechnical design situations is Paragraph 2.4.1(2), which notes that 'knowledge of the ground conditions, which depend on the extent and quality of the geotechnical investigations, and the control of workmanship are usually more significant to fulfilling the fundamental requirements than is precision in the calculation models and partial factors'.

10.9 Codes and standards for geotechnical engineering

10.9.1 Groups of geotechnical codes and standards

The codes and standards in the UK for geotechnical engineering may be divided into four groups, presented in **Tables 10.3**, **10.4**, **10.5** and **10.6**: investigations and testing, geotechnical design, the execution of geotechnical work, and materials used in geotechnical engineering. These tables show the

prefixes and reference numbers of the standards, their titles, their abstract/descriptors/keywords and their status. Standards that have been published by BSI are shown in the tables with their date of publication after the reference number. Standards that are being or have been prepared by CEN or ISO, but have not yet been published by BSI, are shown with no date; the prefix it is anticipated they will have when they become British Standards is shown in brackets, e.g. (BS EN ISO).

10.9.2 Codes and standards for geotechnical investigation and testing

As shown in **Table 10.3**, the current British Standards for geotechnical investigations and testing are BS 5930: *Site investigations* and BS 1377: *Methods of Test for Soils for Civil Engineering Purposes*, which has nine parts covering laboratory and *in situ* tests. Since Parts 1 and 2 of Eurocode 7 have sections that cover the design aspects of site investigations, for example the planning and location of site investigations, these sections supersede those parts of BS 5930 which cover the same aspects and hence an amended version of BS 5930 was published by BSI in 2007 to avoid conflict with the two parts of BS EN 1997–1.

The ISO standards for the identification and classification of soil and rock are:

- 14688 *Identification and Classification of Soil* (2 parts);
- 14689 *Identification and Classification of Rock* (1 part).

These three parts have been published by CEN as EN ISO standards. Hence they have been published by BSI as the BS standards BS EN ISO 14688–1, BS EN ISO 14688–2 and BS EN ISO 14689–1 and have been introduced into UK practice. Since these new BS standards cover the identification and classification of soil and rock that was covered by BS 5930, they supersede those parts of BS 5930. Hence the amended version of BS 5930 issued in 2007 has been revised to remove soil and rock identification and classification to avoid conflict with these new BS standards.

CEN has prepared and is preparing, under the Vienna agreement, the following very comprehensive range of standards for geotechnical investigations and testing which will eventually supersede most parts of BS 5930 and BS 1377:

- 17892 *Laboratory Testing of Soil* (12 parts);
- 22282 *Geohydraulic Testing* (6 parts);
- 22475 *Sampling Methods and Groundwater Measurements* (3 parts);
- 22476 *Field Testing* (13 parts);
- 22477 *Testing of Geotechnical Structures* (7 parts).

All 12 parts of 17892 on laboratory testing have been published by CEN as Technical Specifications (TSs), where a TS is a CEN normative document in an area where the state of the art is not yet sufficiently developed to allow agreement as an EN but it may be adopted as a national standard. The only part of 17892 to be published so far by BSI is Part 6, which was published in 2010 as the Document for Development (DD) DD CEN ISO/TS 17892–6:2004: *Geotechnical investigation and testing. Laboratory testing of soils. Fall cone* – A DD is a provisional document that needs further information and experience of its practical application before BSI can decide to support its conversion into an international standard. BSI is working closely with CEN to convert the TSs of 17892 to ENs and publish them as BSs.

The six parts of 22282 on geohydraulic testing have been issued by CEN for formal vote and it is anticipated that they will be published by BSI as BS EN ISOs in 2012 and supersede parts of BS 5930.

Part 1 of EN 22475 on sampling and groundwater measurement has been published by CEN as TSs and by BSI as a BS, superseding parts of BS 5930. Parts 2 and 3 have been published by BSI as DDs.

Three EN standard parts for geotechnical field tests that have been adopted as BSs are Parts 2, 3 and 12 of BS EN ISO 22476, the standards for dynamic probing, the standard penetration test, and mechanical cone penetration test which supersede parts of Part 9 of BS 1377 on *in situ* tests, while Parts 1–8 of BS 1377 remain as British Standards. Parts 1, 4–11 and 13 of ISO 22476 are at different stages of development. Part 10 has been published by CEN as a TS, Part 11 has been published by BSI as a DD, Parts 1, 4, 5, 7 and 9 are expected to be published in 2012, while Parts 6, 8 and 13 are still under development.

All parts of ISO 22477 on the testing of geotechnical structures are still being developed.

10.9.3 Codes and standards for geotechnical design

The British Standards concerned with geotechnical design are listed in **Table 10.4**. The first codes of practice listed in this table are those that were prepared and published by BSI prior to the publication of Eurocode 7:

- BS 6031 *Code of Practice for Earthworks*;
- BS 8002 *Code of Practice for Earth Retaining Structures*;
- BS 8004 *Code of Practice for Foundations*;
- BS 8006 *Code of Practice for Strengthened/Reinforced Soils and Other Fills*;
- BS 8081 *Code of Practice for Ground Anchorages*;
- BS 8103–1 *Code of Practice for Stability, Site Investigations, Foundations for Ground Floors for Housing*;
- CP 2012–1 *Code of Practice for Foundations for Machinery. Foundations for Reciprocating Machines.*

Since March 2010, the British Standard for geotechnical design is *Eurocode 7: Geotechnical design,* with its two parts:

- BS EN 1997–1 *Geotechnical Design: General Rules*;
- BS EN 1997–2 *Geotechnical Design: Ground Investigation and Testing*.

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Reference	Title	Keywords/descriptors/abstract	Status
BS 5930:1999	Code of practice for site investigations	Site investigations, Soil surveys, Soil sampling, Soil testing, Groundwater, Rocks, Safety measures, Field testing, Excavations, Soil drilling, Aerial photography, Geological analysis, Sampling methods, Sampling equipment, Test specimens, Samples, Surveys, Geophysical measurement, Quality assurance, Reports, Classification systems, Density measurement	Amended in December 2007 to avoid conflict with newly introduced BS EN 1997–1 and retained as a normative reference
BS 1377–1: 1990	Methods of test for soils for civil engineering purposes – Part 1: General requirements and sample preparation	Soil sampling, Soil testing, Field testing, Sampling methods, Specimen preparation, Test equipment, Sampling equipment, Testing conditions, Laboratory testing, Soil-testing equipment	Current BS
BS 1377–2: 1990	Methods of test for soils for civil engineering purposes – Part 2: Classification tests	Soil classification tests, Water content determination, Test equipment, Specimen preparation, Liquid limit (soils), Soil-testing equipment, Testing conditions, Plastic limit (soils), Shrinkage tests, Density measurement, Relative density, Particle size distribution, Sedimentation techniques	Current BS
BS 1377–3: 1990	Methods of test for soils for civil engineering purposes – Part 3: Chemical and electro-chemical tests	Groundwater, Determination of content, Chemical analysis and testing, Organic matter determination, Specimen preparation, Ignition-loss tests, Sulfates, Gravimetric analysis, Carbonates, Volumetric analysis, Chlorides, Extraction methods of analysis, Solvent extraction methods, pH measurement, Electrical resistivity, Electrical measurement	Current BS
BS 1377–4: 1990	Methods of test for soils for civil engineering purposes – Part 4: Compaction related tests	Compaction tests, Soil compaction tests, Moisture measurement, Water content determination, Specimen preparation, Test equipment, Density measurement, Crushing tests, Impact testing, Compression testing, Soil bearing capacity, Penetration tests, Soaking tests	Current BS
BS 1377–5: 1990	Methods of test for soils for civil engineering purposes – Part 5: Compressibility, permeability and durability tests	Construction materials, Consolidation test (soils), Testing conditions, Compression testing, Calibration, Deformation, Mechanical testing, Specimen preparation, Test pressure, Swelling, Permeability measurement, Soil classification tests, Hydraulic tests, Sedimentation techniques, Frost susceptibility, Soil strength tests	Current BS
BS 1377–6: 1990	Methods of test for soils for civil engineering purposes – Part 6: Consolidation and permeability tests in hydraulic cells and with pore pressure measurement	Permeability measurement, Consolidation test (soils), Soils, Soil testing, Triaxial test (soils), Soil-testing equipment, Specimen preparation, Calibration, Testing conditions, Mathematical calculations, Reports, Soil strength tests, Test equipment, Construction	Current BS
BS 1377–7: 1990	Methods of test for soils for civil engineering purposes – Part 7: Shear strength tests (total stress)	Shear testing, Mechanical testing, Soil-testing equipment, Mathematical calculations, Specimen preparation, Testing conditions, Compression testing, Soil strength tests, Vane test, Triaxial test (soils)	Current BS
BS 1377–8: 1990	Methods of test for soils for civil engineering purposes – Part 8: Shear strength tests (effective stress)	Shear testing, Compression testing, Consolidation test (soils), Soil strength tests, Triaxial test (soils), Testing conditions, Test specimens	Current BS
BS 1377–9: 1990	Methods of test for soils for civil engineering purposes – Part 9: *In situ* tests	Field testing, Density measurement, Moisture measurement, Radiation measurement, Penetration tests, Soil strength tests, Soil bearing capacity, Vane test, Shear testing, Mechanical testing, Electrical testing	Partially superseded by BS EN ISO 22476–2 and 3
BS EN ISO 14688–1: 2002	Geotechnical investigation and testing. Identification and classification of soil – Part 1: Identification and description	The basic principles for the identification and classification of soils on the basis of those material and mass characteristics most commonly used for soils for engineering purposes. The general identification and description of soils is based on a flexible system for immediate (field) use by experienced persons, covering both material and mass characteristics by visual and manual techniques. Details are given of the individual characteristics for identifying soils and the descriptive terms in regular use, including those related to the results of tests from the field. Applicable to natural soils ***in situ,*** similar man-made materials ***in situ*** and soils redeposited by man	Introduced into UK practice in 2007 following amendment of BS 5930 published in 2007

Table 10.3 (continued)

BS EN ISO 14688–2: 2004	Geotechnical investigation and testing. Identification and classification of soil – Part 2: Principles for a classification	The basic principles for the identification and classification of soils on the basis of those material and mass characteristics most commonly used for soils for engineering purposes. The classification principles permit soils to be grouped into classes of similar composition and geotechnical properties, and with respect to their suitability for geotechnical engineering purposes. Applicable to natural soil and similar man-made material ***in situ*** and redeposited, but it is not a classification of soil by itself	Introduced into UK practice in 2007 following amendment of BS 5930 published in 2007
BS EN ISO 14689–1: 2003	Geotechnical investigation and testing. Identification and classification of rock. Identification and description	Identifies and describes rock material and mass on the basis of mineralogical composition, genetic aspects, structure, grain size, discontinuities and other parameters. It also provides rules for the description of various other characteristics as well as for their designation	Introduced into UK practice in 2007 following amendment of BS 5930 published in 2007
(BS EN ISO) 17892–1	Geotechnical investigation and testing – Laboratory testing of soil – Part 1: Determination of water content	This document specifies the laboratory determination of the water (moisture) content of a soil test specimen by oven-drying within the scope of the geotechnical investigations according to EN 1997–1 and 2	Published by CEN as a TS but will not be published by BSI as a DD. BSI is working closely with CEN to convert the TS into an EN and will publish the EN as a BS
(BS EN ISO) 17892–2	Geotechnical investigation and testing – Laboratory testing of soil – Part 2: Determination of density of fine-grained soil	This document specifies methods of test for the determination of the bulk and dry density of intact soil or rock within the scope of the geotechnical investigations according to EN 1997–1 and EN 1997–2. This document describes three methods: (a) linear measurements method (b) immersion in water method (c) fluid displacement method	Published by CEN as a TS but will not be published by BSI as a DD. BSI is working closely with CEN to convert the TS into an EN and will publish the EN as a BS
(BS EN ISO) 17892–3	Geotechnical investigation and testing – Laboratory testing of soil – Part 3: Determination of particle density – Pycnometer method	This document describes a test method for determining the particle density by the pycnometer method within the scope of the geotechnical investigations according to prEN 1997–1 and prEN 1997–2. The pycnometer method is based on the determination of the volume of a known mass of soil by the fluid displacement method	Published by CEN as a TS but will not be published by BSI as a DD. BSI is working closely with CEN to convert the TS into an EN and will publish the EN as a BS
(BS EN ISO) 17892–4	Geotechnical investigation and testing – Laboratory testing of soil – Part 4: Determination of particle size distribution	This document describes methods for the determination of the particle size distribution of soil samples. The particle size distribution provides a description of soil, based on a subdivision in discrete classes of particle size. The size of each class can be determined by sieving and/or sedimentation	Published by CEN as a TS but will not be published by BSI as a DD. BSI is working closely with CEN to convert the TS into an EN and will publish the EN as a BS
(BS EN ISO) 17892–5	Geotechnical investigation and testing – Laboratory testing of soil – Part 5: Incremental loading oedometer test	This document is intended for determination of the compression, swelling and consolidation properties of soils. The cylindrical test specimen is confined laterally, is subjected to discrete increments of vertical axial loading or unloading and is allowed to drain axially from the top and bottom surfaces	Published by CEN as a TS but will not be published by BSI as a DD. BSI is working closely with CEN to convert the TS into an EN and will publish the EN as a BS
DD CEN ISO/ TS 17892–6: 2010	Geotechnical investigation and testing – Laboratory testing of soil – Part 6: Fall cone test	This DD specifies the laboratory determination of undrained shear strength of both undisturbed and remoulded specimens of saturated fine-grained cohesive soils by use of a fall-cone	Published by CEN as a TS in 2004 and published by BSI as a DD in 2010
(BS EN ISO) 17892–7	Geotechnical investigation and testing – Laboratory testing of soil – Part 7: Unconfined compression test on fine-grained soils	This document covers the determination of an approximate value of the unconfined compressive strength for a square or cylindrical water-saturated homogeneous specimen of undisturbed or remoulded cohesive soil of sufficiently low permeability to keep itself undrained during the time it takes to perform the test, within the scope of geotechnical investigations according to prEN 1997–1 and 2	Published by CEN as a TS but will not be published by BSI as a DD. BSI is working closely with CEN to convert the TS into an EN and will publish the EN as a BS
(BS EN ISO) 17892–8	Geotechnical investigation and testing – Laboratory testing of soil – Part 8: Unconsolidated undrained triaxial test	This document describes the test method for the determination of the compressive strength of a cylindrical, water-saturated specimen of undisturbed or remoulded cohesive soil when first subjected to an isotropic stress without allowing any drainage from the specimen, and thereafter sheared under undrained conditions within the scope of the geotechnical investigations according to prEN 1997–1 and 2	Published by CEN as a TS but will not be published by BSI as a DD. BSI is working closely with CEN to convert the TS into an EN and will published the EN as a BS

(BS EN ISO) 17892–9	Geotechnical investigation and testing – Laboratory testing of soil – Part 9: Consolidated trail compression tests on water-saturated soils	This document covers the determination of stress–strain relationships and effective stress paths for a cylindrical, water-saturated specimen of undisturbed, remoulded or reconstituted soil when subjected to an isotropic or an anisotropic stress under undrained or drained conditions and thereafter sheared under undrained or drained conditions within the scope of the geotechnical investigations according to prEN 1997–1 and 2	Published by CEN as a TS but will not be published by BSI as a DD. BSI is working closely with CEN to convert the TS into an EN and will published the EN as a BS
(BS EN ISO) 17892–10	Geotechnical investigation and testing – Laboratory testing of soil – Part 10: Direct shear tests	This document specifies laboratory test methods to establish the effective shear strength parameter for soils within the scope of the geotechnical investigations according to prEN 1997–1 and 2	Published by CEN as a TS but will not be published by BSI as a DD. BSI is working closely with CEN to convert the TS into an EN and will published the EN as a BS
(BS EN ISO) 17892–11	Geotechnical investigation and testing – Laboratory testing of soil – Part 11: Determination of permeability by constant and falling head	This document specifies laboratory test methods to establish the coefficient of permeability of water through water-saturated soils	Published by CEN as a TS but will not be published by BSI as a DD. BSI is working closely with CEN to convert the TS into an EN and will published the EN as a BS
(BS EN ISO) 17892–12	Geotechnical investigation and testing – Laboratory testing of soil – Part 12: Determination of Atterberg limits	This document specifies methods of test for the determination of the Atterberg limits of a soil: the liquid limit, plastic limit and shrinkage limit. It also describes the determination of the liquid limit of a specimen of soil using the fall-cone method	Published by CEN as a TS but will not be published by BSI as a DD. BSI is working closely with CEN to convert the TS into an EN and will published the EN as a BS
(BS EN ISO) 22282–1	Geotechnical investigation and testing – Geohydraulic testing – Part 1: General rules	This document provides the general rules and principles for geohydraulic testing in soil and rock as part of a geotechnical investigation. It defines concepts and specifies requirements relating to permeability measurement in soil and rock	This has received a positive Formal Vote in CEN to be published as an EN. Subsequently it will be published by BSI as a BS
(BS EN ISO) 22282–2	Geotechnical investigation and testing – Geohydraulic testing – Part 2: Water permeability tests in a borehole using open systems	This document specifies requirements for determining the local permeability in soils and rocks above and below ground water level in an open hole by water permeability tests as part of a geotechnical investigation. It also includes requirements for estimating the permeability of unsaturated soils	This has received a positive Formal Vote in CEN to be published as an EN. Subsequently it will be published by BSI as a BS
(BS EN ISO) 22282–3	Geotechnical investigation and testing – Geohydraulic testing – Part 3: Water pressure tests in rock	This document deals with the requirements for water pressure tests (WPT) in boreholes drilled into rock to investigate the hydraulic properties of the rock mass, absorption capacity, tightness of the rock mass, effectiveness of grouting and geomechanical behaviour	This has received a positive Formal Vote in CEN to be published as an EN. Subsequently it will be published by BSI as a BS.
(BS EN ISO) 22282–4	Geotechnical investigation and testing – Geohydraulic testing – Part 4: Pumping tests	This document deals with requirements for pumping tests as part of a geotechnical investigation. It applies to pumping tests performed on aquifers whose permeability is such that pumping from a well can create a lowering of the piezometric head within hours or days, depending on the ground conditions and the purpose. It covers pumping tests carried out in soils and rock	This has received a positive Formal Vote in CEN to be published as an EN. Subsequently it will be published by BSI as a BS
(BS EN ISO) 22282–5	Geotechnical investigation and testing – Geohydraulic testing – Part 5: Infiltrometer tests	This document deals with requirements for infiltrometer tests as part of a ground investigation. The infiltrometer test is used to determine the infiltration capacity of the ground at the surface or shallow depth and the permeability coefficient. The principle of the test is based on the measurement of a surface vertical flow rate of water which infiltrates the soil under a positive hydraulic head	This has received a positive Formal Vote in CEN to be published as an EN. Subsequently it will be published by BSI as a BS
(BS EN ISO) 22282–6	Geotechnical investigation and testing – Geohydraulic testing – Part 6: Water permeability tests in a borehole using closed systems	This document specifies requirements for determination of the local permeability in soils and rocks above or below the ground water table in a closed system by the water permeability tests as part of a geotechnical investigation. The tests are used to determine the permeability coefficient k in low permeable soil and rock with k lower than 10^{-8} m/s. These can also be used to determine the transmissivity T and the storage coefficient S	This has received a positive Formal Vote in CEN to be published as an EN. Subsequently, it will be published by BSI as a BS

Table 10.3 (continued)

BS EN ISO 22475–1: 2006	Geotechnical investigation and testing. Sampling methods and groundwater measurements – Part 1: Technical principles for execution	Deals with the technical principles of sampling soil, rock and groundwater, and with groundwater measurements, in the context of geotechnical investigation and testing, as described in EN 1997–1 and EN 1997–2	Partially supersedes BS 5930:1999. Where conflict arises, BS EN ISO 22475–1:2006 should take precedence
BS 22475–2: 2011	Geotechnical investigation and testing. Sampling methods and groundwater measurements – Part 2: Qualification criteria for enterprises and personnel	This standard specifies the qualification criteria for an enterprise and for personnel performing sampling and groundwater measurement services, so that all have the appropriate experience, knowledge and qualifications as well as the correct equipment and groundwater measurements for the task to be carried out according to ISO 22475–1	Current BS
BS 22475–3: 2011	Geotechnical investigation and testing. Sampling methods and groundwater measurements – Part 3: Conformity assessment of enterprises and personnel by third party	This standard specifies the minimum criteria for external audit. It is applicable to the conformity assessment of enterprises and personnel performing specified parts of the sampling and groundwater measurements according to ISO 22475–1 and complying with the technical qualification criteria given in ISO/TS 22475–2 by third-party control	Current BS
(BS EN ISO) 22476–1	Geotechnical investigation and testing. Field testing – Part 1. Electrical cone and piezocone penetration tests	Geology, Penetration tests, Site investigations, Field testing	Publication expected 2012
BS EN ISO 22476–2: 2005	Geotechnical investigation and testing. Field testing – Part 2: Dynamic probing	This standard defines requirements for indirect investigations of soil by dynamic probing as part of geotechnical investigation and testing according to EN 1997–1 and 2	Current BS partially supersedes BS 5930:1999 and BS 1377–9:1990
BS EN ISO 22476–3: 2005	Geotechnical investigation and testing. Field testing – Part 3: Standard penetration test	This standard defines requirements for indirect investigations of soil by the standard penetration test as part of geotechnical investigation and testing according to EN 1997–1 and 2 to complement direct investigations	Current BS partially supersedes BS 5930:1999 and BS 1377–9:1990
(BS EN ISO) 22476–4	Geotechnical investigation and testing. Field testing. – Part 4: Ménard pressuremeter test	This document deals with field testing using the Ménard pressuremeter test as part of geotechnical investigation and testing	Publication expected 2012
(BS EN ISO) 22476–5	Geotechnical investigation and testing. Field testing. – Part 5: Flexible dilatometer test	This document deals with field testing using the flexible dilatometer as part of geotechnical investigation and testing	Publication expected 2012
(BS EN ISO) 22476–6	Geotechnical investigation and testing. Field testing. – Part 6: Self-boring pressuremeter test	No details available	Standard under development
(BS EN ISO) 22476–7	Geotechnical investigation and testing – Field testing – Part 7: Borehole jacking test	This document deals with field testing using the borehole jack test as part of geotechnical investigation and testing according to EN 1997–1 and 2	Publication expected 2012
(BS EN ISO) 22476–8	Geotechnical investigation and testing – Field testing – Part 8: Full displacement pressuremeter test	No details available	Standard under development
(BS EN ISO) 22476–9	Geotechnical investigation and testing – Field testing – Part 9: Field vane test	Soil mechanics, site investigations, field testing, soils, rocks, pressure testing, bore-holes	Publication expected 2012
CEN ISO/TS 22476–10: 2005	Geotechnical investigation and testing. Field testing – Part 10: Weight sounding test	This TS specifies laboratory test methods to establish the effective shear strength parameter for soils within the scope of the geotechnical investigations according to EN 1997–1 and 2. The test method consists of placing the test specimen in the direct shear device, applying a pre determined normal stress, providing for draining (and wetting if required) of the test specimen, or both, consolidating the specimen under normal stress, unlocking the frames that hold the specimen, and displacing one frame horizontally with respect to the other at a constant rate of shear deformation and measuring the shearing force and horizontal displacements as the specimen is sheared. Shearing is applied slowly enough to allow excess pore pressures to dissipate by drainage so that effective stresses are equal to total stresses. Direct shear tests are used in earthworks and foundation engineering for the determination of the effective shear strength of soils	Published by CEN as a TS but not published by BSI as a DD

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DD CEN ISO/TS 22476–11: 2006	Geotechnical investigation and testing – Field testing – Part 11: Flat dilatometer test	Site investigations, Field testing, Soil testing, Soil mechanics, Soil profile, Penetration tests, Physical property measurement, Soil strength tests, Deformation, Construction operations, Dilatometry	Published by CEN as a TS and by BSI as a DD in 2006
BS EN ISO 22476–12: 2009	Geotechnical investigation and testing – Field testing – Part 12: Mechanical cone penetration test (CPTM)	Site investigations, Field testing, Soil testing, Soil mechanics, Soils, Soil profile, Penetration tests, Physical property measurement, Soil strength tests, Deformation, Soil-testing equipment	Current BS
(BS EN ISO) 22476–13	Geotechnical investigation and testing – Field testing – Part 13: Plate loading test	No details yet available	Standard under development
(BS EN ISO) 22477–1	Geotechnical investigation and testing – Testing of geotechnical structures – Part 1: Pile load test by static axially loaded compression test	No details yet available	Standard under development
(BS EN ISO) 22477–2	Geotechnical investigation and testing – Testing of geotechnical structures – Part 2: Pile load test by static axially loaded tension test	No details yet available	Standard under development
(BS EN ISO) 22477–3	Geotechnical investigation and testing – Testing of geotechnical structures – Part 3: Pile load test by static transversely loaded tension test	No details yet available	Standard under development
(BS EN ISO) 22477–4	Geotechnical investigation and testing – Testing of geotechnical structures – Part 4: Pile load test by dynamic axially loaded compression test	No details yet available	Standard under development
(BS EN ISO) 22477–5	Geotechnical investigation and testing – Testing of geotechnical structures – Part 5: Testing of anchorages	Soil mechanics, Testing, Structures, Load measurement, Anchorages, Temporary, Permanent, Design, Installation, Site investigations, Anchors, Tendons, Grouting, Corrosion protection, Corrosion-resistant materials, Sleeves (mechanical components), Inspection, Loading	Standard under development. Expected 2012
(BS EN ISO) 22477–6	Geotechnical investigation and testing – Testing of geotechnical structures – Part 6: Testing of nailing	No details yet available	Standard under development
(BS EN ISO) 22477–7	Geotechnical investigation and testing – Testing of geotechnical structures – Part 7: Testing of reinforced fill	No details yet available	Standard under development

Table 10.3 Codes and standards for geotechnical investigations and testing

Part 1 is intended to be applied to the geotechnical aspects of the design of buildings and civil engineering works and is concerned with the requirements for strength, stability, serviceability and durability of structures. Part 2 provides the requirements for the performance and evaluation of field and laboratory testing.

The UK National Annex to EN 1997–1 lists the following BSI publications as providing complementary, non-conflicting information: BS 1377, BS 5930, BS 6031, BS 8002, BS 8004, BS 8008, BS 8081 and PD 6694–1. PD 6694–1: *Recommendations for the Design of Structures Subject to Traffic Loading to BS EN 1997–1:2004*, was published by BSI in 2011 as a PD and and hence is not to be regarded as a British Standard. Another Eurocode part that is relevant to geotechnical design is Part 5 of EN 1993: *Design of steel structures: Piling.*

Many of the previous British Standards, such as BS 8002 and BS 8004, that have been superseded by EN 1997–1, were descriptive documents that have a broader scope than EN 1997, containing not only requirements for design aspects but also guidance on construction operations, construction materials, occupational safety and legislation, and including a bibliography as shown by the keywords for these standards in **Table 10.4**.

Reference	Title	Descriptors/scope	Comment
BS 6031:2009	Code of practice for earthworks	Earthworks, Land retention works, Construction engineering works, Design, Management, Risk assessment, Occupational safety, Site investigations, Soils, Classification systems, Soil mechanics, Structural design, Stability, Mathematical calculations, Design calculations, Shear strength, Embankments, Excavations, Trenches, Excavating, Landscaping, Drainage, Surface-water drainage, Groundwater drainage, Roads, Maintenance, Inspection, Construction equipment	Current BS, providing non-conflicting complementary information (NCCI) for BS EN 1997–1:2004 of August 2010
BS 8002:1994	Code of practice for earth-retaining structures	Retaining structures, Earthworks, Land retention works, Retaining walls, Design, Structural failure, Structural design, Loading, Foundations, Piles, Piling, Corrosion, Cofferdams, Embankments, Water retention and flow works, Maritime structures, Drainage, Bibliography	Superseded and withdrawn. Replaced by BS EN 1997–1:2004
BS 8004:1986	Code of practice for foundations	Design and construction of foundations for the normal range of buildings and engineering structures, excluding foundations for special structures. General design, shallow, deep and subaqueous foundations, cofferdams and caissons, pile foundations; guidance on geotechnical processes; also tide work, underwater concreting, diving and site preparation for foundation work. Durability of timber, metal and concrete structures. A special reference is made to safety precautions	Superseded and withdrawn. Replaced by BS EN 1997–1:2004
BS 8006–1:2010	Code of practice for strengthened/reinforced soils and other fills	Soil mechanics, Reinforced materials, Soils, Reinforcement, Reinforcing materials, Construction materials, Design, Plastic analysis, Structural design, Performance, Life (durability), Loading, Factor of safety, Stability, Dimensions, Embankments, Earth fills, Foundations, Walls, Retaining walls, Retaining structures, Earthworks, Land retention works, Maintenance, Soil testing, Soil strength tests, Tensile strength, Fasteners	Replaces BS 8006:1995 This standard should be read in conjunction with BS EN 1997–1:2004, UK National Annex (NA) to BS EN 1997–1:2004 and BS EN 14475:2006
BS 8081: 1989	Code of practice for ground anchorages	Anchorages, Structural members, Foundations, Structural design, Structural systems, Design, Construction systems, Wall anchors, Construction systems parts, Soils, Site investigations, Bolts, Rocks, Stress analysis, Corrosion, Corrosion protection, Tendons, Safety measures, Approval testing, Acceptance (approval), Maintenance, Grouting, Rock bolts	Current BS. Partially superseded by BS EN 1957, which is currently being revised
BS 8103–1:1995	Structural design of low-rise buildings – Part 1: Code of practice for stability, site investigation, foundations and ground floor slabs for housing	Buildings, Housing, Structural design, Design, Structural systems, Masonry work, Stability, Site investigations, Foundations, Strip foundations, Footings, Concretes, Slab floors, Dimensions, Shape, Wind loading, Loading, Walls, Load-bearing walls, Structural members	Current BS There is a project under way to revise this standard but a document has yet to be published
CP 2012–1:1974	Code of practice for Foundations for machinery. Foundations for reciprocating machines	Foundations, Structural systems, Structural design, Design, Site investigations, Vibration, Vibration control, Concretes, Dimensions, Fixing, Reinforced concrete, Pile foundations, Vibration dampers, Vibration measurement, Design calculations, Seatings, Construction operations, Grouting, Bibliography, Reciprocating parts	Current BS
BS EN ISO 13793:2001	Thermal performance of buildings. Thermal design of foundations to avoid frost heave	Buildings, Foundations, Thermal behaviour of structures, Thermal design of buildings, Ground movement, Climatic protection, Frost, Frost resistance, Thermal insulation, Thermal resistance, Slab floors, Suspended floors, Construction systems parts, Structural systems	Current BS
BS EN 1997–1:2004 + Corrigendum January 2010	Eurocode 7 Geotechnical design – Part 1: General rules	Soil mechanics, Structural systems, Buildings, Construction engineering works, Structural design, Construction operations, Foundations, Pile foundations, Retaining structures, Embankments, Subsoil, Anchorages, Mathematical calculations, Design calculations, Site investigations, Stability	Current BS The UK National Annex 2007 to BS EN 1997–1:2004 + Corrigendum No. 1 of 1 December 2007 are to be read with this standard

BS EN 1997–2:2007 + Corrigendum October 2010	Eurocode 7 Geotechnical design – Part 2: Ground investigation and testing	Soil mechanics, Structural systems, Buildings, Construction engineering works, Structural design, Site investigations, Soil testing, Soil sampling, Soils, Rocks, Ground water, Soil surveys, Laboratory testing, Field testing, Soil classification tests, Mechanical testing, Physical property measurement, Chemical analysis and testing	Current BS. The UK National Annex 2009 to BS EN 1997–2:2007 is to be read with this standard
BS EN 1993–5:2007 + Corrigendum August 2009	Eurocode 3 Design of steel structures – Part 5: Piling	Steels, Buildings, Structures, Structural systems, Construction engineering works, Structural design, Piling, Piles, Pile driving, Foundations, Sheet materials, Walls, Retaining walls, Anchorages, Mathematical calculations, Loading, Verification	Current BS. The UK National Annex 2009 to BS EN 1993–5:2007 is to be read with this standard
PD 6694–1: 2011	Recommendations for the design of structures subject to traffic loading to BS EN 1997–1:2004	Bridges, Construction engineering works, Traffic, Traffic control, Traffic flow, Spreaders, Foundations, Pile foundations, Retaining walls, Retaining structures	Current PD, not to be regarded as a British Standard

Table 10.4 Codes and standards for geotechnical design

In view of this, and in order not to lose this information, the existing BSs for geotechnical design either have been or will be revised to provide 'residual' documents with non-conflicting complementary information in support of BS EN 1997. Until such time as 'residual' documents are prepared, the Eurocode takes precedence.

The UK National Annex of EN 1997–1 notes that it does not cover the design and execution of reinforced soil structures. It states that, in the UK, the design and execution of reinforced fill structures and soil nailing should be carried out in accordance with BS 8006, BS EN 14475 and prEN 14490 (since superseded by BS EN 14490) and the partial factors set out in BS 8006 should not be replaced by similar factors from Eurocode 7. prEN is an acronym for pre-European norm, in other words, a Draft European Standard.

Another standard for geotechnical design published by BSI but prepared by ISO is BS EN ISO 13793 *Thermal Performance of Buildings. Design of Foundations to Avoid Frost Heave*. This is referred to in the application rule 6.4(3) of EN 1997–1.

10.9.4 Codes and standards for geotechnical construction

The British Standards concerned with the construction aspects of geotechnical engineering, or, to use the CEN terminology, the execution of geotechnical work, are listed in **Table 10.5**. The first standards listed in this table are those that have been prepared by BSI and not by CEN, namely:

- BS 8008 *Safety Precautions and Procedures for the Construction and Descent of Machine-bored Shafts for Piling and other Purposes*;
- BS 8102 *Code of Practice for Protection of Structures against Water from the Ground*.

CEN has recently prepared a comprehensive range of standards for the construction aspects of geotechnical engineering, all with the umbrella title *Execution of Special Geotechnical Works*. These standards cover more than the construction aspects covered by the superseded British Standards BS 8002 and 8004 and, except for the standard for soil nailing, have all been published by BSI as British Standards with the prefix BS EN. These 12 standards are:

- BS EN 1536 *Bored Piles*;
- BS EN 1537 *Ground Anchors*;
- BS EN 1538 *Diaphragm Walls*;
- BS EN 12063 *Sheet Pile Walls*;
- BS EN 12699 *Displacement Piles*;
- BS EN 12715 *Grouting*;
- BS EN 12716 *Jet Grouting*;
- BS EN 14199 *Micropiles*;
- BS EN 14475 *Reinforced Fill*;
- BS EN 14679 *Deep Mixing*;
- BS EN 14731 *Ground Treatment by Deep Vibration*;
- BS EN 15237 *Vertical Drainage.*

As noted in **Table 10.4**, BS EN 1537, covering the construction aspects of ground anchorages, partially replaces BS 8081; and BS EN 14475, covering the construction aspects of reinforced fill, partly replaces BS 8006. The execution standards EN 1536, EN 1537, EN 12063, EN 12699 and EN 14199 are referred to in principles and application rules in Sections 7 and 8 of EN 1997–1 on *Pile foundations and Anchorages*.

10.9.5 Codes and standards for geotechnical materials

Three CEN standards published by BSI that have been prepared for the specific use of certain materials in geotechnical engineering are listed in **Table 10.6** and are:

- BS EN 12794 *Precast Concrete Products*;
- BS EN ISO 13433 *Geosynthetics. Dynamic Perforation Test (Cone Drop Test)*;

Reference	Title	Abstract/Descriptors/Keywords	Comment
BS 8008:1996 + A1:2008	Safety precautions and procedures for the construction and descent of machine-bored shafts for piling and other purposes	Safety engineering, Occupational safety, Safety measures, Emergency measures, Accident prevention, Rescue, Rescue equipment, Underground, Lighting systems, Safety harnesses, Breathing apparatus, Construction equipment, Lifting equipment, Hoists, Passenger hoists, Air compressors, Personnel, Construction workers, Training, Communication equipment, Hazards, Working conditions (physical), Industrial accidents, Piling, Site investigations, Construction operations, Foundations, Pile foundations, Trenches, Access, Barriers, Guard rails, Restraint systems (protective), Hoisting cages, Dangerous materials, Toxic materials, Earthworks	Current BS. The status of this standard is 'Project Underway' which means that there is a project underway to revise it but a document has yet to be published
BS 8102:2009	Code of practice for protection of structures against water from the ground	Protection against water from the ground, Underground structures, Structural design, Buildings, Basements, Waterproofing materials, Waterproof materials, Vapour barriers, Ground-water drainage, Drainage, Grades (quality), Tanking, Site investigations, Render, Construction materials, Weather protection systems, Risk assessment	Current BS
BS EN 1536:2010	Execution of special geotechnical works – Bored piles	Piles, Foundations, Structural systems, Piling, Circular shape, Metal sections, Construction operations, Pile driving, Soil mechanics, Site investigations, Concretes, Grouting, Portland cement, Blast-furnace cement, Cements, Aggregates, Bentonite, Polymers, Steels, Design, Tolerances (measurement), Excavating, Excavations, Boring, Reinforcement, Loading, Records (documents)	Current BS
BS EN 1537:2000	Execution of special geotechnical works – Ground anchors	Soil mechanics, Anchorages, Temporary, Permanent, Design, Installation, Site investigations, Anchors, Tendons, Steels, Grouting, Corrosion protection, Corrosion-resistant materials, Sleeves (mechanical components), Approval testing, Inspection, Mechanical testing, Electrical testing, Electrical resistance, Resistance measurement, Loading, Technical data sheets, Records (documents)	Current BS. Has superseded parts of BS 8081:1989 dealing with anchorages. Currently being revised
BS EN 1538:2010	Execution of special geotechnical works – Diaphragm walls	Soil mechanics, Walls, Construction systems parts, Retaining walls, Concretes, Precast concrete, Reinforced materials, Slurries, Mortars, Bentonite, Suspensions (chemical), Design, Panels, Dimensional tolerances, Non-load-bearing walls, Construction operations, Excavating, Quality control, Technical documents, Technical data sheets	Current BS
BS EN 12063:1999	Execution of special geotechnical work – Sheet pile walls	Foundations, Pile foundations, Sheet-pile foundations, Structural members, Prefabricated parts, Walls, Permanent, Temporary, Construction systems parts, Anchorages, Steels, Wood, Information, Instructions for use, Site investigations, Design, Selection, Construction materials, Construction operations, Welding, Welded joints, Joints, Dimensions, Dimensional tolerances, Angles (geometry), Tolerances of position, Occupational safety, Safety measures, Excavating, Drainage, Performance testing, Records (documents), Storage, Lifting, Installation, Waterproof materials, Equations, Pressure measurement (fluids), Defects	Current BS
BS EN 12699:2001	Execution of special geotechnical work – Displacement piles	Piling, Soil mechanics, Structural systems, Excavations, Excavating, Piles, Steels, Cast iron, Concretes, Wood, Grouting, Site investigations, Design, Pile driving, Construction operations	Current BS
BS EN 12715:2000	Execution of special geotechnical work – Grouting	Soil mechanics, Grouting, Foundations, Site investigations, Design, Stabilised soils, Soils, Construction materials, Safety measures	Current BS
BS EN 12716:2001	Execution of special geotechnical works – Jet grouting	Soil mechanics, Grouting, Foundations, Site investigations, Design, Mechanical testing	Current BS
BS EN 14199:2005	Execution of special geotechnical works – Micropiles	Soil mechanics, Piles, Working load limit, Structures, Foundations, Transfer processes (physics), Loading, Structural design, Construction operations, Construction systems, Reinforcement, Soil settlement, Stability	Current BS

BS EN 14475:2006	Execution of special geotechnical works – Reinforced fill	Soil mechanics, Earthworks, Earth fills, Reinforced materials, Soils, Reinforcement, Reinforcing materials, Construction materials, Design, Structural design, Performance, Life (durability), Loading, Stability, Dimensions, Embankments, Foundations, Soil testing, Tensile strength, Retaining structures	Partially replaces BS 8006:1995
BS EN 14679:2005	Execution of special geotechnical works – Deep mixing	Structural systems, Construction operations, Soil mechanics, Excavations, Mixing, Slurries, Binding agents, Water, Particulate materials, Mixers, Deep foundations	Current BS
BS EN 14731:2005	Execution of special geotechnical works – Ground treatment by deep vibration	Soil mechanics, Construction operations, Compacting, Site investigations, Vibrators (compacting), Compacting equipment, Vibration, Foundations, Structural systems	Current BS
BS EN 14490:2010	Execution of special geotechnical works – Soil nailing	Soil mechanics, Stability, Earthworks, Land retention works, Excavations, Embankments, Tunnels, Stabilised soils, Reinforced materials, Reinforcement, Nailing, Reinforcing materials, Design, Construction operations, Loading, Structural design, Durability, Mechanical testing	Partially replaces BS 8006:1995. Supersedes the prEN version referred to in the UK NA to EN 1997–1 to be used with BS 8006
BS EN 15237:2007	Execution of special geotechnical works – Vertical drainage	Soil mechanics, Vertical drainage, Groundwater drainage, Surface-water drainage, Soils, Soil settlement, Soil improvement, Sand, Prefabricated parts, Geotextiles, Stability, Structural design, Inspection, Construction engineering works	Current BS

Table 10.5 Codes and standards for the execution of geotechnical work

Reference	Title	Abstract/descriptors/keywords	Comment
BS EN 12794:2005	Precast concrete products. Foundation piles	Concretes, Precast concrete, Reinforced concrete, Pile foundations, Pressurised concrete, Performance, Conformity, Marking, Testing conditions	Current BS
BS EN ISO 13433:2006	Geosynthetics. Dynamic perforation test (cone drop test)	Geosynthetic materials, Geotextiles, Textile testing, Fabric testing, Perforating tests, Impact testing, Penetration tests, Drop tests, Test specimens, Specimen preparation, Testing conditions, Test equipment	Current BS. Replaces BS EN 918:1996
BS EN ISO 13437:1998	Geotextiles and geotextile-related products. Method for installing and extracting samples in soil, and testing specimens in laboratory	Geotextiles, Textile products, Textiles, Textile testing, Fabric testing, Performance testing, Durability, Life (durability), Endurance testing, Environmental testing, Environment (working), Test specimens, Installation, Sampling methods, Mechanical testing, Testing conditions, Control samples, Specimen preparation, Visual inspection (testing), Forms (paper), Laboratory testing	Current BS
BS EN ISO 13438:2004	Geotextiles and geotextile-related products. Screening test method for determining the resistance to oxidation	Geotextiles, Textile products, Oxidation resistance, Polypropylene, Polyethylene, Chemical analysis and testing, Screening (sizing)	Current BS

Table 10.6 Codes and standards for materials used in geotechnical engineering

- BS EN ISO 13437 *Geotextiles and Geotextile Products. Method for Installing and Extracting Samples in Soil, and Testing Specimens in Laboratory*.

10.10 Conclusions

Geotechnical design in the UK has moved into a new era, dominated by codes and standards prepared by CEN and published by BSI. Those involved in geotechnical design need to know the new codes and standards, in particular Eurocode 7 and its associated standards, and how to apply them. The move from the traditional permissible-stress design method with overall factors of safety, to the limit state design method with partial factors, presents challenges and opportunities for the geotechnical designer. The special features of soil have influenced the codes and standards for geotechnical design, causing the geotechnical design components to differ from those for structural design. The components of geotechnical design, as embodied in Eurocode 7, are explained and have been presented in the form of the geotechnical design triangle. The geotechnical design triangle is presented as a concept to clarify and improve understanding of the geotechnical design process.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

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Chapter 11
Sustainable geotechnics

Heleni Pantelidou Arup Geotechnics, London, UK
Duncan Nicholson Arup Geotechnics, London, UK
Asim Gaba Arup Geotechnics, London, UK

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Sustainability is undoubtedly the biggest challenge facing engineering in the 21st century. Natural resources are fast depleting, traditional energy production methods are becoming too expensive (post peak oil, etc.), water is turning into a precious and scarce commodity, climatic conditions are changing – threatening existing infrastructure and requiring mitigation measures. Geotechnical engineers need to demonstrate their capacity to make geotechnical practice sustainable and their ability to deliver developments which satisfy all three elements of sustainability: environmental conservation, social betterment and economic improvement.

This chapter discusses the relevance of the sustainability objectives to geotechnical engineering, and how geotechnical engineers may engage with the objectives during the design and construction process.

11.1 Introduction

Sustainability is potentially the widest ranging and most engaging issue for engineering development in the 21st century. Engineers have a key role to play in ensuring that present and future development is carried out according to sound sustainability principles.

According to the Brundtland (WCED, 1987) definition:

> Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs.

The emphasis of the Brundtland definition is that sustainable development must include all three elements – environmental, social and economic – also known as the 'triple bottom line'.

Sustainable engineering practice ultimately aims to ensure that the interface between the natural and engineered environment is harmonious and good design is an essential part of it.

Sustainability is a far-reaching subject, encompassing issues such as worldwide transport, economic and political stability, climate change and energy security, which cannot be fully addressed here. This chapter explores the sustainability framework for civil engineering and how it is interpreted in geotechnical engineering. The points discussed here are mainly UK-centric, focusing on the sustainability aspects that directly involve geotechnical engineering.

11.2 Sustainability objectives – background

Within the triple bottom line context, but more specifically focused on civil engineering and geotechnics, seven key sustainability objectives have been identified, as shown in **Table 11.1** (an evolution of the Buildings Sustainable Design Strategy (Arup, 2010)), setting the framework for sustainable development.

Triple bottom line sustainability objectives		Environmental	Social	Economic
1	Energy efficiency and carbon reduction	•	•	•
2	Materials and waste reduction	•	•	•
3	Maintained natural water cycle and enhanced aquatic environment	•	•	•
4	Climate change adaptation and resilience		•	•
5	Effective land use and management	•	•	•
6	Economic viability and whole-life cost		•	•
7	Positive contribution to society		•	

Table 11.1 Sustainability objectives in civil engineering and geotechnics

11.2.1 Energy efficiency and carbon reduction

Mackay (2008) gives three reasons why our addiction to fossil fuels is unsustainable:

> First, easily accessible fossil fuels will at some point run out, so we'll eventually have to get our energy from someplace else. Second, burning fossil fuels is having a measurable and very-probably dangerous effect on the climate. Avoiding dangerous climate change motivates an immediate change from our current use of fossil fuels. Third, even if we don't care about climate change, a drastic reduction in Britain's fossil fuel consumption would seem a wise move if we care about security of supply: continued rapid use of the North Sea oil and gas reserves will otherwise soon force fossil-addicted Britain to depend on imports.

Embodied energy (EE) and embodied carbon (ECO_2) are the two most commonly used indicators for quantifying the environmental impact of a structure. Hammond and Jones (2008) define EE as the total primary energy consumed during the lifetime of a product. Ideally, the boundaries would be set from the extraction of raw materials (including fuels) to the end of the product's lifetime (including energy from manufacturing and the manufacturing of capital equipment, transport, heating and lighting of factories). ECO_2 is an expression of the same environmental impact but in terms of CO_2 emitted in the environment rather than energy consumed.

Energy efficiency also has social implications; society will only be able to function with a continuity of energy supply. It also has economic consequences, as the cost of energy is likely to rise. Energy efficiency introduces the fundamental conundrum: build new (infra)structures with lower operational costs, or adapt existing (infra)structures and accept higher operational costs. New constructions often generate additional energy use, as in the case of a new highway.

11.2.2 Materials and waste reduction

Producing traditional construction materials (e.g. concrete and steel) is energy intensive and resource hungry. Natural resources are depleting and there is a need to find more sustainable ways of producing construction materials. The emphasis should be on re-using materials, recycling aggregates, incorporating cement replacement additives and returning to more sustainable forms of construction materials such as wood (from sustainably managed sources). Responsible sourcing of materials (RSM) is also an important consideration in construction, with both social and whole-life cost implications (BES 6001, 2009).

A significant by-product of the construction industry is waste. Waste generated by construction, demolition, excavation and refurbishment in the UK amounts to over 100 million tonnes per year, a third of the country's total waste production (DEFRA, 2007b). Reducing waste not only benefits the environment, but also makes good business sense. The true costs of waste are not just related to disposal. They include hidden costs such as the purchase of unused materials, handling and processing costs, management and time, lost revenues or potential business liabilities and risks (Foresight, 2009).

Tackling waste has many facets. DEFRA (2007a) introduced the inverted triangle of waste hierarchy: reduce, re-use, recycle – see **Figure 11.1**.

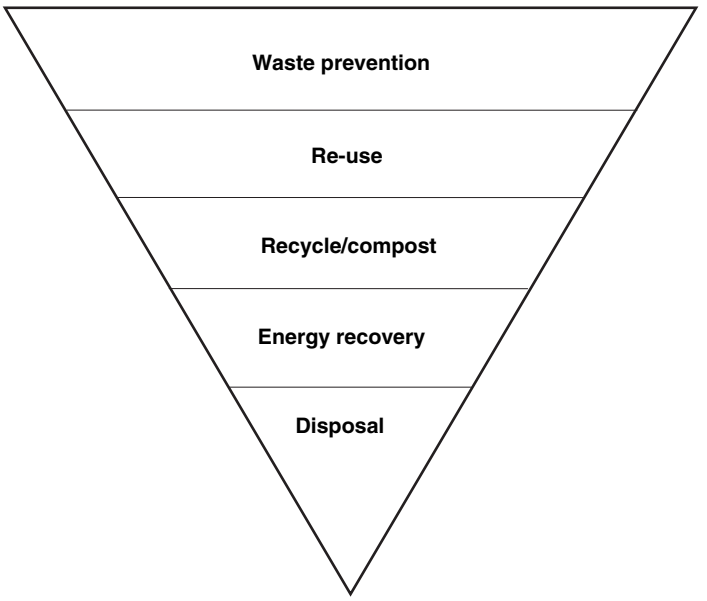


Figure 11.1 Waste hierarchy inverted triangle
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11.2.3 Maintained natural water cycle and enhanced aquatic environment

Water is one of the most important elements of the natural environment. It is a resource that may become less readily available. Water availability is unevenly distributed geographically; in some parts of the world significant investment in energy and infrastructure is required to transport it to where it is needed. Water pollution and shortage of water are key social (and economic) issues – as the recent problems in Northern Ireland (December 2010), Pakistan (August 2010) and Queensland flooding (January 2011) show.

The aquatic environment is a dynamic system and its equilibrium may be easily disrupted by artificial intervention with far-reaching, unanticipated consequences. Planning and managing what impact buildings and infrastructure have on the natural water cycle is paramount. Every engineering project should be actively dealing with:

- safeguarding and enhancing the quantity and quality of water resources in lakes, rivers and groundwater;
- minimising the project's impact on local water systems and their functions;
- minimising the project's impact on the wider water cycle and its dependants;
- valuing water appropriately in optimising project development, construction and operation choices.

The Environment Agency (2009) gives an overview of the existing pressures and future threats on the water environment and proposes a strategy to reduce pressures and improve resilience to climate change.

11.2.4 Climate change adaptation and resilience

Climate change is happening and is increasingly testing the robustness of existing infrastructure and building stock. Climate change adaptation (CCA) is the process of reducing vulnerability to the physical impacts of climate change, such as more severe weather, and long-term changes in temperature, rainfall and sea level rise. The role of CCA is to ensure robustness and longevity of a project under changing climatic conditions. Project design needs to take current climatic conditions and

future changes into consideration. Retrofitting existing infrastructure and building stock needs to take place to accommodate extreme weather events, sea level rise and other direct and indirect consequences of global warming.

The main impacts associated with climate change are likely to be:

- overheating: resulting from average and extreme summer temperatures, high solar irradiance;
- flood risk – coastal, fluvial (river) and pluvial (drainage): resulting from sea-level rise, increased rainfall and groundwater level fluctuation;
- water scarcity: resulting from changes in seasonal precipitation and drought risk, as well as areas with increasing demographics;
- structural failure: resulting from extreme storms;
- erosion and landslip: resulting from increased rainfall frequency and intensity;
- subsidence: resulting from changes in groundwater regime and rainfall;
- loss of green landscape and biodiversity: resulting from changes in seasonal precipitation and drought risk, change in the timing of the seasons and deforestation;
- air quality deterioration: resulting from increased sunshine hours, heat waves, wind speeds and direction, pollution.

Climate change scenarios (CCSs) provide quantitative information from climate models regarding future changes in different aspects of climate and sea-level rise under different scenarios for greenhouse gas emissions. They may be used as a basis for qualitative or quantitative CCA assessments and options appraisals. The UK currently has well developed scenarios issued by the UK Climate Impacts Programme (UKCIP) on behalf of the government. These scenarios are disseminated via the UKCIP website (www.ukcip.org.uk) and have been widely used for academic and commercial consultancy projects. Hacker *et al.* (2005) give helpful guidance on the use of the CCSs on designing a building; they are equally applicable to infrastructure designs.

11.2.5 Effective land use management

Population increase, combined with the legacy of uncontrolled historic industrial development, calls for a sustainable strategy for land use, balancing the needs for housing (and infrastructure), industry and agriculture. The Environment Agency (2005) has estimated that 300 000 hectares of land at over 325 000 sites in England and Wales are potentially affected by contamination from previous industrial land use. Over 250 new sites have been classified as contaminated each year since 2000. Remediation and re-development of brownfield sites with various levels of contamination should become a priority to reduce development on greenfield land, where appropriate and financially viable in whole-life cost terms.

More specifically, within an urban landscape where space and land availability are at a premium, pushing effective land use to its limits by seeking additional space underground in deep basements and tunnels is becoming the solution of choice. Moreover, enhancement or creation of agricultural land nearer the consumers (urban agriculture) may soon become a necessity, challenging the current boundaries between city and rural environments.

11.2.6 Economic viability and whole-life cost

The cost of construction (materials and labour) has been the traditional driver for the development of any civil engineering design and will always be one of the main decision factors for a project. It is the single, overarching indicator of value and benefit for any activity. However, cost should be considered in a holistic manner, taking into account the social and environmental impact relative to the economic impact during the operational life and beyond. Is the cheapest solution the most sustainable one, or is a cost premium sometimes necessary to minimise environmental and societal impacts?

Recent work in the US (Chester and Horvath, 2009) highlights the importance of a holistic approach to the provision of new transport and infrastructure. It puts into context the relative contribution (in carbon and energy terms) of the construction of infrastructure on which the transport operates. As an example, for a given road alignment, an optimised vertical profile which seeks to minimise changes in gradient and associated fuel consumption would yield substantial environmental benefits beyond the construction choices embedded in the construction of the road itself (O'Riordan *et al.*, 2011).

Economic viability must also address legacy issues including maintenance, change of use and decommissioning. It puts whole-life management at the heart of any sustainable engineering project, giving appropriate weight to financial, environmental and societal considerations. The aim is to produce:

- buildings and infrastructure which are fit for the intended purpose;
- durable, low maintenance buildings and infrastructure;
- an infrastructure flexible for successful adaptation beyond its design life;
- minimal social, environmental and economic impacts during construction and operation.

11.2.7 Positive contribution to society

Civil engineering can have a positive influence on people's lives, improving the local environment, economy, etc. However, projects can also have negative impacts to society and these must be explored.

Understanding the societal impacts of an engineering project requires an appreciation of who the stakeholders are and establishing whether the project objectives are consistent with their needs, interests and capacities. The social impact assessment on the various stakeholders, particularly vulnerable groups, includes a study of population characteristics, political,

social and community resources, community and institutional structures, as well as their cultural practices, beliefs and values systems.

Social risks that may affect the success of a project include community severance and displacement; nuisance to neighbours; loss of livelihood; impact on quality of life; loss of trust; loss of community stability; loss of historically and culturally significant places or artefacts.

A project can positively contribute to society by avoiding such risks and ensuring stakeholder satisfaction through early consultation and engagement at the masterplanning and design stages. Government bodies such as the Environment Agency, Olympic Delivery Authority and Transport for London *inter alia* are increasingly adopting policies and appraisal methodologies that encourage good practice approaches to social sustainability.

11.3 Geotechnical sustainability themes

How can geotechnical engineers adapt their traditional practices to this sustainability framework? They need to embrace a combination of imaginative creativity, holistic thinking and common sense. Geotechnical input can contribute to the sustainable design from early stages. The requirement for sustainable geotechnical development brings opportunities and endless new areas of growing interest and business, with sustainability focus for geotechnical engineers. Following the seven-point framework outlined in the previous section, the opportunities for geotechnical engineering contribution to sustainability are outlined below.

11.3.1 Energy and carbon

11.3.1.1 Environmental impact indicators

The first step in reducing the carbon footprint of geotechnical structures is to understand design and construction processes and quantify their energy consumption and contribution to carbon emissions in the environment.

There are established methodologies for assessing the EE and ECO_2 of a structure and these also apply to geotechnical structures. Comparing carbon emissions of different types of foundations, together with a comparison of their cost, can be an effective option selection tool for foundation design.

Appendix J of the NHBC guidance on piled foundations (NHBC, 2010) shows such a comparison for the foundation solution of a typical low rise building: it demonstrates that the traditionally favoured deep trench foundations have a significantly higher carbon footprint than the equivalent piled foundation option.

On a larger scale, Pantelidou *et al.* (2011) carried out a carbon emissions comparison of the various components of a new motorway. They estimated that earthworks (always a significant part of a linear infrastructure project) were only responsible for 6% of the total CO_2 emitted during construction, almost an order of magnitude less than the emissions attributed to the pavement construction (42% of the total emissions, **Figure 11.2**), thus identifying pavement construction as the big carbon emitter and where the focus for carbon reduction should be concentrated.

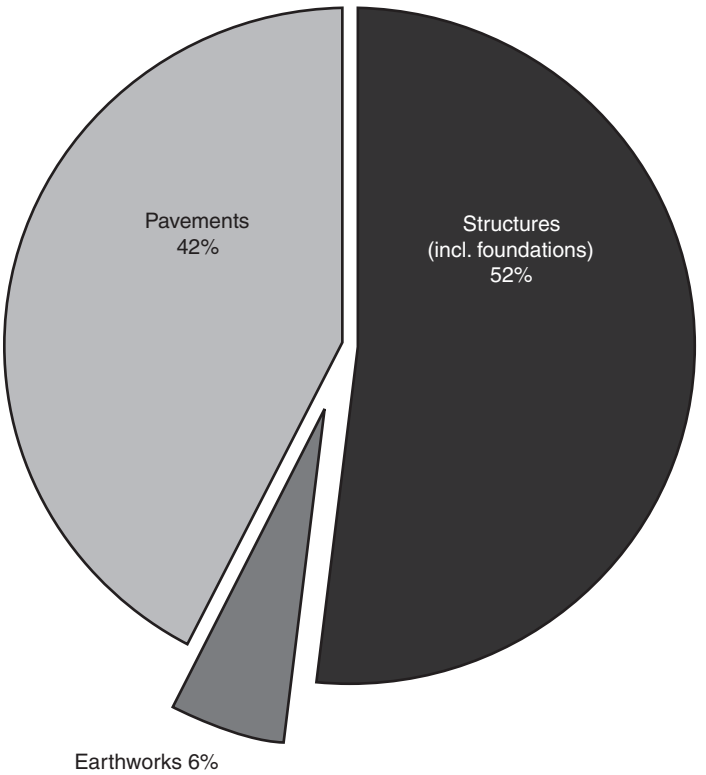


Figure 11.2 Construction and operational CO_2 emissions for a UK motorway project
Data taken from Pantelidou *et al.* (2011)

Within the context of whole-life assessment, construction emissions were compared with the vehicle emissions during the operational life of the motorway, suggesting that construction emissions are equivalent to the first four years of operation.

11.3.1.2 Ground energy

The need for alternative means of energy generation has provided new opportunities for geotechnical engineers. The ground is not only used for supporting the structure, but also for providing energy. Geothermal energy is heat stored in the earth's core. It originates from the radioactive decay of minerals and solar energy absorbed at the surface. The background to the development of geothermal energy is described by Dickenson and Fanelli (2004). Geothermal energy has three main uses: electricity generation, district heating and ground source heat pump (GSHP), described below.

1. *Electricity generation systems.* Conventional geothermal plants use water at temperatures greater than 180°C and binary power plants use water in the range 80–180°C. Electricity generation is mainly associated with the tectonic plate boundaries where fault lines and volcanic activity provide easy access to high temperature ground water in fractured permeable rock masses at relatively shallow depths,

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say 1 km. Extraction and injection wells are used to circulate and reheat the water in the rock mass. Heat exchangers are used to transfer the heat to steam, which then powers electricity generators. The waste heat from the generators may also be used for district heating systems. Within the tectonic plates are areas of rock, e.g. granites, which are undergoing radioactive decay and generate heat. An example of this can be found at Cornwall's granite outcrop in the UK. In these areas, the rock mass permeability may be low and Enhanced (Engineered) Geothermal Systems (EGS) are used to hydraulically fracture and improve the rock mass permeability (Tester, 2007). These systems transfer the heat through heat exchanges to power electrical turbines. The EGSs are currently under development.

2. *District heating systems* use water in the range 30–120°C that is not hot enough to generate electricity, but can be used in a heating ring mains to supply neighbouring buildings. This geological source of heat can be used with boreholes to shallower depths because of the lower temperature requirements. Additional geological sources of heat for district heating are the deep aquifers capped with impermeable insulating layers. In the UK, the Southampton geothermal project is an example of this type of district heating system, using boreholes about 2 km deep (Southampton City Council, 2009).
3. *Ground source heat pump (GSHP) systems* use the low temperature (<30°C) zone to depths of less than 300 m, where the temperatures are controlled by solar energy as well as thermal gradient. In the UK, the average annual ground surface temperature is about 12°C with a thermal gradient of about 3°C per 100 m. The GSHP systems are used to increase the temperatures to between 30°C and 55°C for heating buildings and domestic hot water. Alternatively, the ground water can be used for directly cooling buildings or in conjunction with heat pumps (Banks, 2008). In many shallow systems, the ground is used as a thermal store with heat extracted during the winter and replenished during the summer. The GSHP systems can be classified as:

- *Closed systems* – horizontal loops, vertical boreholes where the heat transfer is by conduction. These systems can also be incorporated into piles, walls and tunnels (Brandl, 2006) and house foundations (NHBC, 2010).
- *Open systems* – ground water is pumped from wells, then through heat exchangers and returned to the ground via injection wells. In this case the circulating water transfers the heat to and from the surrounding ground. This process is known as advection.

11.3.1.3 Carbon capture and storage

It is recognised that we will continue to need fossil fuels as part of a diverse energy mix for some time, but the use of coal and gas must become cleaner. Carbon capture and storage (CCS) is an emerging combination of technologies which could reduce emissions from fossil fuel power stations by as much as 90% (IPCC, 2005).

Supercritical/dense phase CO_2 transportation via pipeline is likely to be preferred as it is relatively stable compared to liquid CO_2. CO_2 can be stored (long term) in geological formations (also known as geo-sequestration). There are three basic types of geological formations that are widespread and which have adequate CO_2 storage potential: deep aquifers, oil and gas reservoirs, and deep, unmineable coal seams. CCS provides new challenges and opportunities for geotechnical engineers, with skills required in areas such as geo-hazard assessment for site locations and engineering of deep underground storage.

11.3.2 Materials and waste

11.3.2.1 Lean design

Geotechnical engineering has evolved using appropriately applied conservatism to counteract the ground variability and uncertainty. A sustainable geotechnical design should aim to eliminate conservatism by optimising the interlinked components of geotechnics on Burland's triangle (Chapter 4 *The geotechnical triangle*): theoretical modelling, experimentation (ground investigation) and empiricism. An efficient geotechnical design makes optimum use of construction materials and hence reduces impact on natural resources – the waste prevention part at the top of the waste hierarchy inverted triangle of **Figure 11.1**.

11.3.2.2 Intelligent material choice

Consideration of the environmental impact and appropriate specification of the construction materials adopted for geotechnical structures is important: choosing between steel and concrete as a structural material, specifying a minimum requirement for recycled content of steel and steel reinforcement, encouraging use of recycled aggregates and cement replacement additives in concrete, and considering earthworks instead of structure.

11.3.2.3 Alternative materials

Twentieth century geotechnical structures employ hard engineering solutions (i.e. concrete and steel). More recently, new techniques and materials have found their way into geotechnics as alternative means of supporting a structure on the ground. This section aims to give a flavour of what are the sustainability implications on the choice of geotechnical materials, but is not intended as the definitive guide on material choice.

Ground improvement linked with reduced foundations' size, and soil reinforcement instead of retaining walls, are examples of 'doing geotechnics differently'. It is currently difficult to evaluate the pros and cons of available techniques, as there is no comprehensive environmental impact comparison between them. Geosynthetics may be good for stabilising fill, but making them out of plastic is resource depleting, energy intensive and environmentally polluting. Similarly, lime, cement or other stabilisation systems are ideal for allowing poor quality soils to

remain *in situ*, thereby avoiding their removal and landfill disposal, but produce high carbon emissions during manufacture (Hughes *et al.*, 2011). Until the benefits and drawbacks are systematically identified and quantified, geotechnical engineers will not be able to make sound, sustainable material choices.

The ever more popular use of tyre bales instead of lightweight fill would be a prime example of turning waste into a resource. Such an example is the lightweight embankment at Brodborough Lake for the A421 Improvement (Kidd *et al.*, 2009).

At Dartford Creek in East London, slope remediation was achieved by encouraging natural depositional processes by means of a soft engineering solution consisting of brushwood mattresses installed on the retreating slope (**Figure 11.3**). It was a revival of a medieval technique for controlling erosion, which was abandoned during Victorian times in favour of new technologies and engineering.

New technologies that can make a difference in dealing with geotechnical materials are always emerging. Accelerated Carbonation Technology (Carbon8, www.c8s.co.uk) may be one such example, where CO_2 becomes a resource instead of waste, with the potential of *in situ* soil improvement or production of alternative materials from waste.

11.3.2.4 Waste reduction

Geotechnical engineering produces its fair share of waste in the construction industry. Unsuitable soil, be it weak or contaminated, is often excavated and disposed into landfills. Reducing construction waste would require re-evaluating the need to excavate the material in the first place: can contamination be contained and treated *in situ*? Can the weak/unstable soil be improved *in situ*? Can waste material produced elsewhere be used as a resource on the project under consideration? For example, soil stabilisation may use by-products (ground granulated blastfurnace slag, pulverised fuel ash, furnace bottom ash) so, whilst the main process may produce CO_2, the use of waste in construction material blurs the boundaries between waste and material use. The earlier example of the tyre bales embankment is a similar case of turning waste into a resource. However, does using waste as a construction material make the preceding processes more acceptable, even if they are consumptive?

Help for using resources more efficiently and strategies and advice on reducing waste to landfill is available from Waste and Resources Action Programme (WRAP, www.wrap.org.uk/construction/).



Figure 11.3 Example of an alternative use of materials: brushwood mattresses for slope remediation at Dartford Creek
From Pantelidou and Short (2008)

11.3.3 Water

Water is an integral part of geotechnical engineering. Changes in groundwater result in ground movements and impact on geotechnical structures.

11.3.3.1 Impact on existing water environment

One of the main considerations for a geotechnical project should be its impact on existing water resources and their users. Water and groundwater at a project site is also part of a wider aquatic system and any changes to it will disturb the balance of that system. **Figure 11.4** shows a groundwater model for the Florence TAV Station, investigating the damming effects that the underground station box has on the regional groundwater flow, within the shallow unconfined sand aquifer, see Hocombe *et al*. (2007).

11.3.3.2 Sustainable drainage systems

To sustain balance in the natural water cycle, it is important to respect natural water processes. Providing large surface areas of hard covering inhibits natural water infiltration into the underlying aquifer.

According to CIRIA (www.ciria.org.uk/suds/) surface water drainage methods that take account of quantity, quality and amenity issues are collectively referred to as Sustainable Drainage Systems (SUDS). These systems are more sustainable than conventional drainage by trying to replicate the natural infiltration systems because they:

- manage runoff flowrates, reducing the impact of urbanisation on flooding;
- protect or enhance water quality;
- are sympathetic to the environmental setting and the needs of the local community;
- provide a habitat for wildlife in urban watercourses;
- encourage natural groundwater recharge (where appropriate).

11.3.3.3 Aquifer storage and recovery

Artificial recharge (AR) is a method of supplementing or replacing natural infiltration into an aquifer by storing water

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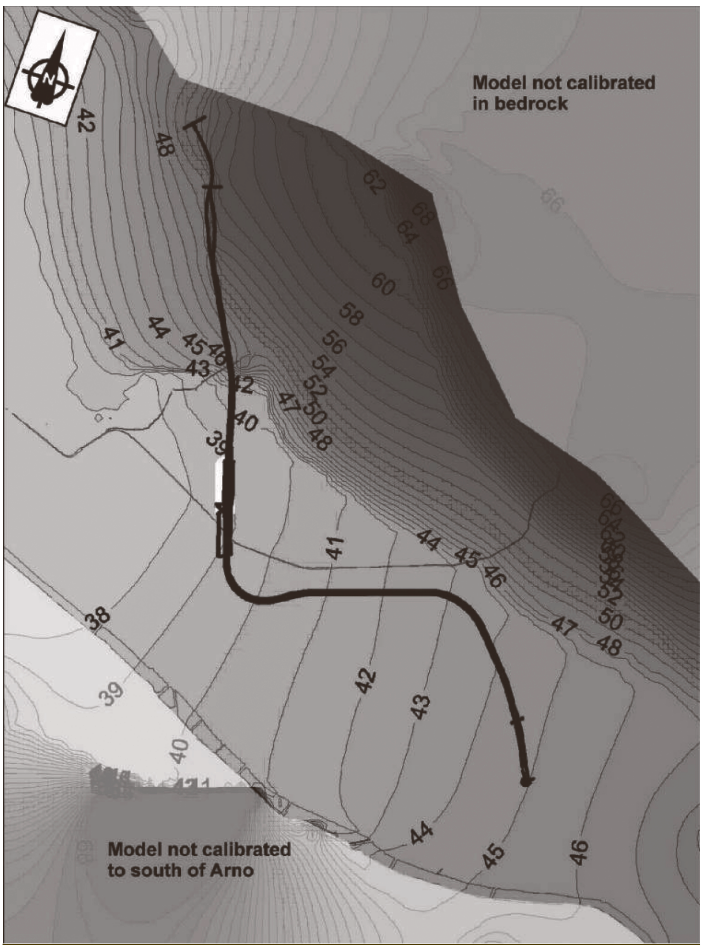


Figure 11.4 Groundwater modelling for the Florence TAV Station. Study of the damming effects of the station box within the shallow unconfined sand aquifer
Reproduced from Hocombe *et al.* (2007)

in an aquifer at times of surplus, for use at times of scarcity (Jones *et al.*, 1998). Aquifer storage and recovery has been proven to be a robust, cost effective and environmentally sustainable water resource management tool. The technique is well established in many parts of the world and for many companies it represents an integral component of their water resources strategy. In the UK, the benefits of aquifer storage recovery and aquifer recharge and recovery (ASR/ARR) are beginning to be realised, particularly by companies where resources are strongly influenced by peak demands. ASR should not be considered as a source of new water but represents a form of water conservation and efficiency. As a result the Environment Agency strongly supports the technique, since it maximises the use of existing licensed water with only marginal environmental impacts. ASR/ARR is consistent with the principles for proper management of water resources and can represent a Best Practical Environmental Option (BPEO).

11.3.3.4 Water in construction

Geotechnical construction invariably uses a considerable amount of water. Sustainable construction practices should aim to minimise water consumption by:

- optimising water usage during construction and operation – grey and black water re-use, recycling, low water use technologies;
- ensuring that the water quality for discharge is appropriate for the receiving aquatic environment.

11.3.3.5 Design for reduced operational energy

It is common practice designing sub-structure systems that rely on permanent dewatering for stability (e.g. numerous basements constructed in central London). Permanent dewatering requires energy consumed to pump water out of the system during the operational life of a structure. A sustainable alternative should be considered at design stage that eliminates the need for permanent pumping.

11.3.4 Climate change

Adaptation of buildings and infrastructure to climate change requires significant geotechnical input: highway and railway assets (embankments and cuttings) are vulnerable to climatic changes, be it from changes of groundwater regime and seasonal water fluctuation or from increase in frequency and magnitude of extreme rainfall events and surface runoffs, leading to surface erosion and eventually failure. Marine and freshwater flood defences are also vulnerable to weather extremes and climatic changes, including increased storm surges and magnitude of tidal ranges.

Figure 11.5 is taken from Shaw *et al.* (2007), a guide on climate change adaptation by design. It makes suggestions for timely monitoring and intervention for ground-related risks attributed to climate change. The main geotechnical requirements identified for climate change adaptation are:

- retrofitting/upgrading existing and construction of new flood defences, coastal and fluvial;
- slope reinforcement and stabilisation to resist increasing surface and ground water fluctuation;
- provision of sustainable water drainage and aquifer recharge schemes;
- retrofitting or replacement of foundations to compensate for increased ground movements due to changes in the groundwater regime.

Furthermore, prolonged dry weather may also become a frequent occurrence, even in the UK, which may create more widespread unsaturated ground conditions. Unsaturated soil mechanics may become more relevant to the geotechnics practice in the future.

Menu of strategies for managing high temperatures

The diagram summarises the range of actions and techniques available to increase adaptive capacity. Detail is given in the text on the proceeding pages.

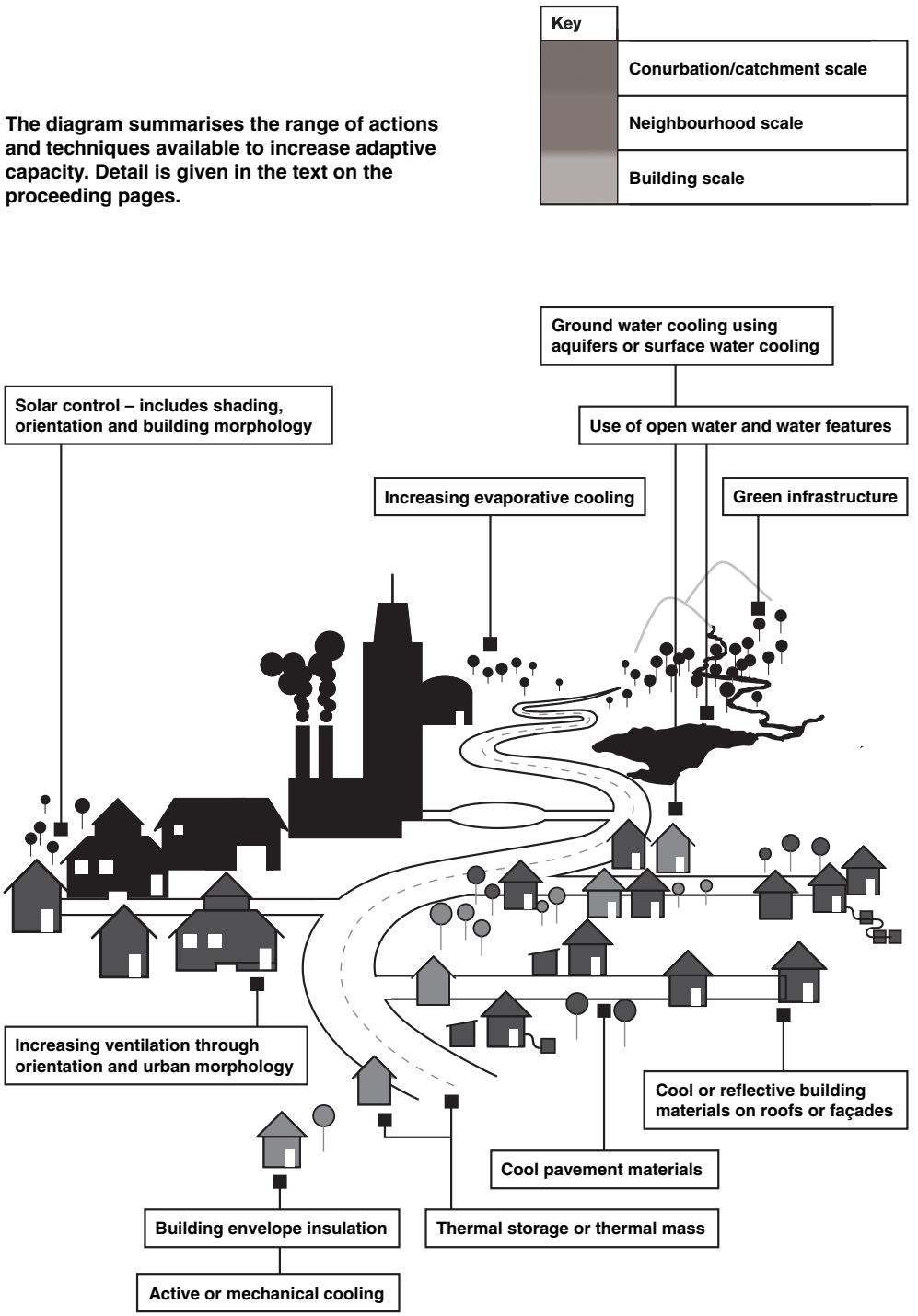


Figure 11.5 Strategies for managing ground conditions to increase adaptive capacity

Reproduced, with permission, from Shaw *et al*. (2007), Climate change adaptation by design: a guide for sustainable communities, Town and Country Planning Association © TCPA. Designed by thomas.matthews (www.thomasmatthews.com)

11.3.5 Effective land use

11.3.5.1 Contaminated land – remediation and containment

Brandl (2008) classified contaminated land remediation methods into three broad categories:

- *geotechnical methods:* including excavation, physical containment, groundwater control and soil treatment (e.g. soil mixing);
- *process-based methods:* using specific physical, chemical or biological processes to remove, immobilise, destroy or modify contaminants;
- *natural remediation methods:* using plants, natural fungi or natural groundwater processes to degrade contaminants.

The Geotechnical methods are more widely applicable and may combine with the process-based and natural remediation methods to a hybrid to achieve optimum remediation. Geotechnical engineers can address how contaminated land is treated (and how it came to be contaminated, in some cases).

CL:AIRE (www.claire.co.uk) are raising awareness of sustainable remediation technologies, with a variety of publications on geo-remediation topics.

11.3.5.2 Underground space in urban environment

In 2007, the world's urban population exceeded the rural population for the first time and the trend for migration from the countryside to the city is set to continue. Lack of urban space is an ever increasing challenge and discussions on strategic use of underground space for sustainable transportation and mobility in urban and rural areas are already taking place (COB, 2007). The ever increasing requirement for underground space presents opportunities to geotechnical engineers in particular, with underground expansion of transport corridors or provision of car parking and large retail and entertainment spaces. Tunnelling as well as multi-level basements require substantial geotechnical input. The technical challenges, substantial investment required and complex logistics involved in underground construction within the urban fabric have to be holistically considered for the successful development and long term viability of the underground urban space.

11.3.6 Whole-life management

Figure 11.6 schematically represents the stages involved in the life of a geotechnical structure. A life cycle assessment of the structure would include consideration of the impacts (environmental, social and economic) that each stage contributes.

Management of railway and highway earthworks and flood defences (river and coastal) assets should take on board whole-life management considerations. Advice can be found in Hooper *et al*. (2009).

Chapman *et al.* (2001) pointed out that the presence of old foundations in the ground is going to present an ever increasing problem and will ultimately inhibit future development on valuable urban sites. Re-using existing foundations makes financial and logistical sense, extending their usefulness beyond their intended life. Guidance on how to establish re-usability of existing foundations and incorporation into a new project is given by Butcher *et al.* (2006) and Chapman *et al.* (2007).

Design of new foundations should ideally take this a step further, by incorporating into their design the potential for re-use after decommissioning.

11.3.7 Societal contribution

Project involvement of geotechnical engineers is usually remote from early stakeholder engagement and social impact

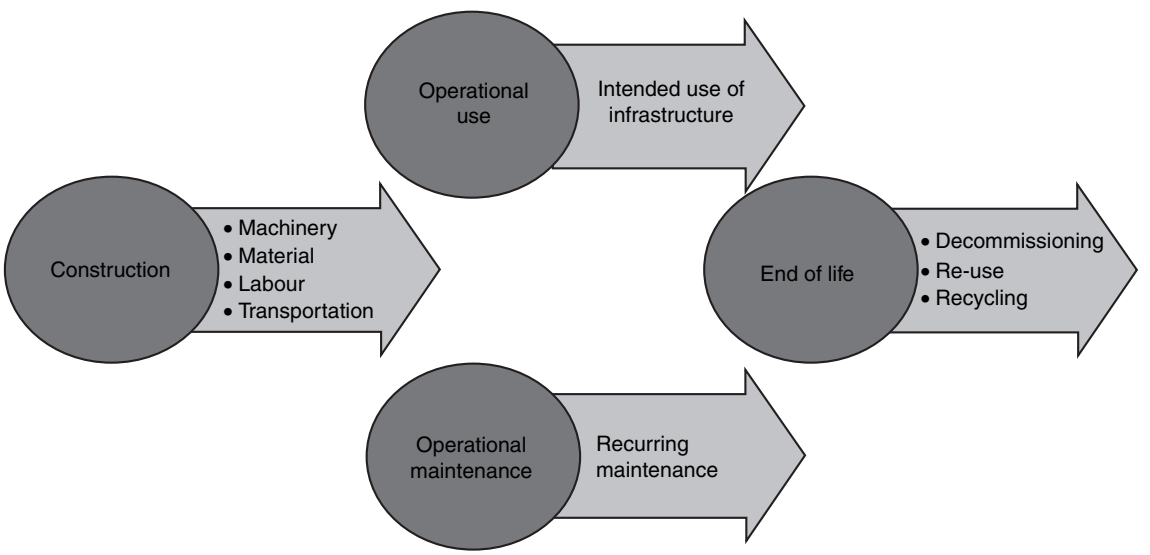


Figure 11.6 Stages involved in the life of a geotechnical structure

assessments. Nonetheless, positive contribution to society is within our brief and is our responsibility.

An integral part of the ground, especially in areas of rich cultural history, is archaeology. Archaeological resources are finite, non-renewable and increasingly dwindling as a result of inconsiderate development. Geotechnical desk studies and specifications must ensure early identification of archaeological issues related to a project. Geotechnical design and procurement should necessitate adoption of appropriate mitigation measures and programme allowances for implementation. Advice from and collaboration with archaeologists is essential for appropriate archaeological guidance, making archaeology an asset of a project rather than a hindrance.

Geotechnical construction may adversely affect the health and wellbeing of stakeholders (neighbours, labour, etc.). Noise, vibration and dust are common nuisances associated with piling and excavation works and must be minimised. This is strongly linked with the health and safety in construction issues discussed in detail in Chapter 8 *Health and safety in geotechnical engineering*.

11.4 Sustainability in geotechnical practice

The majority of the geotechnical sustainability themes discussed can be readily incorporated into current geotechnical practice. Some will need to be developed further; others will require familiarity to geotechnical engineers before they are fully endorsed. In any case, geotechnical engineering can make a big impact on the sustainability of the built environment.

An important issue is the timing of the sustainability considerations during the life of a project. **Figure 11.7** schematically represents the opportunities for adopting sustainable solutions rapidly diminishing as the planning and design progress. The biggest opportunities are given at the planning stage (e.g. the choice of the alignment of a highway); they reduce during design (optimisation of materials and structures) and become quite limited during construction (mainly focusing on energy efficiency of plant and process).

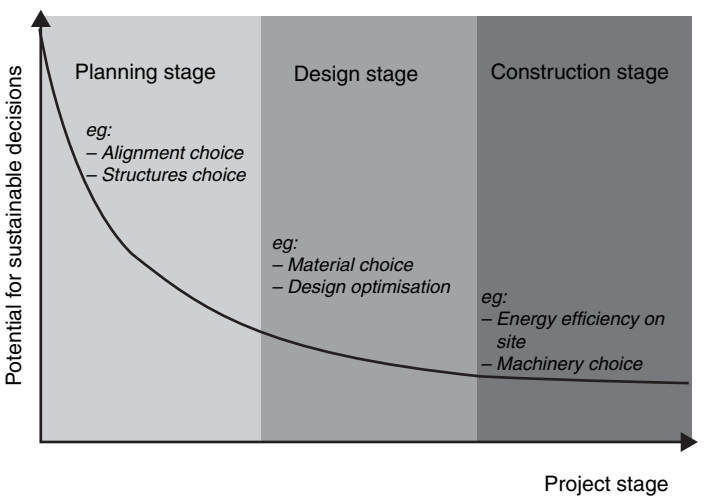


Figure 11.7 Opportunities for adoption of sustainable solutions during the life of a project

Geotechnical Engineers are specialists and as such are usually involved in a project after its design outline is fixed. A conscious shift of geotechnical involvement on earlier stages of the design process will have a positive contribution to the implementation of sustainability elements in the project outline.

The following explores the opportunities for applying the sustainability objectives within the traditional design and construction activities of a geotechnical engineer.

11.4.1 Desk study and ground investigation

A good quality geotechnical desk study for a project initiates prospects for sustainable intervention early on in the design process. A geotechnical desk study recognises key hazards (considered in Chapter 7 *Geotechnical risks and their context for the whole project*) at an early stage, thereby allowing the design to be adapted at optimum cost. It is also an excellent opportunity to identify information such as local sourcing of materials, site and project constraints and potential sustainability implications. It is the time to explore the potential for re-using existing foundations and outline the associated constraints, identify archaeological constraints and opportunities, carry out relevant geohazard assessments, and pull together all information that may assist in making the project more sustainable.

Based on the conclusions and recommendations of the desk study, a comprehensive site investigation should be designed, specified and executed to minimise the ground- and ground water-related uncertainties and inform an efficient geotechnical design.

Section 4 of this manual contains further details on site investigation.

11.4.2 Design

An efficient, lean geotechnical design is akin to a sustainable design. As an industry, geotechnical engineers should make every effort to understand the soil constitutive laws and improve the advanced design and modelling tools to enable design efficiency.

Innovation is an important ingredient of a sustainable design – lateral thinking (outside the box), considering new materials, turning waste to resource.

The design development of a project involves weighing up a number of design options, as discussed in Chapter 9 *Foundation design decisions*. Early geotechnical involvement in the design development can ensure early identification of the ground-related constraints and risks affecting the design solution.

Whole-life considerations should also be included in the design of a project, both financially and in terms of environmental impact.

Also see Sections 5, 6 and 7 of this manual for further detail on geotechnical design.

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11.4.3 Construction

Geotechnical construction aids sustainability by ensuring process efficiency and care for the neighbours and the environment through reduction and efficient management of waste, water and energy, local sourcing of materials and plant, and control of noise, vibration and dust.

11.4.4 Sustainability accreditation schemes

Measuring the social, environmental and economic impact of a structure is important, for comparison, accountability and improvement. Economic impact is usually well documented for any project, although whole-life costing is rarely considered. Environmental impact rating methods are used to assess design performance relative to a predetermined, industry recognised standard. The most commonly used schemes are BREEAM (www.breeam.org), LEED (www.usgbc.org) and GreenStar (www.gbca.org.au/green-star/) (applicable to Buildings) and CEEQUAL (www.ceequal.com) (applicable to infrastructure projects, see **Table 11.2**).

The above accreditation schemes generally address ground-related topics, although some give less emphasis to geotechnical issues than may be appropriate. For example, BREEAM are currently excluding foundations from their appraisal, although this is under development and will be included in their future issues.

There are also other tools that aim to support the decision-making process regarding sustainability issues and how they can be addressed at different spatial scales. Examples include SPeAR® (www.arup.com/environment/), IRM, LCA and Eco-footprinting. These methods do not specify a performance level to be achieved. Rather, they provide an approach to integrating sustainability into a development process and identifying outcomes and constraints. None of them is geotechnically specific, although geotechnical aspects are relevant to a greater or lesser extent.

	Section	Score weighting %	Geotechnical input
1	Project management	10.9	
2	Land use	7.9	•
3	Landscape	7.4	•
4	Ecology and biodiversity	8.8	
5	The historic environment	6.7	•
6	Water resources and the water environment	8.5	•
7	Energy and carbon	9.5	•
8	Material use	9.4	•
9	Waste management	8.4	•
10	Transport	8.1	
11	Effects on neighbours	7.0	•
12	Relations with the local community and other stakeholders	7.4	•

Table 11.2 CEEQUAL's 12 sections

11.5 Summary

This chapter provides a brief overview of the sustainability issues that concern engineering in general and geotechnics in particular. A framework of seven sustainability objectives is proposed: energy efficiency and carbon reduction, materials and waste reduction, maintained natural water cycle and enhanced aquatic environment, climate change adaptation and resilience, effective land use, economic viability and whole-life cost, and positive contribution to society. The geotechnical themes under each of the seven headings are discussed and constraints and opportunities for the geotechnical engineers indicated. The chapter concludes with suggestions on how sustainability elements can be incorporated in the current geotechnical practice: desk study and ground investigation, and design and construction, including reference to current sustainability accreditation schemes.

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11.6.1 Further reading

More detailed advice on SUDS can be found at:

CIRIA's SUDS website www.ciria.org.uk/suds/ (includes the SUDS manual, C697, free to download)
Environment Agency SUDS advice www.environment-agency.gov.uk (search on 'SUDS')
Guidance on soakaway design (BRE365) is available from the Building Research Establishment www.bre.co.uk

11.6.2 Useful websites

UK Climate Impacts Programme (UKCIP); www.ukcip.org.uk
Environment Agency (EA); www.environment-agency.gov.uk
National House-Building Council (NHBC); www.nhbc.co.uk
Waste and Resources Action Programme (WRAP); Construction sector www.wrap.org.uk/construction/
CL:AIRE (Contaminated Land: Applications In Real Environments); www.claire.co.uk
BREEAM® (UK); www.breeam.org
LEED (USA); www.usgbc.org
GreenStar (Australia), applicable to buildings; www.gbca.org.au/green-star/
CEEQUAL (UK), applicable to infrastructure projects; www.ceequal.com

All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

Section 2: Fundamental principles

Section editor: **William Powrie and John B. Burland**

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Chapter 12

Introduction to Section 2

William Powrie University of Southampton, UK
John B. Burland Imperial College London, UK

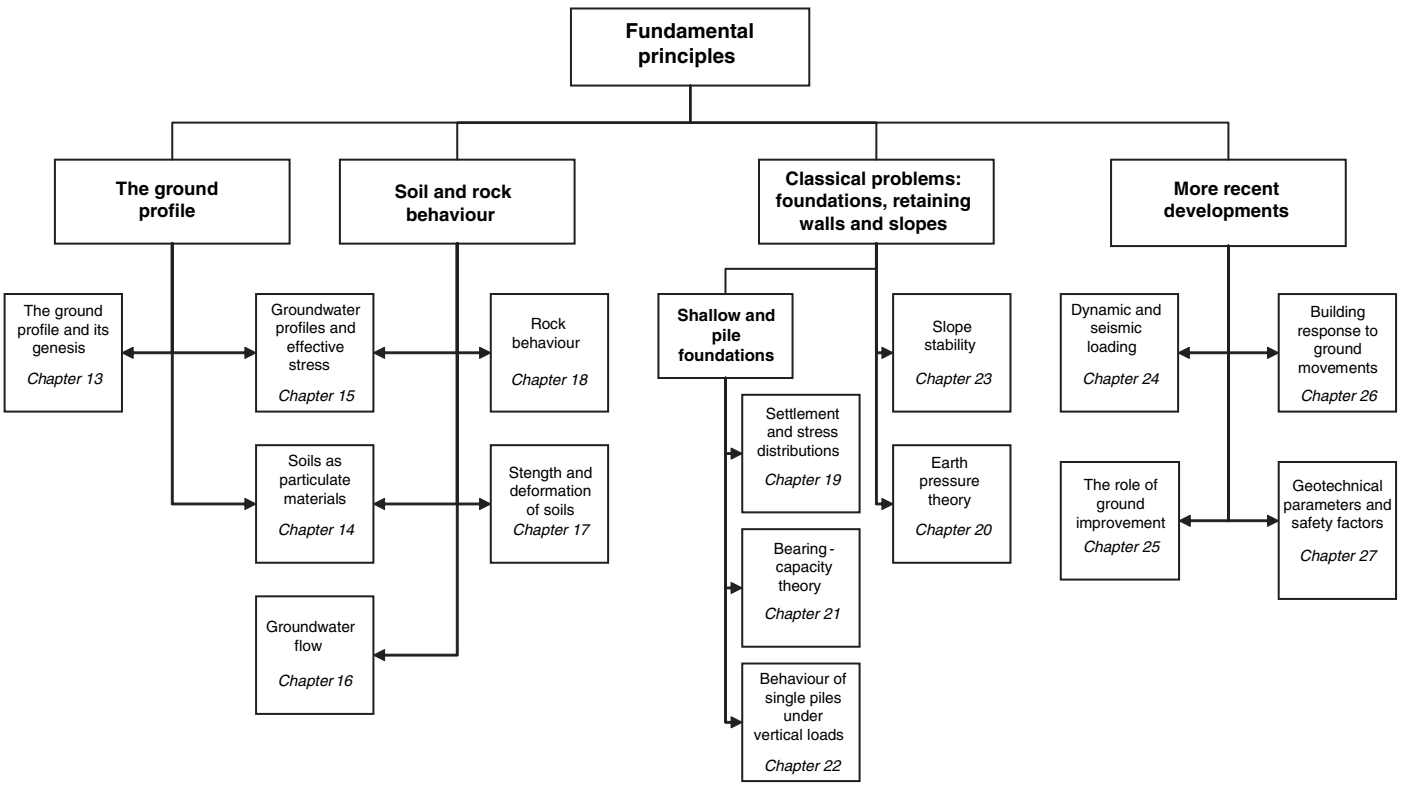


Figure 12.1 Layout of chapters in Section 2

Figure 12.1 outlines the layout and contents of Section 2 *Fundamental principles*.

Section 2 serves as an aide-memoire to the fundamental principles and theories underlying geotechnical engineering, from characterising the ground, through the behaviour of geomaterials, to the basic conceptual models used in the analysis and design of geotechnical engineering structures.

The first six chapters (Chapters 13 *The ground profile and its genesis* to 18 *Rock behaviour*) are concerned with characterising the ground and the fundamental aspects of material behaviour. Chapter 13 *The ground profile and its genesis* describes and discusses the development of the ground profile from a geological perspective, and how this knowledge may be used to inform the development of a conceptual ground model for use in geotechnical engineering. Chapter 14 *Soils as particulate materials* rehearses the factors that make soil, as a particulate material, fundamentally different from many other civil engineering materials such as steel and concrete.

Without groundwater, geotechnical engineering would be much simpler – but also less of a challenge and much less interesting. In many parts of the world, the soils of interest to the geotechnical engineer are saturated or nearly so; thus an

understanding of pore water pressures and groundwater flow is essential. Chapter 15 *Groundwater profiles and effective stresses* describes the estimation of pore water pressure and effective stress profiles in the ground, while Chapter 16 *Groundwater flow* discusses the calculation of flow rates and changes in pore water pressure induced by construction activities such as excavation. Chapters 17 *Strength and deformation behaviour of soils* and 18 *Rock behaviour* summarise the important aspects of soil behaviour and rock behaviour, and the conceptual and mathematical models commonly used to characterise and quantify them.

The next five chapters, Chapters 19 *Settlement and stress distributions* to 23 *Slope stability*, address the underlying theory applied to the three classic problems in geotechnical engineering: foundations (Chapters 19 *Settlement and stress distributions*, 21 *Bearing capacity theory* and 22 *Behaviour of single piles under vertical loads*), retaining structures (Chapter 20 *Earth pressure theory*) and slopes (Chapter 23 *Slope stability*).

The last four chapters, Chapters 24 *Dynamic and seismic loading of soils* to 27 *Geotechnical parameters and safety factors* address a range of topics representing arguably more recent developments in geotechnical engineering. The dynamic and seismic loading of soils (Chapter 24 *Dynamic and seismic loading of soils*) is generally considered more difficult than the static behaviour, but for certain applications and in many parts of the world cannot be ignored. Ground improvement (Chapter 25 *The role of ground improvement*) encompasses a number of techniques that geotechnical engineers use to improve certain properties of the ground. These often require an understanding of chemical or electro-chemical phenomena in addition to the more familiar mechanics, and given the uncertainties involved in applying these to the ground, they represent to many an area still closer to art than science. Understanding and predicting the response of buildings to ground movements (Chapter 26 *Building response to ground movements*) has become increasingly important as more and more ambitious construction projects involving open excavations and tunnels are undertaken in highly built-up areas. This need, combined with the development of computing power enabling complex and sophisticated, three-dimensional numerical analysis to be carried out, has driven a major area of advance in geotechnical knowledge and understanding in the last decade or so. Finally, Chapter 27 *Geotechnical parameters and safety factors* addresses the choice of design parameters and factors of safety. This is a topic of huge importance, about which there is a general lack of coherence and clarity in the geotechnical profession generally, partly because it is difficult to revise customs and codes of practice in a way that keeps pace with developments in geotechnical engineering knowledge and understanding.

Section 2 sets out to cover a huge range of soil and rock mechanics and geotechnical engineering knowledge. Inevitably, it can serve as no more than a reminder for those already familiar with a topic, or an introduction for those who are not. The following bibliography may be helpful to those wishing to find out more about any of the variety of subjects covered.

12.1 Further reading

Engineering geology (Chapter 13)

Waltham, A. C. (2009). *Foundations of Engineering Geology* (3rd Edition). London and New York: Taylor & Francis (Spon Press).

Fundamental soil mechanics (Chapters 14–17 and 19–22)

Atkinson, J. H. (2007). *The Mechanics of Soils and Foundations* (2nd Edition). London and New York: Taylor & Francis (Spon Press).

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Jaeger, J. C., Cook, N. G. W. and Zimmerman, R. W. (2007). *Fundamentals of Rock Mechanics* (4th Edition). Oxford: Wiley-Blackwell.

Slope stability (Chapter 23)

Bromhead, E. N. (1992). *The Stability of Slopes* (2nd Edition). London: Blackie Academic & Professional.

Soil dynamics and seismic loading (Chapter 24)

Verruijt, A. (2010). *An Introduction to Soil Dynamics*. Dordrecht, Heidelberg, London and New York: Springer.

Ground improvement (Chapter 25)

Moseley, M. P. and Kirsch, K. (2004). *Ground Improvement* (2nd Edition). London and New York: Taylor & Francis (Spon Press).

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Chapter 13

The ground profile and its genesis

Michael H. de Freitas Imperial College London and Director of First Steps Ltd, London, UK

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A ground profile, or vertical profile, is the starting place for developing an understanding of the ground for design and analyses. It is a record of the ground at depth and an interpretation of its formation up to the present. The correct identification of these materials and interpretation of their history indicates how ground is likely to respond to changes arising from either construction or natural processes such as rainfall and earthquakes. There is no guarantee that decisions based on a profile that is incorrect can be corrected by subsequent calculations, no matter how sophisticated they may be, because these calculations will be quantifying something that does not exist. Mistakes made with the ground profile delay works and even stop contracts; many result in claims. Profiles enable cross-sections to be constructed and these are the templates upon which all other geotechnical data should be superimposed. This is the only way a relationship can be obtained between geotechnical properties for the materials present and their geological history. Many case histories demonstrate this. Case histories are part of the inherited wisdom of geotechnical engineering and remove any excuse for either ignoring a ground profile or for obtaining one but failing to use it appropriately.

13.1 Overview

A ground profile, also called a vertical profile, describes the ground beneath a particular location. It lists the materials encountered with depth, as would be intersected by a borehole or trial pit, and then uses this and other data to interpret how the profile was formed and how it might behave in response to engineering within it. A ground profile therefore has two basic components: the vertical sequence and its interpretation. The vertical sequence usually starts at ground level and extends to whatever depth is deemed necessary for the geotechnical work for which it was requested. The age of the materials encountered increase with depth, so a profile represents the history of the ground, with the most recent part of that history being at ground level. The correct interpretation of the materials and history of a profile can indicate how the ground is likely to respond to engineering.

There are many forms of profile; all are crucial to decisions taken for geotechnical design, analyses and construction. If the description of the materials with depth is wrong or is missing important data, then decisions based on it will be wrong no matter how correct and accurate subsequent calculations may be. This is because these calculations will be quantifying something that does not exist. Mistakes made with the ground profile can be so seriously wrong that they may result in ground work being stopped and contracts being delayed, sometimes even being abandoned.

Having established the profile at one location it is usually necessary to compare it with profiles at other locations nearby, so that a series of profiles can be related to each other. There are hazards associated with such a seemingly simple task. The outcome of such comparisons is a depiction of the site geology in 3D. Rarely can this be done without considering how each profile has been formed, i.e. its genesis – analysis and construction come from understanding what has happened to the ground in the past.

Thus a ground profile is not just the sequence of soils and rocks but incorporates an understanding of what has happened to these during their formation and subsequent history to the present day. Aspects of this are explained because understanding the formation and history of a profile is one of the key tasks for an engineering geologist to complete. Working with appropriate material descriptions is critical, and using a scale appropriate for relating the ground to the intended engineering project is necessary. The sections, maps and 3D depictions so created become the basis upon which all other geotechnical data should be superimposed; only in this way can the distribution of geotechnical properties reflect the materials present and their geological history (de Freitas, 2009).

The acquisition and use of geotechnical data that has not been related to the materials and history to which it relates, is substandard geotechnical engineering; there are many case histories that demonstrate this. Case histories are part of the inherited wisdom of geotechnical engineering and they remove any excuse for either ignoring a ground profile, or for obtaining it but failing to use it appropriately.

13.2 The ground profile

A basement foundation, 10 m × 15 m to be placed at 7 m below ground level, has been investigated using 5 boreholes, each taken to a depth of 15 m and each used to recover a core of 100 mm diameter. If the core recovery is 100%, the volume of ground actually seen is less than 0.01% of the ground to be excavated and considerably less than that of the material relied upon to carry the foundation loads beneath the excavation for

the next 60 to 100 years. Thus the commercial investment that has facilitated this construction is based on virtually no knowledge. Such simple calculations demonstrate the value that must be placed on these tiny pieces of information and the need for this information to be gathered with care; this subject area is known as ground investigation. Retrieving a sample of ground and describing it so that it reflects the condition of the ground from which it came, are fundamental steps towards creating a ground profile; they require skill, knowledge and experience. However, such a log, by itself, is insufficient; its data has to be used for design and analyses, and the first step in using it this way is to generate a ground profile.

Two logs are shown in **Figure 13.1**, one from London and the other from Hong Kong; they illustrate that a description of a vertical sequence is the starting point for a ground profile. Note that they provide little direct information about either the origin or the history of the ground represented by the log; this comes later. Thus ground profiles require either logs of borehole cores or the boreholes themselves, or their equivalent from a quarry, or trial pit or other excavation. Such a profile refers to the ground beneath the point on the surface of the earth below which it is made, but may also be relevant to locations nearby if the ground does not vary significantly from place to place.

Such logs are used for many purposes, e.g. for the basic design of subsurface structures, for geotechnical analyses, for deciding methods of construction, for the installation of monitoring equipment, for the assessment of material volumes. Thus logs have to provide an accurate record of the ground as it exists, *in situ*. This can be considered the most basic of ground profiles.

Logs are the starting point for developing a fuller ground profile because the information they contain is used to understand how that sequence will respond to the particular changes that engineering work is likely to create. They also indicate aspects of the ground which should be studied further and how the data that arises should be used. These are two subjects that require care and attention, and improve with experience of the observer. Profiles that have to be created from the descriptions of others present different problems from those created by personal observation. The usefulness of a profile created from the records of others depends upon the extent to which those records are a relevant, complete and correct representation of the ground.

The most obvious characteristics to observe are the sequence that is present in the log and the materials it contains, as shown in **Figure 13.1**. The less obvious, yet sometimes crucial, characteristics to note are the fabric of the materials, the structure of their layers and the nature of their boundaries – their interfaces. Even more subtle is recognising what is missing; this is not usually

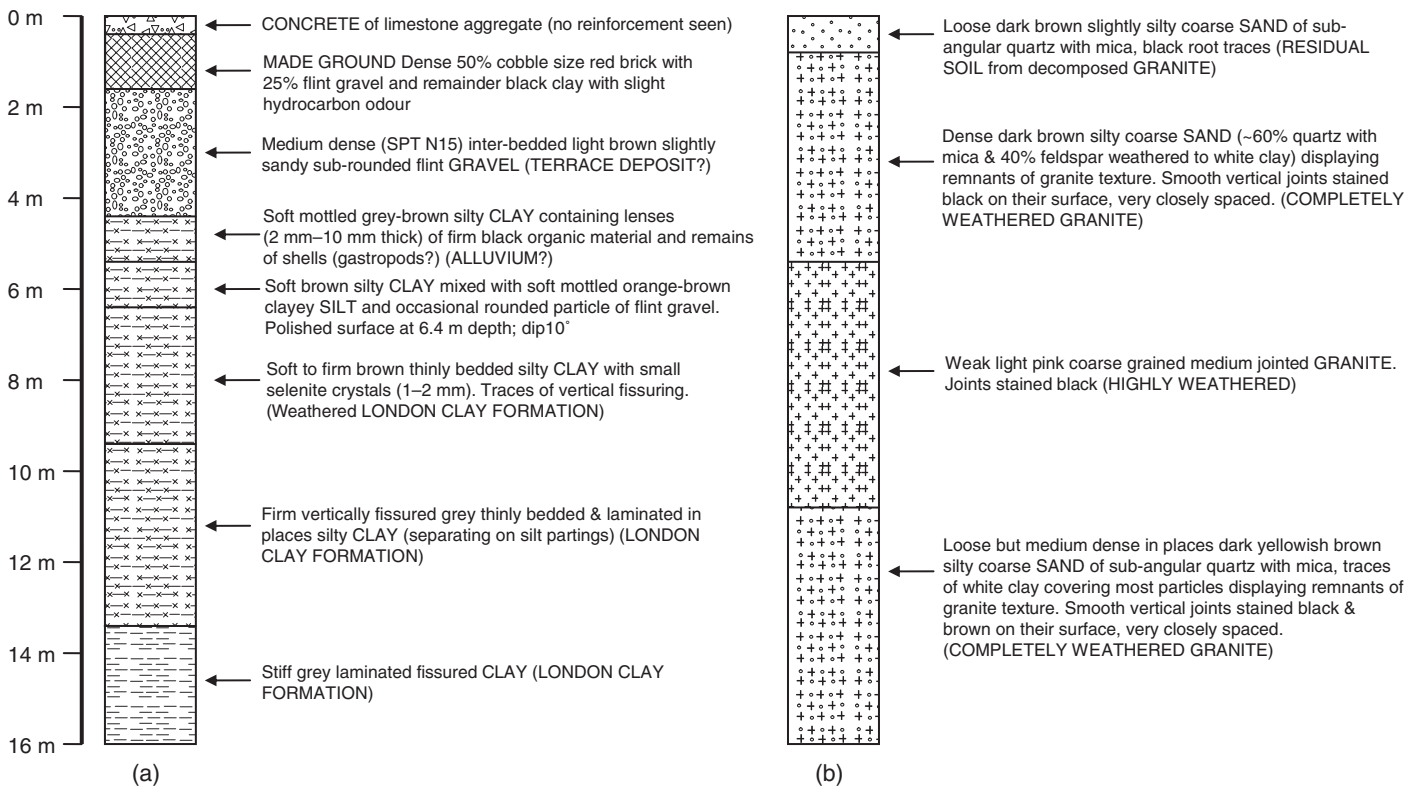


Figure 13.1 Borehole logs from (a) London and (b) Hong Kong illustrating vertical sequences formed mainly by sedimentation and *in situ* weathering of pre-existing granite. Details of recovery, water returns, etc., are omitted

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a problem when a vertical exposure exists, as in the face of a quarry, cliff or trial pit, but becomes a key decision when using recovered samples, especially with borehole core where, during its recovery, material might be lost or its strength or consistency changed. All these aspects are static in character – they are either present or not present in the profile. However, an important component of most profiles is not static but dynamic; it can come and go. Liquids and gasses are the most obvious of these; the free face that exposes a profile and enables it to be observed in the side of a trial pit, shaft, quarry or cliff can affect the presence of liquids and gasses and their location in the ground.

Water will drain towards a free face if its level on the free face side is lower than that in the ground behind the face, so the level of water recorded on the free face may not be that further back in the ground. Air will replace the voids vacated by water and as this is rich in oxygen it will tend to chemically change the ground with which it comes into fresh contact. Black colours become yellowy brown, brown or even whitish, dark browns become light browns, solid wet surfaces become powdery and damp, or even dry. Other gasses such as methane, and other organic rich volatile material, all pertinent to the geotechnical character of the ground, may escape. All these are changes that occur with time but they need not be the only ones to occur. Any excavation, natural or man-made, removes the loads that were on the ground and the ground will respond to their loss and relax towards the free face that has been created. This relaxation can itself create many changes whose magnitude and extent depends on the time they have had to develop.

A good log is thus essential for developing a ground profile for use in design and analysis; some aspects of the ground can be captured by a photograph and it is always useful to obtain an image of the evidence used. Some aspects cannot be captured; they need to be deduced from considering what can be seen, what has changed and what has been lost. The geographical location of the profile also needs to be recorded (to the nearest decimetre) on the surface of the earth. The elevation of the datum from which the profile has been measured is also required, at least to the nearest decimetre, and if possible to the nearest centimetre. The age of the exposure or borehole core upon which the profile is to be based is also relevant and should be recorded, or if unknown, this should be noted. Finally, the profile needs a complete description. Should any one of these pieces of information be missing the profile is incomplete; that means the geotechnical engineering intended at the site will commence in ignorance of essential information. Its absence will call into question the geotechnical competence of those involved.

13.3 Importance of a profile

Section 1 of this manual has set geotechnical engineering in context; it is a branch of engineering that came from the need to engineer in soil and rock. Experience in this field is recorded as case history; it describes what happened in certain circumstances, even if the reason for the ground's response was not understood at the time. Geotechnical engineering requires knowledge of the ground and of its mechanics to be used in a relationship that is described by the geotechnical triangle. The ground profile forms the apex of the triangle and describes the material and its history to which the 'well-winnowed' case histories of the triangle refer. It is easy to appreciate why this is so. Ground has four attributes of direct relevance to geotechnical work: strength (the loads that can be carried), deformability (the deformations that the loads will generate), conductivity (the flows that will be permitted) and durability (the alterations that will occur as a result of changing the environment under which the ground existed before engineering began). All this information comes from a ground profile, either directly by testing and monitoring the ground, or indirectly by calculation and analogy. The success with which this is done is a function of the quality of the ground profile. A basic log is the minimum required; one that has been put into context and interpreted is better.

The profile illustrated in **Figure 13.2** comes from the investigations for the underground car park at the Palace of Westminster in London; otherwise known as the Houses of Parliament (Burland and Hancock, 1977). The site is adjacent to the bell tower of Big Ben and the north embankment of the River Thames. The works required an excavation to 20 m below ground level into the London Clay and the creation of a box supported on piles cast *in situ* under-reamed at their base. Diaphragm walls were considered for lateral support but their depth gave cause for concern once the ground profile for the site was properly studied. The core was carefully taken and logged in detail by splitting it along its length to reveal the fabric and grain size of the material. Thin layers of silt were found within the clay and these were identified as a serious potential hazard, because they could transmit water from the alluvium of the Thames, if hydraulically connected to it, possibly causing softening and erosion of the adjacent clay. Such silt layers were a source of drainage for the inter layers of clay, so facilitating consolidation; their presence would also jeopardise the efficiency of any under-ream executed within that zone. None of this was considered an acceptable risk adjacent to so important a national monument. The design was changed to deepen the diaphragm walls so as to cut off all water access through the 10 m of clay inter-bedded with silt, and the piles were lengthened so that their under-reamed base started below the 10 m section containing the silt horizons.

It is important to appreciate that it was the profile which made the case for design change self-evident and unquestionable; analyses of the changes followed but no calculation had been required to see the need to change the design. Interestingly, the problems arising from silts seem not to have been considered until the succession had been studied as a profile. Prior to that, emphasis had been placed on assessing strength; over 100 unconfined compression tests had already been conducted.

The profile in **Figure 13.3** comes from an area where soft marine sediments overlie very stiff clays that cover limestone bedrock. A linear excavation to bedrock for cut-and-cover tunnelling was required and needed diaphragm walling for permanent support. **Figure 13.3(a)** illustrates the character of the

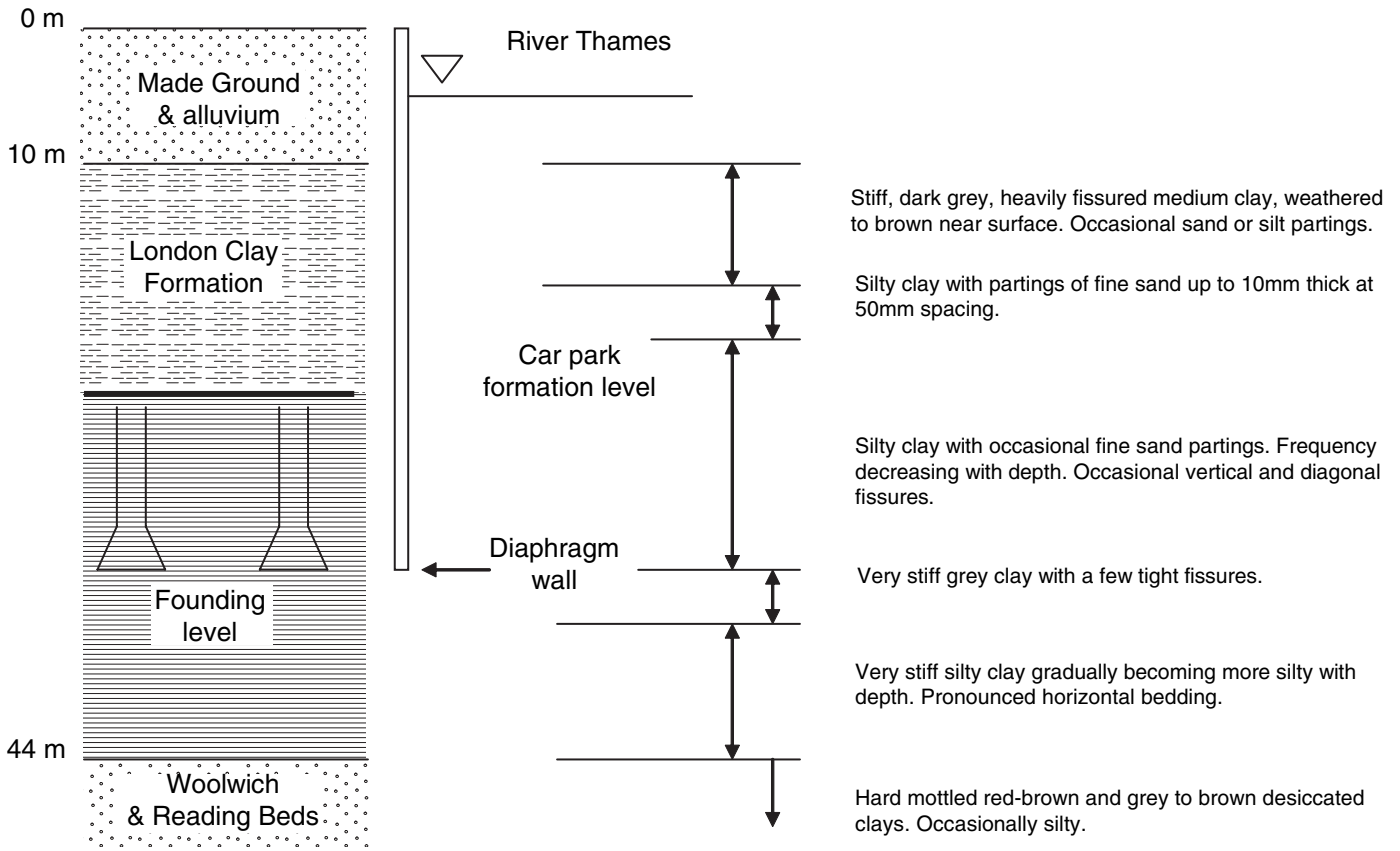


Figure 13.2 Portion of a log from the underground car park at the Palace of Westminster in London (Burland and Hancock, 1977)

ground and **Figure 13.3(b)** as it appeared in the borehole logs together with its description. The contractor had priced on the basis of the pictorial log and found that soil conditions terminated at higher levels, so upsetting both his progress and his contract price; delays followed and a claim resulted.

These examples highlight why the correct interpretation of a profile is important and illustrate some of the difficulties in creating one. In the case of the underground car park, the first mistake came from not appreciating the subtleties of geological nomenclature. 'London Clay' is short-hand for the name of a geological formation (the London Clay Formation) and not a lithology. A lithology is a sediment type whereas a formation is a deposit of a certain age usually named after the dominant lithology within it – clay, in this case. Other lithologies, notably silt, can also be present. Always ask: 'What is this material?' and request the answer in plain English, not in technical geological terms. For sediments and sedimentary rocks the description should be in terms of grain size, mineralogy and fabric; for igneous rocks it will be in terms of crystal size, mineralogy and fabric (sometimes called texture). Metamorphosed equivalents of these, created by the geological action of heat and pressure, can also be described in these terms, depending on whether they are now more like sedimentary or igneous rocks.

The second mistake followed from the first because the problem was perceived to be restricted to strength and deformation. Obviously strength and deformation were relevant but they were not the only issue of substance; much damage to both the ground and the surrounding structures could have resulted during excavation, long before the 'strength' of the ground in response to either lateral or vertical loads was mobilised. In other words, the design and an important part of the ground investigation to support it had missed a key component governing ground response to the engineering work. That is how the second profile caused problems for the excavation of diaphragm walls.

A number of contractors had declined to bid for this work sensing something was 'odd' about the site; the pictorial representation did not seem to reflect the written description. To understand the evidence it was necessary to appreciate the origins of the materials on site and their history. The limestone was of Carboniferous Age (~360 million years old) and much deformed by folding and faulting; further, there had been plenty of time and opportunity for dissolution and karst to develop within it. The clay above was hard, because it had been accumulated beneath a glacier, and contained large pieces of limestone picked up by glaciers as they travelled over the country. Glaciers had covered the area in the geologically

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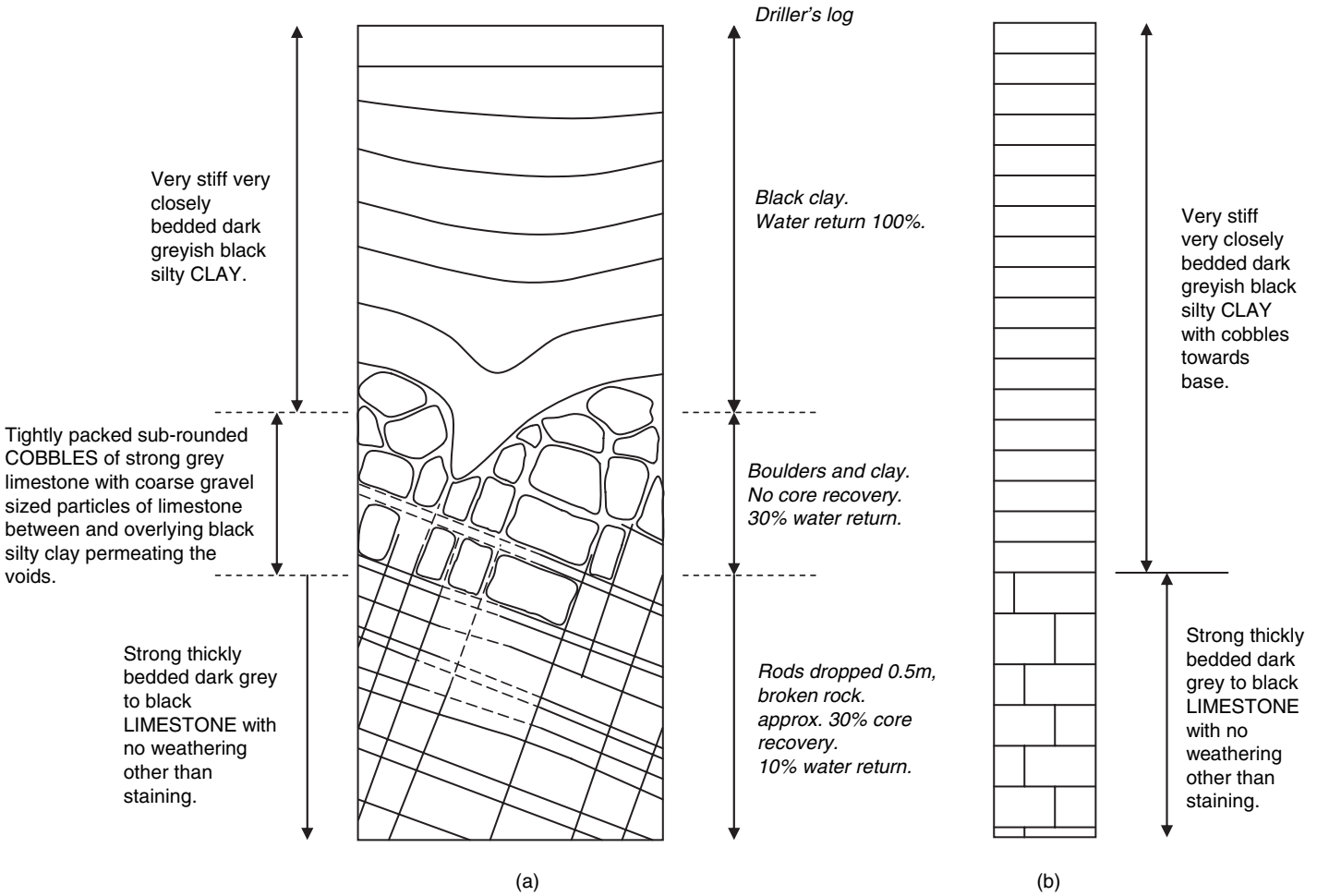


Figure 13.3 The borehole log of soft marine sediment over glacial deposits blanketing karstic limestone in Eire; (a) sketch of vertical profile as exposed by excavation and (b) as described by the driller and in the final logs

recent past (~10 000 years ago) and any debris they deposited or disturbance they created to the rock surface over which they travelled, would have filled karstic voids within the underlying limestone. None of this appears in the description; it has to be deduced from the profile and the geological history of the area. The problem here was that a zone of karstic rock with its voids infilled (as described above) had been recovered during the borehole investigation as loose debris with fragments of limestone, and interpreted as 'soil' above more solid bedrock. The cutters for the diaphragm walling machine encountered the top of the karstic limestone at depths where this so-called 'soil' had been expected.

These examples, two of many, illustrate that a ground profile is more than just the log of a borehole core or a trial pit, or a similar exposure. The observation and thought required to construct a profile comes from interpreting the factual records that such logs provide in terms of the response of that ground to the engineering within it; this brings such logs 'to life'. Thus, a qualitative ground profile leads to the quantitative means for predicting ground response to engineered change, viz mechanical properties and analyses; these are the two other points of the geotechnical triangle. Selecting the mechanical properties to be considered and the analyses to be used needs the direction and focus provided by a ground profile to ensure they address relevant behaviour. A ground profile can prevent serious mistakes being made in the quantitative aspect of the work.

13.4 The formation of a profile

Figure 13.4 illustrates four ways a ground profile can form. In **Figure 13.4(a)** layers of sediment have been deposited one on top of the other, and this would be the situation in many areas where sediments accumulate more quickly than they weather and erode. In **Figure 13.4(b)** weathering has changed the material near ground level so that the fresh parent material is now found at depth. This is commonly the case in regions where tropical weathering is either operating now or has operated in the past and alters material more quickly than it erodes. Such weathering can be well developed in crystalline materials, especially in granite and gabbro, where it can leave pseudomorphs of the original minerals. When that happens, shapes that were

clearly crystals of feldspar, amphibole and pyroxene originally are now found composed almost entirely of clay. Where these have remained undisturbed their structure and fabric remain intact and produce a zone of remarkable character known as a saprolite.

In many places a profile contains mixtures of weathered and unweathered material, because the rates of weathering, accumulation and erosion can change with time. Many of the profiles engineers will be dealing with in natural ground (as distinct from Made Ground) will extend from the present (at ground level) back to at least 10 000 years, to the start of the Pleistocene and the beginning of the Quaternary. Anyone working with Tertiary sediments is likely to encounter sediments whose ages can go back up to 65 million years and these would have experienced tropical conditions when the earth was a much warmer place than it is at present. Deep excavations near Aberdeen in Scotland (latitude ~57° N) penetrated stiff glacial clay that was laid down in the Pleistocene and now overlies much older granite that had been tropically weathered, probably during the Tertiary.

Figure 13.4(c) illustrates a variation of **Figure 13.4(b)**, as both are associated with the transport of material in a dissolved form. In **Figure 13.4(b)** the movement is predominantly downwards as infiltrating rainwater percolates through the profile carrying solids it has dissolved out of the profile – typical of weathering in wet climates. In **Figure 13.4(c)** dissolved solids in groundwater close to ground level are drawn up by evaporation and capillarity, and precipitate in the pores of material and at ground level as a mineral cement. This binds particles together and fills pores to produce a natural 'concrete' called a pedocrete; this is named after its predominant constituents, e.g. ferricrete also known as laterite (iron), calcrete also known as caliche (calcium), silcrete (silica), gypcrete (after gypsum; calcium and sulphur), alucrete also known as bauxite (alumina) and salcrete (after salt; sodium and chlorine). The durability they impart to the materials they indurate causes them to resist weathering and form a more durable crust to the landscape, hence they are also known as duricrusts. The layers beneath these crusts are often leached (weathered) and weaker than they were originally. Such weathering occurs in hot climates, especially in dry ones.

Figure 13.4(d) illustrates one of the consequences of disturbing the ground. In **Figure 13.4(d1)** the commonly occurring effects of overconsolidation are illustrated; fractures and bedding separation increase in number and become closer to each other towards ground level. This occurs when the ground has carried a greater effective stress than it does at present, the reduction normally being due to erosion over geological time and relaxation following the retreat of the glaciers at the end of the Ice Age. Vertical relaxation can also occur as a result of excavation, e.g. below open cast mines, quarries and deep foundations, and can start whilst these excavations are in progress. An example of relaxation associated with excavation is illustrated in **Figure 13.4(d2)**; in this case the collapse of a

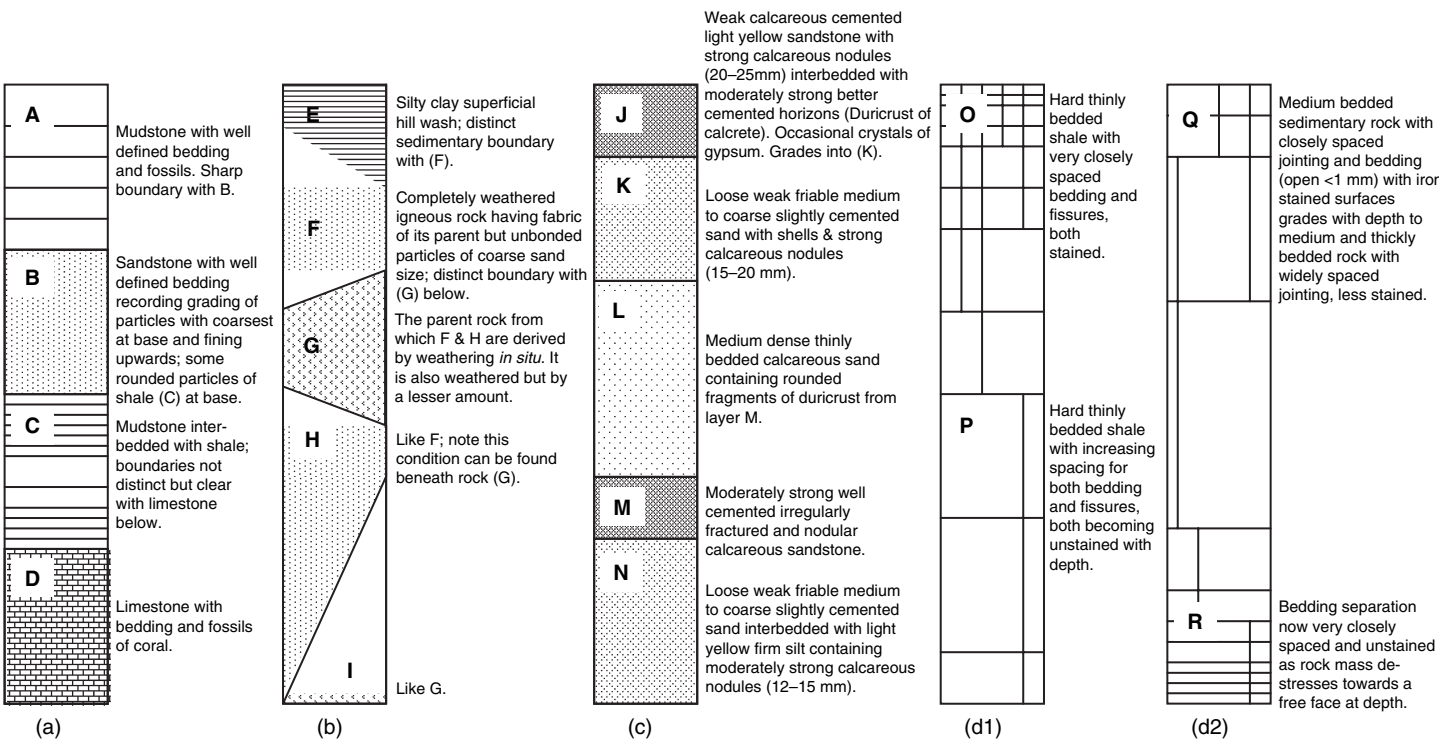


Figure 13.4 Examples of profile formation by (a) sedimentation, (b) weathering, (c) evaporation and (d) stress relief

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mine below the base of the profile being studied. As layers of strata drop into the void at depth so bedding opens starting with the beds immediately above the void; the effect decreases with distance from the point of collapse. Thus the profile detects an increase in fractures per metre depth, interpreted (correctly) as a progressively deteriorating ground condition with depth – rather the opposite of what is normally encountered. Ground adjusting to the excavation of tunnels, shafts, inclines, basements and the like can create similar changes which increase in intensity approaching the boundary of the excavation.

Although profiles can form in a variety of ways, two features of their formation are unchanging and referred to in geology as the principles of superposition and uniformitarianism.

Any layer found above another is younger than the layer it covers – this is the principle of superposition: **Figure 13.5** illustrates how care needs to be taken even when applying this simplest of laws. Mass movement has transported clay of greater age (layer B) downhill over younger beds (layer A) to form the deposit (layer E); detailed analyses of the included particles, fossil content, and possibly carbon dating, of layer E all confirm the original age of the material making up the layer is greater than that of layer A. However the crucial point is that layer E came after layer A and is stratigraphically younger than layer A. To understand what has happened attention also needs to be directed to the different fabrics the layers may have and to the boundary between them as palaeo-shear surfaces can be expected beneath layer E. Such a situation is commonplace on many slopes and excavation across the junction between such layers can re-activate slope instability (Skempton and Weeks, 1976).

If the laws of physics and chemistry permit an action to occur naturally, then that action can be expected to have happened in the past. Thus, in **Figure 13.5**, the log of an excavation or borehole across layers E, A and B would be correctly interpreted as younger over older if the observer makes a connection between the properties of the profile and the presence of mass transport on the slope, i.e. landslides. This connection would be made because the interpreter either has experience with landslides or has read case histories describing them. In other words, actions experienced at present are usually relevant to interpreting similar actions that occurred in the past. This is the principle of uniformitarianism, often over simplified, as 'the present is the key to the past'.

The log of a vertical section or borehole records geological products as a sequence of materials arranged in layers and zones, with their depth, and by applying the principles of superposition and uniformitarianism this record of products can be used to reveal the likely processes that formed them. All this helps to secure a better understanding of the implications that the profile has for engineering work that is to come. It also means that care must be taken when investigating such profiles.

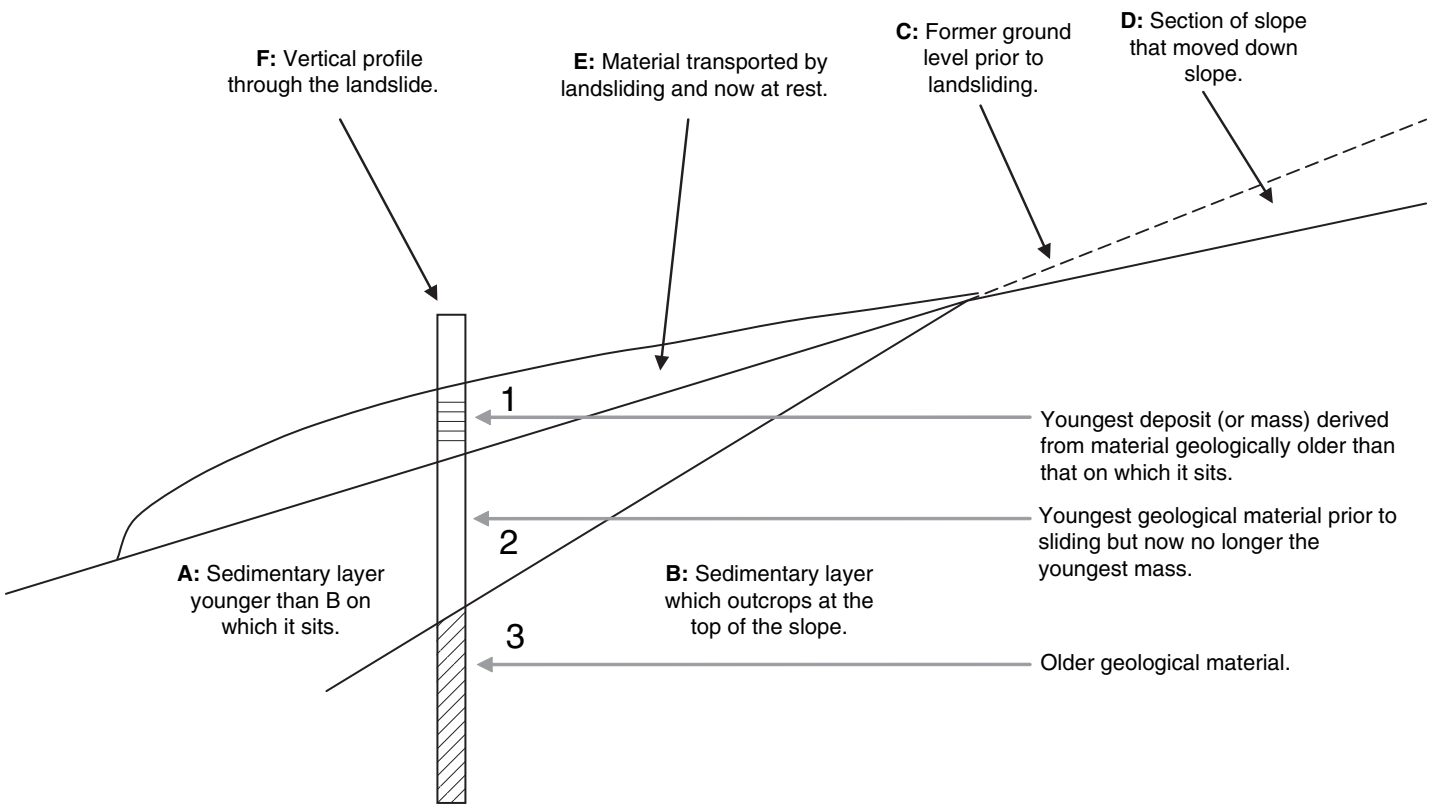


Figure 13.5 A profile from ground covered by landsliding

13.5 Investigating a profile

From what has been said about a ground profile in section 13.2 of this chapter it is evident that if resources and circumstances permit, it is best for the person who logs the ground to also provide the profile.

By far the greatest number of profiles will consist of natural geological sequences possibly covered by a thin layer of Made Ground; it is unlikely that at the start of an investigation the processes involved in forming these would be fully known. Thus any investigation into the ground and any description of it usually take place against this background of ignorance. The ground, even most Made Ground, is not a material manufactured to specification; there is no standard against which it can be compared; it is what it is and its investigation is assisted by considering as much of the ground as possible.

The easiest profile to investigate is one which is totally exposed, as in the face of a quarry or a trench. Compare that with a good quality borehole core from the same profile. The difference between these two pieces of evidence is that the quarry and trench enable the profile to be seen in context (ground to the left and right of it, and possibly below it too) whereas no such context exists for the borehole core, good though it may be.

There are three reasons why the context of information leading to a profile is important:

1. The first is that any investigation is limited; only when the ground is opened by excavation can it be properly seen. The unseen volume of ground that carries the reaction to applied loads can only be seen directly through small excavations such as boreholes, shafts and pits; thus their context is important.
2. The second reason is that the descriptions have, of necessity, to be idealised summations; every detail cannot be recorded. Seeing a profile in context makes a description easier because it helps the identification of significant features around which the idealisation and summation can be focused; thus it may be that the dominant feature is the stiffness and fissuring of clay rather than its silty laminations.
3. The third reason is that the scale used for recording has to be decided. Relevant features in a profile do not have to be large; Terzaghi records the importance of small-scale features to the stability of dams, in this case discontinuities measured in millimetres (Terzaghi, 1929). If recording had been at a scale of 1:1 the discontinuities would have been recorded on a pictorial log; however such a scale is impractical for most purposes. If the pictorial log reduces to 1:10 the fractures can barely be shown. Logs presented at around a 1:50 scale result in such small-scale features to be recorded entirely by words in the written description, supplemented by a good quality photograph, or image, of the core.

It is normal not to know the full context of a single ground profile and this should be remembered when interpreting the log of a succession. That is why it helps to refer to other related information, especially reports from a geological survey, geological maps, aerial photographs and historical land-use maps, and to visit the site and its surroundings. It also helps to have a number of profiles that are close to each other because they start to put each other into context. In addition to context there is the descriptive log upon which the profile is based and here there are rules and guidance from codes of good practice and norms. There is a tendency to believe that it is unnecessary to go beyond what a standard requires. That thinking is wrong – the standards outline the *minimum* that is required; sound guidance is provided by Norbury (2010). Man did not make the ground and often it is being described out of context; it is for this reason that Burland was instrumental in recommending the following descriptors (Anon, 1993) – some of which are used in standards – be given particular attention when logging for the purposes of creating a ground profile.

Moisture condition: this can be indicated using simple tactile descriptors such as *dry, moist* (i.e. neither wet nor dry), *slightly moist, moist, very moist,* and *wet* when water is visible. It is the state of the material as inspected and a condition that can change with time; it may be the only record of moisture content in a profile where samples are not sent to the laboratory for it to be measured. Many profiles penetrate partially saturated soils whose properties can change dramatically on wetting; clays that have shrunk can swell and sands that cohere through capillarity lose that bond when saturated.

Colour: this is the most obvious characteristic of sediments and rocks, regardless of their particle size. Although required for most standard descriptions, it is more than just an identifier because it is often related to moisture content and length of exposure to oxygen. In fissured and jointed materials the colour on the surface of these discontinuities can differ from that of the material they bound. In this way the relative ages of discontinuities can be determined, those *in situ* being stained and those formed later (e.g. by coring and/or stress relief) being less stained. A change in the colour of the sample between the field and the laboratory is the first sign that the sample has changed since its collection. Colour is not easily described; a colour chart designed for soil and rock description, such as the Munsell charts, should be used.

Consistency: this can be considered as a tactile assessment of hardness or density and thus reflects stiffness and strength. Such assessments are a normal requirement of standard descriptions and use such terms as *very soft*, *soft*, *firm*, *stiff* and *very stiff* in clayey plastic materials, to describe the ease with which material can be manipulated. It is important to handle a soil: to roll it in the fingers, squeeze it and push it to see how it holds together and whether it is on the wet or dry side of its plastic limit, and to record that for it may be the only record made if samples are not sent for testing. These simple tactile tests become even more important when dealing with silt contents that begin to alter the behaviour of the soil. The presence of silt can soon be detected by its grittiness and provided the sample has not been contaminated, this can be readily detected on the tongue and between

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the teeth. An alternative and safer method comes from the ease with which 'clay' can be brushed from hands and clothing. If your clothing can be cleaned under the blows of a good beating from your hands (which produce clouds of fine dust) then the 'clay' is largely silt. Failure to remove the soil this way suggests the presence of clay; see further comments under *Soil and rock type*. Silts and sands exhibit little in the way of plasticity and crumble under finger pressure; this has led to use of such terms as *loose* (easy to excavate or to penetrate), *dense* (difficult to do so) and *slightly cemented* (strong enough to be abraded). It is always worth attempting to test silts and sands this way as the origin of resistance within them may be no more than the capillary bond of water and easily lost with the addition of water; mineral cement provides something that can be much stronger as can fabric and not lost by the simple addition of water.

Although consistency is not a word associated with rock, weathered rock can share some of the characteristics described for soils, especially the silty-sandy soils. Weathered rock is by no means easy to describe and tends to grade from the surface of joint and bedding-bounded blocks to their interior. It is extremely valuable to describe the materials present, their locations and the volumetric percent they occupy within the mass. All this is best done qualitatively by careful observation, aided by picking and prodding the material (Martin and Hencher, 1986; Hencher and McNicholl, 1995; Norbury, 2010).

Structure: there are arbitrary divides between *structure*, *fabric* and *texture.* In geology these terms have been used synonymously; here *structure* will be used for features that extend for some distance within a mass and are easily visible to the naked eye. In profiles it is important to observe the structure of the mass – *bedding*, including *laminations*, *fissuring* (a term used for steeply inclined fractures in clayey soils), *jointing* (the term used for such fractures in rock) and *shear surfaces*, especially those from landsliding and faulting. Between these surfaces the material will possess a fabric, or texture, that will determine how the unjointed and unfissured material will behave. These elements of the mass and its materials become important when *in situ* properties are considered, as clearly seen when paper-thin shear surfaces govern the overall strength of a mass and reduce it to residual values. Structure is easy to describe using terms such as *thinly laminated* and *very thickly bedded* for the range of bedding and *extremely closely spaced* to *very widely spaced* for the range in fissures and joints.

Texture, or fabric, are much more difficult to describe yet they can largely control the behaviour of laboratory samples. Although there is no agreement on usage, it is helpful to think of fabric as being the spatial distribution of materials (like the pattern in a cloth woven from different threads) and texture to be the morphological result of their size and distribution (like the surface of such a woven material, which could be rough or smooth).

Small-scale features are significant (Rowe, 1972). They are often revealed by inclusions of different material (e.g. rounded chalk pellets and angular gravel-sized fragments of flint in a homogeneous matrix of putty-like chalk) and subtle changes in the roughness of a freshly formed surface produced by breaking the soil and allowing it to air dry. It is extremely difficult to describe much of what can be seen at this scale and use has to be made of concepts such as *structureless*, *heterogeneous*, and *mottled* (where colour and heterogeneity combine). Rarely can a description be considered complete and so the objective at this scale must be to record as much as can be seen with the vocabulary available, and support that with a scaled photograph. The difficulties here are well reflected in the problems associated with describing weathering (Anon, 1995). Sometimes clayey sediment separates easily on bedding, as partings, especially if it has traces of silt, and this should be noted.

Texture is something perceived on surfaces and should be considered because if the surfaces have formed by shearing they will almost certainly contain grooves and striations that betray *slickensides*; these indicate that their shear strength will be at a residual value in their direction of grooving. If surfaces are rough, their shear strength will be a function of friction and dilation in the direction of shear.

Soil and rock type: every description requires this and care must now be taken because the exercise of description slips into one of classification.

For soils, grain size and plasticity dominate the decision of what to call them; the former in materials where individual particles can be seen and the latter for materials where individual particles are indiscernible. The boundary between these two is where mistakes are easily made – in the silt/clay transition of the soil classification. Here tactile evidence is of great help; it is important to know whether silt is present in clay and a warning that laboratory tests may be needed to be certain what the soil should be called. A particularly sensitive tactile test requires a sample of the sediment to be placed in the palm of the hand and gradually dispersed with water until a paste is produced which can then be squeezed, by closing the palm, and relaxed by reopening the palm. The paste can behave in different ways depending on its silt content. If purely silt the paste, on opening the palm, will appear dry and possibly brittle, but tapping the palm will soon bring its free water to the surface as its brittleness melts away and it reverts to a paste again. All this reflects the response of pore water in the sample to dilation of its particles, and to capillary tensions between them; it indicates that the paste dilates readily, that its particles are larger than clay and permit the easy movement of water between them (unlike clay). In contrast, pure clay will produce a paste, which is still wet and creamy after the palm is squeezed and reopened. The name eventually given (*silt, clay, silty clay,* etc.) must conform to the specifications of the standards and codes that govern these things, and can normally only be achieved with the aid of laboratory tests.

For rocks the task is rather different. Rock names, unlike soil names, are traditionally based on their geological origin, petrological fabric and mineralogical composition e.g. granite, gabbro, limestone and sandstone. Their mechanical properties are linked to fabric and material, and as yet there are no rules that

can be used to relate rock names to their mechanical properties; the link for this is empirical. It is therefore important to name them correctly, as their name will be used to link into archives of behaviour for rocks of similar character. Serious difficulties for both description and naming arise when the rock is weathered; a working group that reviewed this cautioned that in every case the material should be described as well as possible but that a classification of weathering (grade 1, 2, 3, etc.) should only be used in cases where it can be applied unambiguously (Anon, 1995). Although some standards and codes may do things differently, the advice within the 1995 reference (*ibid*) is scientifically sound and worth recording as it can be applied to rocks in all climatic zones.

Origin: until now, the work of recording a profile has mainly involved description with some classification when dealing with soil and rock type, but to deduce the origin of a material is to venture into an interpretation, unless the material has been seen to form. The many ways in which geological materials may form can be grouped into two categories: either they were transported to where they are now – by wind, water or ice, or as a viscous flow driven by gravity, or by man – or they originated in place and are the remains, or residues, of what was there before. Geological experience is required to decide on origin especially as the materials within a profile may have formed in a number of ways. Section 13.4, The formation of a profile, introduces the four basic routes that can result in a profile, and any one route does not preclude the operation of another at a later date. Thus a saprolite that has formed in place may have its upper levels disturbed by hill wash and be covered by transported material that is later indurated to create a duricrust – the whole profile being subsequently disturbed by adjacent excavation which relaxes the ground. In other words, deducing the origin of a profile need not be straightforward.

Allocating a correct name to a material, particularly a stratigraphical name (e.g. Oxford Clay Formation) is helpful but not sufficient. It refers to the accumulation of the material at its *time zero* (when it was formed). However, the engineering behaviour of the profile will also reflect what has happened to it since then – and in geology *time zero* could be a long time ago.

Ground water: it is often not appreciated that this is truly an independent variable; its level can rise and fall for no ground-related reasons, but arising from infiltration, recharge and discharge. Its level is a fact in time and is important; it should be recorded both when encountered and not encountered. It may be that the descriptive log records no ground water and that standpipe and piezometric data needs to be obtained later to complete that record.

13.6 Joining profiles

Having obtained individual profiles from specific locations and in so doing acquired an insight into the character of the ground on site, it is necessary to use that knowledge to reveal what may lie elsewhere on site in locations that have not been investigated. Extrapolating the individual profiles laterally and possibly vertically (as results when they are joined to form vertical cross-sections) does this, and enables the tiny pieces of information they represent to be used to best advantage. However, there are three hazards associated with this exercise.

The first hazard when using extrapolation is quality: if the exercise uses data from different sources, e.g. logs from elsewhere and your own records, or a mixture of records from a variety of sources, then it is important to relate and calibrate the data so that like can be joined with like. Different schemes of logging or observers with different experiences might have been used. Similarly, descriptions might have been acquired at different times of the year when the profile may have exhibited seasonal changes linked to moisture content and water level. The second hazard is scale: it is necessary to consider the scale at which data has been recorded, as mentioned in section 13.5 above – to know what has been combined and is now presented as a 'homogeneous' unit. The third is origin: here, attempts at understanding the origins of a profile are rewarded because that knowledge constrains the data that can be joined. Units of different origin, even if they are at the same elevation, should not be joined. The fact that they cannot be joined suggests there is some other reason on site, perhaps not yet revealed, that prevents this, e.g. weathering, erosion, faulting, landsliding, subsidence.

Figure 13.6 illustrates a vertical section created from individual ground profiles. Three boreholes intersect firm, thinly-bedded clay that becomes soft towards ground level and stiff above a sand layer at depth. The only clear marker horizon is the clay–sand junction – and with no other information, that would be the only line on the section. However, when standard penetration test results obtained in the clay are plotted against the profile (**Figure 13.6a**), the change in consistency described qualitatively can be seen to have quantitative limits which divide the profile into three, so making four zones (**Figure 13.6b**). These can be detected across the site and, when used, show that although the site has horizontal strata there is superimposed upon this original structure a second profile that has sloping boundaries. Estimates of mechanical parameters and material response based on depth alone might therefore be in error. Such profiles can be generated by weathering and erosion.

13.7 Interpreting profiles

Interpretation is applied both consciously and subconsciously throughout the assembly of a profile; every description requires an interpretation to convert what is seen into a decision on what it is. Photographs involve no interpretation, but they have to be interpreted to help create a profile, and are often better if some thought is given to what is being recorded. Decisions are made about the material seen, the definition of its boundaries, its origins and its relationships with other profiles nearby. All this is a prerequisite to an interpretation of the response of the ground to engineering.

This interpretation might be possible without any further information, as was partly the case with the excavation at the

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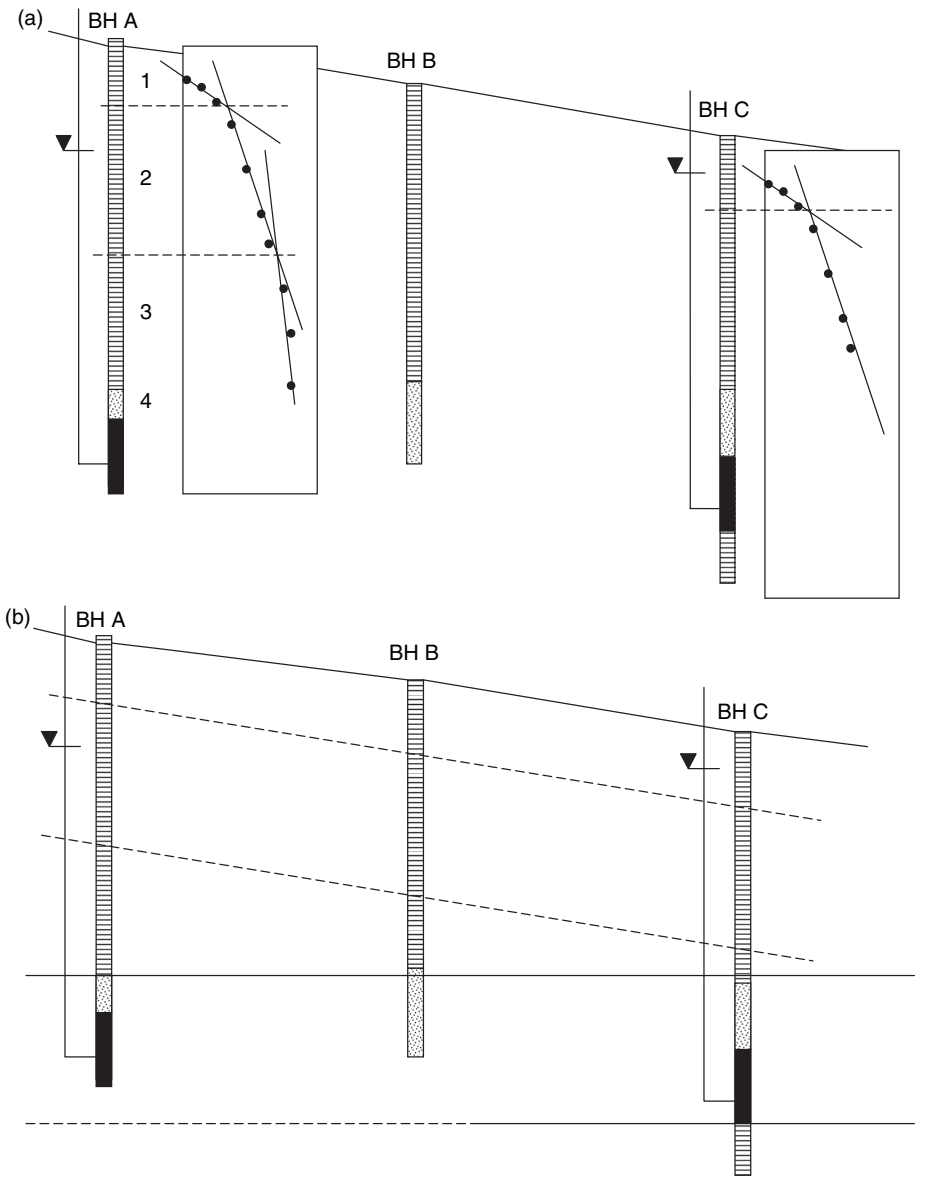


Figure 13.6 A vertical cross-section created by joining ground profiles

Palace of Westminster (**Figure 13.2**). However, in many cases there is other information of a more mechanical character that can be superimposed on the basic vertical cross-section(s) created from the individual profiles. Core recovery, fluid returns, water levels, penetration rates, borehole instability, *in situ* tests, laboratory tests, etc. can all be superimposed upon the vertical sections and profiles to assist their interpretation (**Figure 13.6**).

When using ground profiles it is helpful to consider the ground response that can be anticipated in terms of strength, deformability, permeability and durability, and to review the profile in relation to each, selecting whichever is pertinent to the work in mind; often this will be three of the four. From this will come an holistic view of the likely response of the ground: one that is qualitative but can be justified and its magnitude approximated to give sufficient focus to more quantitative methods for analyses that will have to idealise the ground for the practical purposes of calculation. In this way the basis for that idealisation is placed on reasoned judgement and good geotechnical practice.

13.8 Conclusions

1. A ground profile relies on an accurate and complete description of the ground, usually in the form of a log.
2. The primary purpose of that log is to describe the ground as it exists in the ground, *in situ*; it forms the most basic of ground profiles.

3. To that basic data should be added the observations and interpretations that put the log into context in terms of geological history, especially in terms of characters that are time dependent, such as ground water, gas, *in situ* stress and stress relief.
4. This allows some thought to be given to the history of the ground, from its formation to the present day, and especially over the past 10 000 years – essentially since the end of the Ice Age.
5. These data should then be considered at a scale that is appropriate for the engineering work to which they will be applied, and joined to give cross-sections from which a conceptual model of the ground can be gained.
6. Such a model enables the grouping of material, which will be allocated characteristic values, to be justified on the basis of available evidence, verified as the ground is exposed and adjusted as needs be, with the implications of such adjustments being fed directly into the mechanical properties of the ground and its analysis.
7. Ground profiles are therefore both the starting point for and the justification of quantitative analyses of the ground. Securing a good set of profiles is the best investment a ground investigation can make.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

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Chapter 14
Soils as particulate materials

John B. Burland Imperial College London, UK

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Soil is made up of countless particles of different shapes and sizes which are in frictional contact with each other and are acted on by gravity. This chapter illustrates the key features of mechanical behaviour that stem directly from the particulate nature of soils. A simple base friction apparatus is described that demonstrates some of these features. In particular the importance of self-weight and microfabric are illustrated, together with the phenomena of contractancy and dilatancy. The apparatus is used to study particle movements associated with active and passive earth pressures, settlement and bearing capacity. A simple mechanistic approach is presented to illustrate the importance of pore water pressure, leading to a demonstration of Terzaghi's effective stress principle. A similar mechanistic approach is used to introduce some key features of unsaturated soil behaviour.

14.1 Introduction

It was pointed out in the Introduction to Chapter 4 *The geotechnical triangle* that soil is a particulate material. Thus a sample of soil is made up of countless particles of a variety of shapes and sizes. The particles are in contact with each other, and the arrangement of particles is often referred to as the 'soil skeleton'. Loads are transmitted through this skeleton at points of contact between the particles. Frequently the contacts between the particles are essentially frictional. However, in many natural soils there is a small amount of bonding between the particles either due to cementation or physico-chemical effects (Mitchell, 1993). The presence of even a small amount of bonding between the particles can have an important influence on both the stiffness and strength of a soil.

The purpose of this chapter is to illustrate in a very simple mechanistic way the key principles governing the behaviour of particulate materials. In order to carry out calculations of ground displacement and stability it is necessary to idealise the soil as a continuum with certain stiffness and strength properties. The danger is that we get so used to thinking in terms of the stress–strain response of idealised continua (e.g. porous elastic materials or elastic–perfectly plastic materials) that we forget all too easily that the soil is actually particulate and its behaviour is controlled by this fact.

14.2 Phase relationships

Figure 14.1 illustrates an element of soil made up of a number of discrete particles. The spaces between the particles are referred to as the 'voids'. The voids may be filled with water or a mixture of water and air. Completely dry soils whose voids are filled with air are not often encountered in nature but can be reproduced in the laboratory.

The closeness of packing of the particles has a dominant influence on the mechanical behaviour of a soil. The more densely packed the particles the greater will be the stiffness and strength of the soil and the lower will be its permeability. A

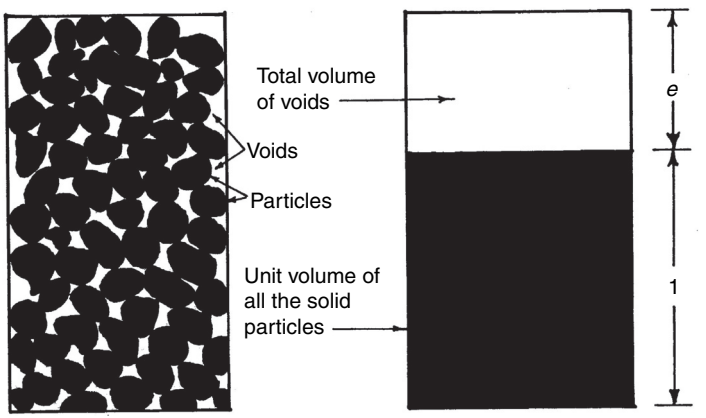


Figure 14.1 Soil is a particulate material. Void ratio e = volume of voids per unit/volume of solids

widely used measure of the 'closeness of packing' of the particles is the volume of the voids in an element of soil divided by the volume of all the particles (solids). This is termed the 'void ratio' of the soil and is denoted as e. Thus the void ratio e is the volume of the voids per unit volume of the solids – see **Figure 14.1**. Another less widely used measure of the closeness of packing of the particles is *porosity* (n), which is defined as the ratio of the volume of voids to the total volume of the soil element, so that, from **Figure 14.1**, $n = e/(1 + e)$. Because of its ease of determination, the closeness of pack of the particles in fully saturated soils (particularly clayey soils) is frequently specified in terms of the mass of water divided by the mass of the solids and is defined as the moisture content w. The various relationships between the volumes and masses of solids, voids, water and air are termed *phase relationships* and these can be found in any basic textbook on soil mechanics – e.g. Craig (2004) and Powrie (2004). These relationships are not complicated but remembering them can be tedious!

14.3 A simple base friction apparatus

It is possible to illustrate many of the basic mechanisms of behaviour of particulate materials using a simple physical model. **Figure 14.2** shows a photograph of what is termed a 'base friction apparatus'. It consists of a perspex base across which a standard acetate strip is drawn by means of a small variable speed electric motor. The arrow in **Figure 14.2** shows the direction of movement of the sheet. In this case the model particles consist of short lengths of copper tube of three different diameters. When the electric motor is switched on the acetate sheet carries the particles with it until they come up against a boundary, after which the acetate sheet 'drags' against each particle, simulating the force of gravity acting on it. The behaviour of the particles can be projected on to a vertical screen by means of an overhead projector so that they appear to be 'dragged' downwards under the action of gravity.

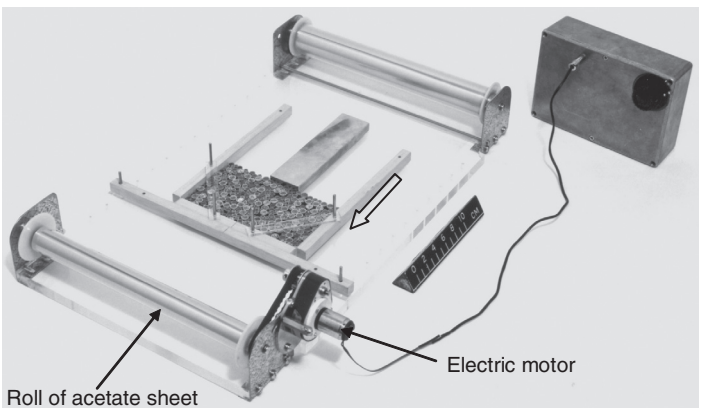


Figure 14.2 Base friction model (arrow shows direction of movement of acetate sheet)
Reproduced from Burland, 1987

The key feature of a base friction apparatus is that it simulates the forces of gravity which give rise to the all-important effects of self-weight. In soil mechanics, the self-weight of the material generates the confining pressures that govern its strength and stiffness, and it also provides the dominant stabilising and destabilising mechanisms for most stability problems.

14.3.1 The process of deposition

Figure 14.3 illustrates the process of vertical deposition of a dispersed suspension of particles acted on by gravity into a container with a horizontal base and vertical sides. **Figure 14.3(c)** shows the particle arrangements on completion of deposition and illustrates a number of important features. The small particles tend to form into clusters. There are a number of voids around which the particles tend to 'arch' so that beneath an arch the particles will not be transmitting much load. If a top plate is placed on the assemblage and gravity is 'switched off', then a gentle up and down movement of the whole assemblage shows that there are numerous 'loose' particles that are not transmitting any load, i.e. there are many more 'arches' than was at first apparent. The whole assemblage is clearly in a very loose state with a high void ratio.

Another key observation is that it is possible to trace numerous vertical and sub-vertical columns of particles showing well-defined preferred arrangements or 'structures'. Thus the particles have arranged themselves so as to resist the dominant vertical gravity forces and the assemblage is stiffer and stronger in the vertical direction than in the horizontal direction. In other words the assemblage has anisotropic properties which are inherent to the mode of deposition under the action of gravity.

The assemblage of particles shown in **Figure 14.3(c)** is termed 'normally consolidated' because the vertical stresses acting on it are those imposed as a result of deposition. If the vertical stresses are reduced subsequent to deposition the

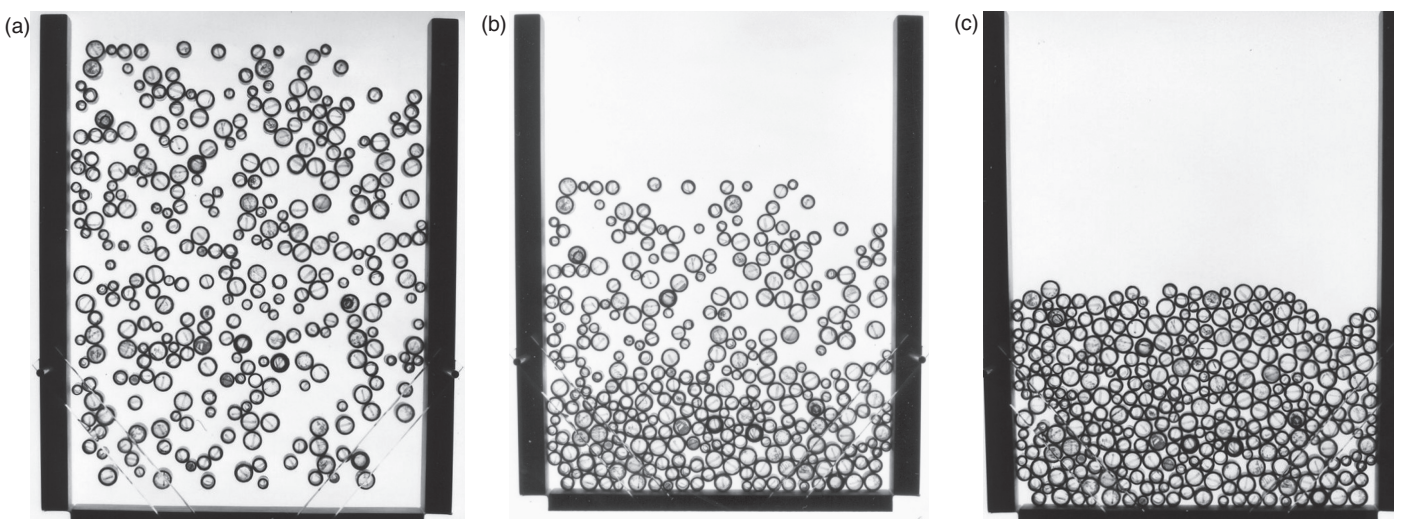


Figure 14.3 Successive stages of the deposition of an initially dispersed suspension of particles
Reproduced from Burland, 1987

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material is defined as 'overconsolidated'. The 'overconsolidation ratio' is the maximum previous vertical stress divided by the present vertical stress. Normally consolidated soils are invariably in a loose state.

14.3.2 Contractant and dilatant behaviour

If a top plate is placed on the surface of the assemblage of particles, gravity is 'switched on' and the sides of the container are then slowly rotated by equal amounts in the same sense, then the assemblage will be deformed under what is termed 'simple shear'. During this process of shearing it will be observed that the various 'arches' of particles are broken down and the particles move to take on a closer packing, i.e. the void ratio *decreases*. This process of void ratio reduction during pure shearing is known as 'contractancy'.

The assemblage of particles can be compacted to a closer pack by 'switching on' gravity and tamping the top plate. If after compaction the assemblage is again subjected to simple shear it can then be observed that the particles ride over one another and the overall volume *increases*. This process of void ratio increase during pure shearing is known as 'dilatancy'.

The phenomenon of contractant and dilatant behaviour during shearing is almost unique to particulate materials and is an extremely important property having a profound influence on the shearing resistance of the material.

14.3.3 Active and passive earth pressures

The base friction apparatus can be used to illustrate the development of active and passive regions behind and in front of retaining walls. Refer to Chapter 20 *Earth pressure theory* which discusses active and passive earth pressures. **Figure 14.4(a)** shows what happens if gravity is 'switched on' and one of the sides is then rotated away from the retained assemblage of particles. It can be seen that the movement of the particles is confined to a relatively narrow wedge-shaped region close to the wall. This is known as the 'active' wedge. At a relatively small rotation the earth pressure reaches a steady minimum value termed the 'active earth pressure'.

Figure 14.4(b) illustrates the movement of the particles when the wall is rotated inwards towards the retained material. It can be seen that the region of movement of the particles extends much further than for the active wedge; it is known as the 'passive' region – in this case it is still wedge-shaped. Much larger rotations are required to fully mobilise full 'passive earth pressure' than for active conditions.

14.3.4 Foundation settlement and bearing capacity

The mechanisms involved in foundation settlement and bearing capacity can be studied by placing a thin rectangular strip of wood on the acetate sheet of the apparatus and then 'switching on' gravity. The foundation moves slowly down until it is in contact with the ground surface. By placing a relatively light weight on the wooden strip the frictional drag on the strip is increased (simulating a small increase in foundation loading)

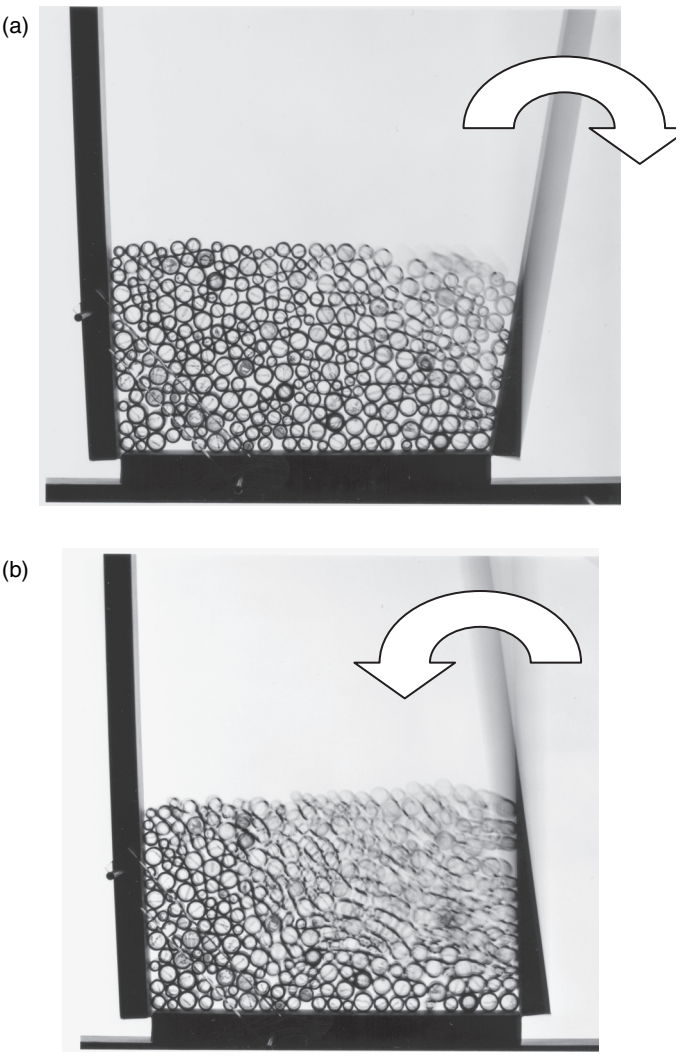


Figure 14.4 (a) Development of an active earth pressure region. (b) Development of a passive earth pressure region
Reproduced from Burland, 1987

and the foundation settles due to the compaction of some of the particles immediately beneath the foundation. The zone of compaction is confined to a depth equal to about half the width of the foundation.

The load can be increased further by replacing the wooden strip with a brass strip. When gravity is 'switched on' the foundation settles significantly and the zone of particle movement extends further beneath the foundation and outwards on either side as shown in **Figure 14.5**. The foundation has reached bearing-capacity failure and the shearing resistance of the assemblage of particles has been fully mobilised within the observed zone of movement. Further loading of the brass strip causes it to plunge and the zone of particle movement extends around the base and upwards to the surface alongside the strip. This behaviour is modelling the driving of a pile into granular material, and it is possible to visualise the development of high end resistances and also frictional resistance along the shaft of the pile.

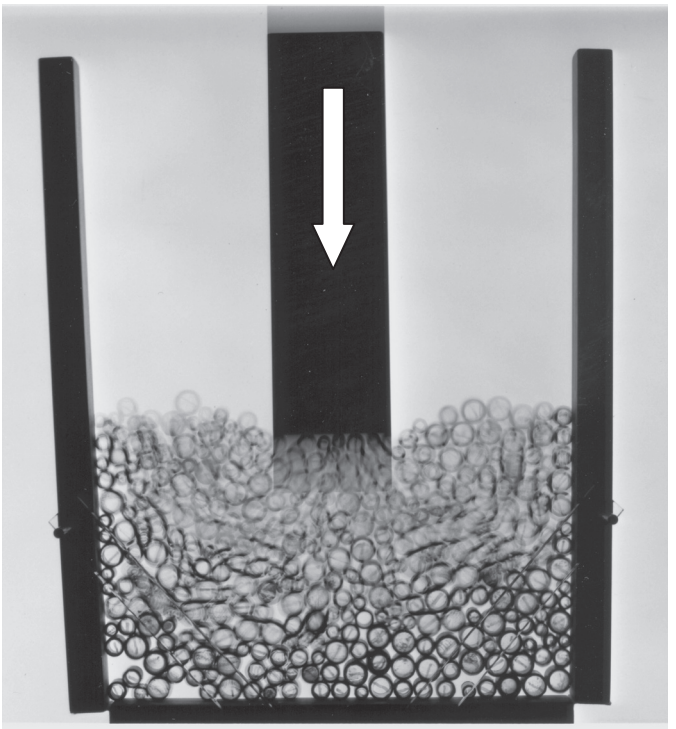


Figure 14.5 Development of bearing-capacity failure
Reproduced from Burland, 1987

The base friction apparatus can be used to illustrate mechanisms of ground behaviour for other geotechnical problems as well. For example it can be used to illustrate subsidence above tunnels and arching around them as they are excavated. Also it is possible to illustrate the mechanisms of deep-seated movement around propped excavations.

14.4 Soil particles and their arrangements

In the previous section we examined the mechanical response of a relatively simple assemblage of round particles of three different sizes. It was evident that, even with an assemblage composed of uniformly shaped and sized particles, relatively complex arrangements can occur. Most natural soils are made up of much more complex arrangements of particles.

14.4.1 Particle size

The range of sizes of particles that make up a soil is huge, varying from boulders, through gravels, sands and silts down to clay sizes. **Table 14.1** shows the approximate sizes of these particles and some important visual and tactile descriptions. **Table 14.2** illustrates the amazing range of sizes, numbers and surface areas of typical grains in a gram of soil. **Table 14.3** describes dimensions of three typical clay minerals. The engineering importance of grain size lies primarily in the drainage characteristics of the soil thereby influencing the permeability and rate at which the volume change and strength respond to changes in loading.

Very few soils in nature fall entirely within one grain size classification. Mostly soils are mixtures (e.g. silty clay; very sandy, fine to coarse gravel). The distribution of grain size in a soil is termed its 'grading'. Thus the 'soil type' is primarily based on grain size and grading. There is an unfortunate tendency in practice to refer to 'cohesive' and 'non-cohesive' soils to distinguish between soils with a high or a low clay content. This can be very misleading as many normally consolidated

Soil type		Size (mm)	Visual or tactile description
Boulders			**Visible to the naked eye**
Gravel	Coarse	>60	**Particle shape:**
	Medium	20	Angular; subangular; rounded; flat; elongated
	Fine	6	
Sand	Coarse	2	**Texture**: Rough; smooth; polished
	Medium	0.6	**Grading**: Well graded (wide range of sizes)
	Fine	0.2	Poorly (uniformly) graded
			Gap graded
Silt		0.01	**Not visible to naked eye**
			Gritty to hand or teeth
			Disintegrates quickly in water
			Exhibits dilatancy when squeezed in hand
Clay		0.002	Feels soapy when rubbed with water in hand
			Sticks to fingers and dries slowly
			When spread with knife leaves shiny surface
Organic soils			Contains substantial amounts of organic matter
Peats			Predominantly plant remains, dark brown or black
			Low bulk density

Table 14.1 Soil types

Particle	Diameter (mm)	Mass (g)	No. per gram	Surface area m^2/g
Boulder	75	590	1.7×10^{-3}	3×10^{-5}
Coarse sand	1	1.4×10^{-3}	720	2.3×10^{-3}
Fine sand	0.1	1.4×10^{-6}	7.2×10^{5}	2.3×10^{-2}
Medium silt	0.01	1.4×10^{-9}	7.2×10^{8}	0.23
Fine silt	0.002	5.6×10^{-12}	9×10^{11}	1.1

Table 14.2 Typical grain dimensions (assumed spherical)

Mineral	Width (mm)	Width/thickness	Surface area (m^2/gr)
Kaolinite	0.1–4.0	10:1	10
Illite	0.1–0.5	20:1	100
Smectite	0.1–0.5	100:1	1000

Table 14.3 Typical clay mineral dimension

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clay soils do not exhibit cohesion and many granular soils can be bonded. Moreover, all soils that are sheared without allowing drainage exhibit undrained strength which is often treated as an equivalent cohesion in analysis. True cohesion in a soil is a very difficult property to determine and its precise definition is far from clear. It is much better simply to refer to 'clayey soils' and 'granular soils' without implying anything about their cohesion (sometimes the terms 'fine-grained' and 'coarse-grained' are used).

14.4.2 Particle shape

The particles used in the simple base friction model illustrated in section 14.3 of this chapter are circular in shape. Soil particles vary in shape. For granular soils (boulders down to silt-size particles) the variety of shapes may be embraced by the following descriptors: angular, sub-angular, rounded, flat, elongated – see **Table 14.1**. The texture of the particle surfaces also varies ranging from rough, through smooth, to polished. The shape and angularity of the particles have a profound influence on its shearing resistance. This was impressed firmly on the author when he went for a walk on Chesil Beach in Dorset. The beach is made up of smooth rounded gravel-sized particles – see **Figure 14.6**. As a consequence it is extremely difficult to walk on the beach because the grains move around so much underfoot!

Clay particles are generally much smaller than their granular counterparts and are usually plate-like in shape with plan dimensions much greater than the thickness. **Table 14.3** gives some typical dimensions of three common clay minerals: kaolinite, illite and smectite (montmorillonite). The clay content in a soil, together with stress history, has a dominant influence on its compressibility.

14.4.3 Fabric and micro-structure

The above sizes and shapes of particles combine to form a bewildering array of particle arrangements – termed the 'fabric' of the soil. **Figures 14.7** and **14.8** show, respectively, some typical

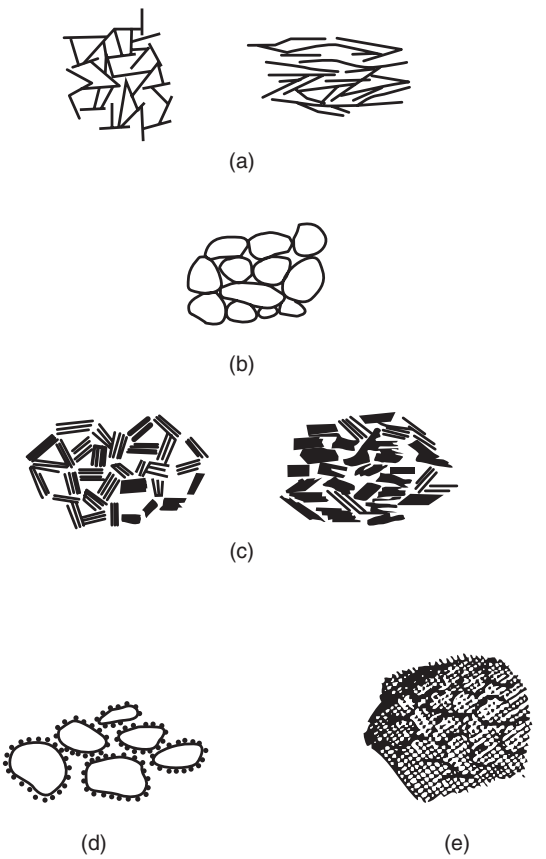


Figure 14.7 Schematic representation of elementary particle arrangements. (a) Individual clay platelet interaction. (b) Individual silt or sand particle interaction. (c) Clay platelet group interaction. (d) Clothed silt or sand particle interaction. (e) Partly discernible particle interaction
Reproduced from Collins and McGown (1974)

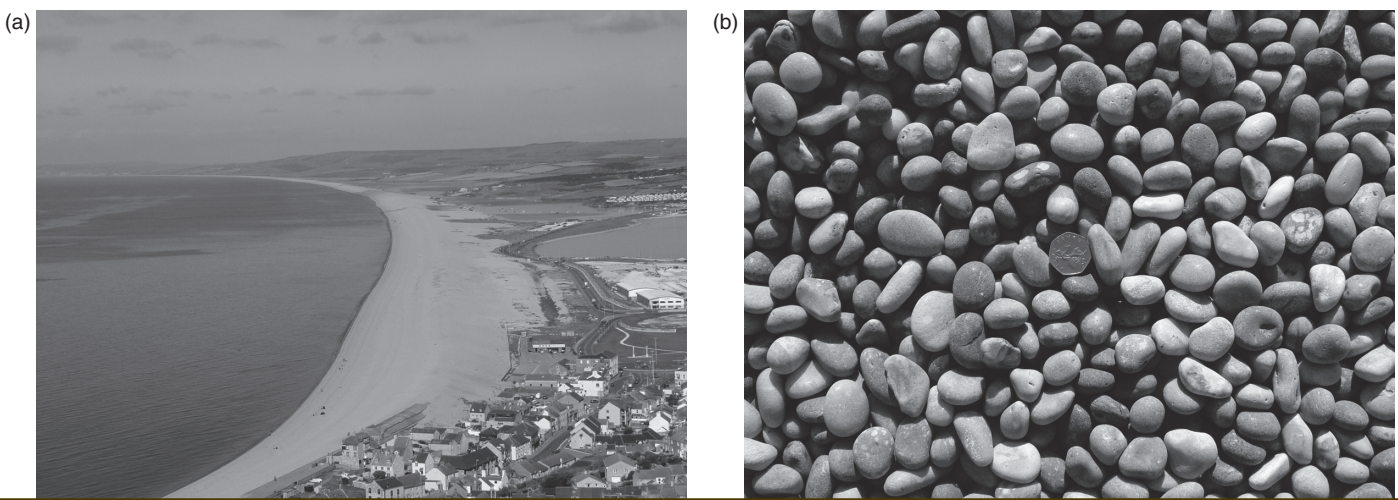


Figure 14.6 Chesil Beach, Dorset, made up of smooth rounded gravel

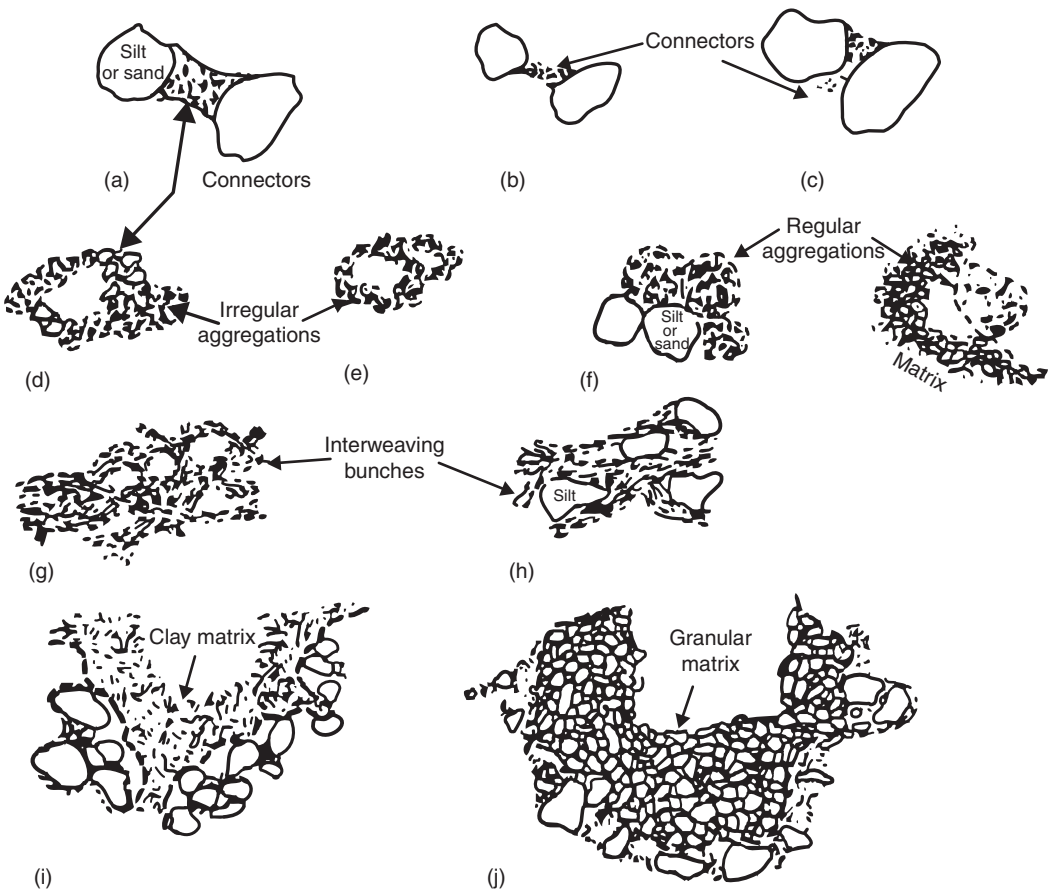


Figure 14.8 Schematic representation of particle assemblages. (a), (b) and (c) Connectors. (d) and (e) Irregular aggregations. (f) Regular aggregations interacting with particle matrix. (g) and (h) Interweaving bunches of clay. (i) Clay particle matrix. (j) Granular particle matrix
Reproduced from Collins and McGown (1974)

schematic representations of elementary particle arrangements and particle assemblages (Collins and McGown, 1974).

As mentioned previously, the particles of soil are usually in mechanical contact with each other, and resistance to sliding at the contact results from friction and interlocking. Frequently in natural soils the particles are lightly bonded at their grain contact points due to cementation or physico-chemical effects.

The term 'micro-structure' means the combination of 'fabric' (the arrangement of the particles) and interparticle 'bonding' (Mitchell, 1993).

14.4.4 Macro-structure

It is important to appreciate that many soils have discontinuities through the mass of the soil as a result of geological or man-made processes. For example joints and fissures can form due to deformations during deposition or tectonic activity. These give rise to important planes of weakness, particularly if clay particles on the joint surface have realigned themselves to give very low frictional resistance. Similarly, pre-existing shear planes can exist in clayey soils due to previous landslipping or geological processes. The weathering of rocks containing relic joints can also lead to important planes of weakness in the soil mass. The presence of such discontinuities can have a dominant influence on the strength of the ground.

Varying depositional environments can lead to alternating layers of clays and granular materials. These more permeable layers usually need to be taken into account in the design and construction of excavations, slopes and embankments. An example of this is given in Chapter 4 *The geotechnical triangle* where it is shown that the design of the underground car park at the Palace of Westminster was dominated by the discovery of thin silt and sand partings in the clay just beneath excavation level. It is also important to note that the presence of joints and fissures can profoundly influence the mass permeability of the ground, particularly if any bonding or cementation is present in the soil mass, because there will be a tendency for these discontinuities to remain open.

14.5 The concept of effective stress in fully saturated soils

Appreciation that soils are essentially particulate materials, albeit very complicated ones, leads to a simple mechanistic understanding of the underlying factors that govern the behaviour of fully saturated soils. **Figure 14.9** shows an element of

soil containing an aggregation of particles where the voids between the particles are full of water. The sides of the element are acted on by the stresses σ_v, σ_h and τ_{vh} as shown, and there is a positive pressure u acting in the water. The stresses acting on the boundaries of the element (called total stresses) are made up of two parts: (i) the water pressure u, which acts in the water and in the solids in every direction with equal intensity as shown, and (ii) the balances $(\sigma_v - u)$, $(\sigma_v - u)$, and τ_h which are carried by the soil skeleton. Terzaghi (1936) defined these balances $(\sigma - u)$ and τ as 'effective' stresses. He then stated his *effective stress principle*, which is that 'all measurable effects of a change of stress, such as compression, distortion and a change of shearing resistance, are exclusively due to changes in effective stress'. Thus the two variables, the total normal stress σ, and the pore pressure u, have been replaced by a single variable $\sigma' = (\sigma - u)$. This is a tremendous advantage when measuring the mechanical properties of a soil, as only the relevant effective stresses have to be applied and not the absolute values of the total stress and pore pressure. On the other hand the effective stress principle requires that, in carrying out a soil investigation, it is essential to know the *in situ* pore pressures as well as the total stresses acting on the soil.

It is important to note that the effective stress principle says nothing about the way that the effective stresses are transmitted through the soil skeleton, i.e. it gives no information about the stresses at the particle contact points. Thus an effective stress is *not* an inter-granular stress even though it is sometimes called that. The effective stress is simply that part of the total stress that is carried by the soil skeleton. Note that a shear stress is always an effective stress because water cannot carry shear.

The validity of Terzaghi's effective stress principle can be demonstrated mechanistically as follows. **Figure 14.10(a)** schematically illustrates an element of loose granular material which in **Figure 14.10(b)** is compressed by the application of an all-round stress σ with the pore pressure u held constant. At each grain contact point is acting a normal force P and a shear force T as illustrated in **Figure 14.10(b)**. In view of the large number of grain contact points for each grain, relative displacement between grains can only take place as a result of slip at the contact points. For a grain to be in equilibrium under its contact forces the ratio T/P must be less than or equal to μ at every contact point, where μ is the coefficient of friction of the material composing the grains.

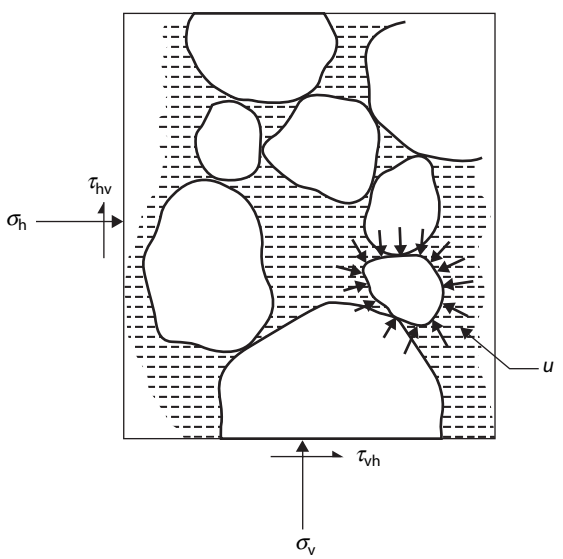


Figure 14.9 Mechanistic model of a loose granular material
Reproduced from Burland and Ridley, 1996

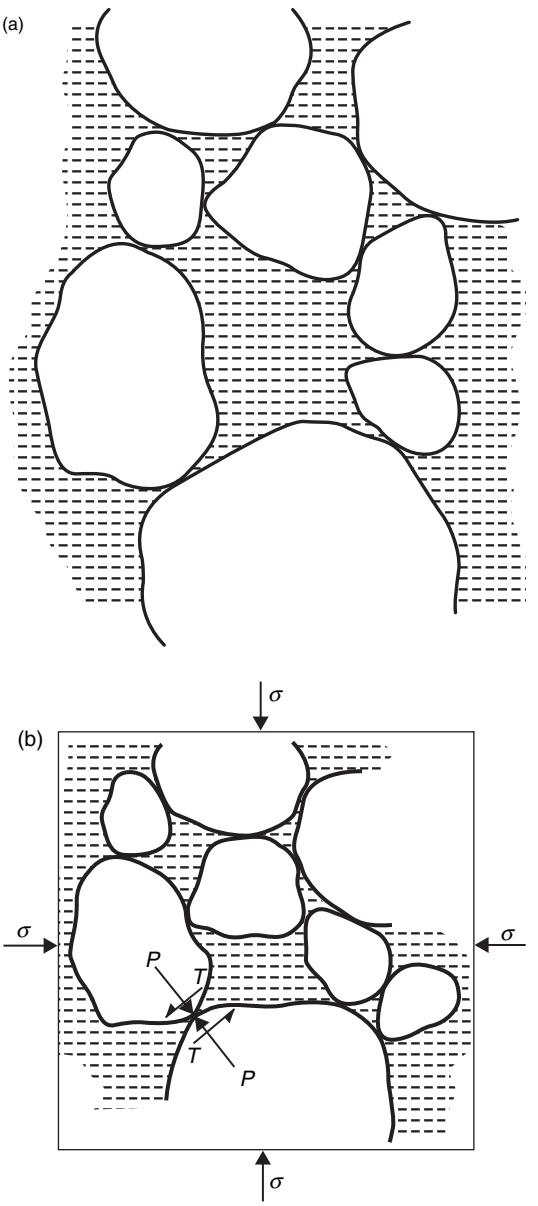


Figure 14.10 Mechanistic model of a loose granular material compressing under equal all-round effective stress. (a) uncompressed. (b) compressed
Reproduced from Burland and Ridley, 1996

An increase in all-round stress $\Delta\sigma$ at constant u will cause a small decrease in volume. This occurs as a result of grain slip at some contact points at which $T/P = \mu$ giving rise to a closer pack of the particles. Therefore an increment in all-round stress induces increases in shear force as well as normal force at the grain contact points. If, instead of an *increase* in total all-round stress σ, the pore pressure u is *decreased* by the same amount, the assemblage of grains would behave in exactly the same way. Clearly a decrease in u is equivalent to an increase in σ in its effects on the grain contact forces. This is a simple mechanistic demonstration of Terzaghi's principle of effective stress for fully saturated soils. It provides a clear demonstration that knowledge of the pore pressure in a soil is just as important as a knowledge of the applied stresses.

14.6 The mechanistic behaviour of unsaturated soils

When a soil is unsaturated the voids contain both air and water. If the air voids are continuous then the water tends to migrate towards the grain contact points due to surface tension effects. **Figure 14.11** shows two spherical particles in contact, with a lens of water around the contact point. It can be shown that surface tension within the curved surface of the water, called the meniscus, generates a compressive force F between the particles. This phenomenon is of profound importance to the behaviour of an unsaturated assemblage of particles. The pressure within the lens of water is less than atmospheric, i.e. the water pressure is negative and is often termed 'suction'.

14.6.1 The process of drying a soil

Figure 14.12 illustrates mechanistically the process of drying for a loose granular soil. In **Figure 14.12(a)** the assemblage of grains is fully saturated and the pore pressure is positive. Evaporation, or drying, then starts to take place, reducing the pore pressure. In **Figure 14.12(b)** the assemblage is still fully saturated but menisci have formed at the boundary. In this situation the pore pressure is less than atmospheric and the tension in the boundary menisci induces compression of the assemblage, resulting in some grain slip and a small reduction of volume.

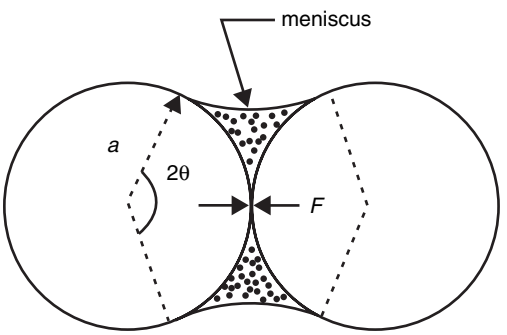


Figure 14.11 Inter-particle force due to surface tension
Reproduced from Burland and Ridley, 1996

At a limiting value of suction (known as the 'air entry value') air will enter the pores of the assemblage and the pore water will form lenses around the grain contact points as illustrated in **Figure 14.12(c)**. These water lenses will generate inter-particle contact forces which are essentially normal to the planes of contact. It is evident that the inter-particle forces generated in this way will tend to inhibit grain slip, thereby stabilising the aggregation of grains. We may think of the water at the grain contact points as acting rather like blobs of glue or bonds which increase both the stiffness and the strength of the soil. This is why a sand-castle constructed from damp sand is so much more stable than one constructed from either perfectly dry sand or very wet sand.

In clayey soils the process of drying can give rise to considerable shrinkage of the soil, often resulting in the formation of shrinkage cracks and fissures. Tree roots are a prime cause of drying out of soils in periods of drought, and in clayey soils the resulting shrinkage, and consequent subsidence of the foundations, can be very damaging to buildings.

14.6.2 The process of wetting up of a dry soil

In the previous section we saw that the process of drying an initially saturated granular soil stabilises the grain contact points and increases both the stiffness and strength. The converse is that if an initially dry soil is wetted up, the stabilising forces at the grain contact points are removed, resulting in a loss of strength and, for loose soils, a rapid reduction in volume. This process of rapid volume reduction during wetting

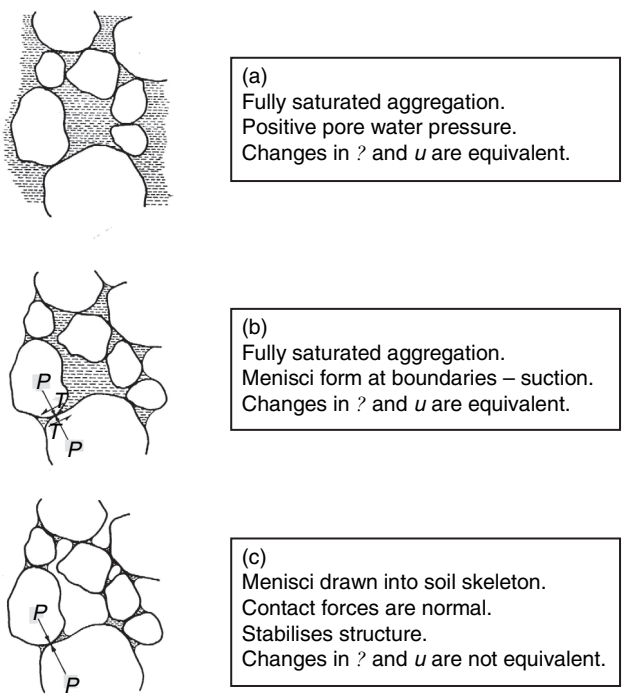


Figure 14.12 Mechanistic model showing the drying of a loose granular material
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is known as 'collapse of grain structure' or simply 'collapse'. The phenomenon of 'collapse' on wetting is well known for dry loose sandy soils in arid climates. This phenomenon is also well known for fill materials which have been inadequately compacted during placement and which subsequently wet up, often due to a rising groundwater table.

The wetting up of initially dry clay soils can also give rise to serious problems. Such soils are often fissured and cracked due to shrinkage. If water becomes available it can penetrate the clay rapidly through the cracks and cause the clay to swell, which can be very damaging to buildings founded on such soils. Also an excavation with steep sides in a dry clay soil may become unstable during heavy rain due to the rapid penetration of surface water down shrinkage cracks.

See Burland (1965) and Burland and Ridley (1996) for a detailed discussion of the mechanistic behaviour of unsaturated soils.

14.7 Conclusions

The key conclusions that can be drawn from this chapter are listed below. Some of them are oversimplified but they form a good basis for gaining an understanding of the complex behaviour of soils.

- All soils, even high plasticity clays, consist of countless particles of a variety of shapes and size. The spaces between the particles are referred to as the 'voids'. The void ratio is a measure of the closeness of pack of the particles and is an important parameter in determining the stiffness, strength and permeability of a soil.
- The particles are in contact with each other and may be thought of as forming the soil 'skeleton'. The arrangement of the particles is referred to as the 'microfabric'.
- The contacts between the particles are largely frictional but in most natural soils there is frequently a certain amount of bonding between the particles as well.
- A fundamental feature of the ground is that it has weight. The self-weight of the soil generates the confining pressures that govern its strength and stiffness. Moreover, the self-weight provides the dominant stabilising and destabilising mechanisms that operate in most stability problems.
- A simple base friction apparatus is described which simulates that action of gravity on the soil particles. The apparatus is used to introduce the formation of soil fabric during deposition. It is demonstrated that deposition under vertical gravity leads to preferred orientation of the particle arrangements and this leads to inherent anisotropy of the soil skeleton.
- When sheared, loosely packed particulate materials contract and densely packed particulate materials dilate. The base friction apparatus is used to illustrate these important phenomena.
- A number of standard soil mechanics problems are simulated in the base friction apparatus to illustrate the behaviour of particulate materials under gravity, including active and passive pressures, settlement and bearing capacity.
- The importance of particle grading, shape and fabric are discussed as well as the dominant influence that macro-structure can exert on the behaviour of a mass of soil.
- A simple mechanistic approach is presented to illustrate the importance of pore water pressure leading to the demonstration of Terzaghi's effective stress principle.
- This mechanistic approach is then used to illustrate some of the basic features that control the behaviour of unsaturated soils during the process of drying and wetting. In unsaturated soils the pore water forms lenses at particle contact points which provide an important stabilising effect on the aggregation of particles. During wetting up, this stabilising effect is reduced, leading to collapse of grain structure and reduction in strength.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

Chapter 15

Groundwater profiles and effective stresses

William Powrie University of Southampton, UK

This chapter discusses the generation of profiles of effective stresses and pore water pressures within the ground for use in geotechnical engineering calculations. Total stresses and pore water pressures are usually calculated independently, with the effective stress given by the difference between them. The calculation of geostatic and hydrostatic vertical total stresses and pore water pressures arising from the self weight of the soil and the water, respectively, is described. Non-hydrostatic pore pressure profiles (corresponding to artesian conditions of higher pore pressures in an underlying aquifer) and underdrainage into an underlying, more permeable stratum, are discussed. Finally, the difficulties of estimating pore water pressures above the water table, and lateral soil stresses, are considered.

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15.1 Importance of pore pressure and effective stress profiles

The nature of soil as a particulate material, generally with at least two phases present, was described in Chapter 14 *Soils as particulate materials*. If the soil is saturated, the two phases present are liquid (i.e. water in the pores) and solid (i.e. the soil grains). These two phases have very different stress–strain–strength responses – in particular, the pore water cannot carry any of the shear stress τ, but the presence of a pore water pressure u reduces the normal effective stress σ' to below the total stress σ and hence the ability of the soil skeleton to resist shear. According to Terzaghi's *principle of effective stress*:

$$\sigma' = \sigma - u \tag{15.1}$$

and the effective stress failure criterion for soils is:

$$\tau = \sigma' \tan \phi', \tag{15.2}$$

where ϕ' is the effective angle of shearing resistance or the effective angle of friction. Effective stresses, which govern the behaviour of all soils, cannot generally be calculated unless both the total stresses and pore water pressures are known. Thus in geotechnical engineering, it is essential to be able to define the distributions of vertical total stress and pore water pressure with depth as part of the initial ground model for the site.

15.2 Geostatic vertical total stress

It is usually relatively straightforward to estimate the profile of vertical total stress within the ground. Imagine a column of soil of unit weight γ, cross-sectional area A and height z. The force resulting from vertical total stress acting on the base, $\sigma_v(z)A$, must balance the total weight of the column, γAz, giving $\sigma_v(z) = \gamma z$. If the column of soil has a number of different layers having different unit weights, the calculation of vertical total stress must take this into account, so that more strictly,

$$\sigma_V(z) = \int_0^z \gamma(z)\,\mathrm{d}z. \tag{15.3}$$

Normally, equation (15.3) is discretised into finite layers and of course the effect of any surface surcharge q must be taken into account.

The resulting distribution of vertical total stress with depth corresponds to natural equilibrium conditions in the ground, and is often termed *geostatic*.

When applied to a body of soil, equation (15.3) gives the average total vertical stress over a cross-section. We have already seen in Chapter 13 *The ground profile and its genesis* how the stress distribution around a cavity such as a tunnel or a collapse (or, conversely, around a stiff inclusion) will be altered, and in Chapter 14 *Soils as particulate materials* that the structures set up during soil deposition may result in some contacts being relatively unstressed.

15.3 Hydrostatic conditions for pore water pressures

If the groundwater is static, the water pressure is unaffected by the presence of the soil skeleton as long as the pore network is continuous. Thus in the absence of groundwater flow, pore water pressures below the water table (defined as the level at which the pore water pressures are zero) can be calculated in a similar way to the total vertical stress, by considering a column of water of depth z below the water table. The pore water pressure would therefore be:

$$u(z) = \gamma_w z \tag{15.4}$$

where γ_w is the unit weight of water. These conditions, and the profile represented by equation (15.4), are known as *hydrostatic*.

(The reason that the presence of the soil grains makes no difference to the pressure in the pore water column is that each grain receives an upthrust equal to the weight of water displaced, while the residual weight of the soil grains, i.e. the actual weight minus the upthrust, is carried by contact forces between the grains.)

15.4 Artesian conditions

Most ground profiles will consist of a succession of strata of differing permeabilities (see Chapter 16 *Groundwater flow*). For example, two relatively high permeability strata through which water may flow relatively easily, may be separated by a low permeability stratum through which water flows only slowly and with difficulty. The relatively permeable strata are known as *aquifers*, and the relatively impermeable stratum is known as an *aquiclude*. It is quite possible for the water tables (or, more strictly, the *piezometric levels*) in the two aquifers to be different. In this case, vertical flow will occur through the separating aquiclude in response to the head difference across it (Chapter 16 *Groundwater flow*). In these circumstances, the upper aquifer is known as an *unconfined* or *water table aquifer* (because it is not overlain or confined by a low permeability layer); while the lower aquifer is termed *confined* (because it is).

A well known example of this is in the London basin, England, where a near-surface aquifer comprising alluvial sands and gravels (the Thames Gravels) is separated from the underlying chalk aquifer by the relatively impermeable London Clay. As the chalk is in hydraulic continuity with high ground to the north of London, the piezometric level within it is naturally higher than (i) ground level in the valley and (ii) the water table in the surface aquifer, which is hydraulically connected to the River Thames. The natural flow of groundwater will be upward through the London Clay, and the pore water pressures in this stratum will be greater than hydrostatic, relative to the upper surface of the clay (**Figure 15.1**, in which the upper aquifer is omitted for clarity). These conditions are known as *artesian*. A borehole drilled through the clay into the chalk would overflow: such a device is known as an *artesian well*. Originally, the fountains in Trafalgar Square, London, operated on this principle and did not need to be pumped.

The pore water pressure at the upper surface of the underlying aquifer cannot exceed the vertical total stress due to the weight of overlying stratum without the latter becoming unstable as the vertical effective stress is reduced to zero. Excessive pore water pressures at depth in nature may result in *boiling* or *piping* of water up through the overlying soil, or *quicksand* if the overlying material is more permeable. The creation of unstable uplift conditions by the removal of overlying material to form an excavation is discussed in Chapter 16 *Groundwater flow*. *Sub-artesian* conditions, in which the piezometric level in the underlying aquifer is above the water table in the overlying material (but below ground level), are probably more common than true artesian conditions.

15.5 Underdrainage

If the piezometric level in the underlying aquifer is below the water table in the upper aquifer, the direction of natural groundwater flow across the intervening aquiclude is reversed and is slowly downward. Pore pressures in the clay will be below the hydrostatic line drawn from its upper surface, as shown in **Figure 15.2**. This is known as *underdrainage*, and represents

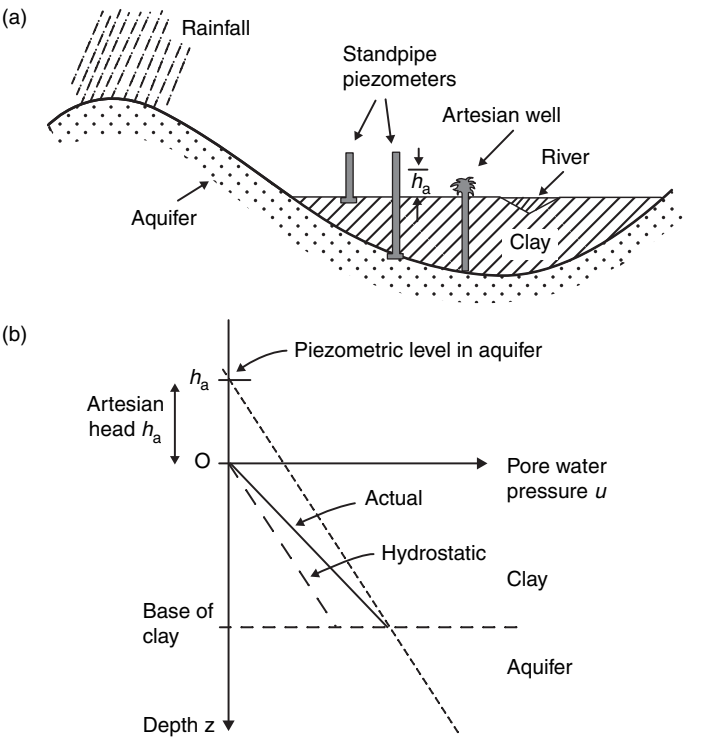


Figure 15.1 (a) Artesian groundwater conditions and (b) associated pore water pressures in the confining clay aquiclude
Reproduced from Powrie (2004)

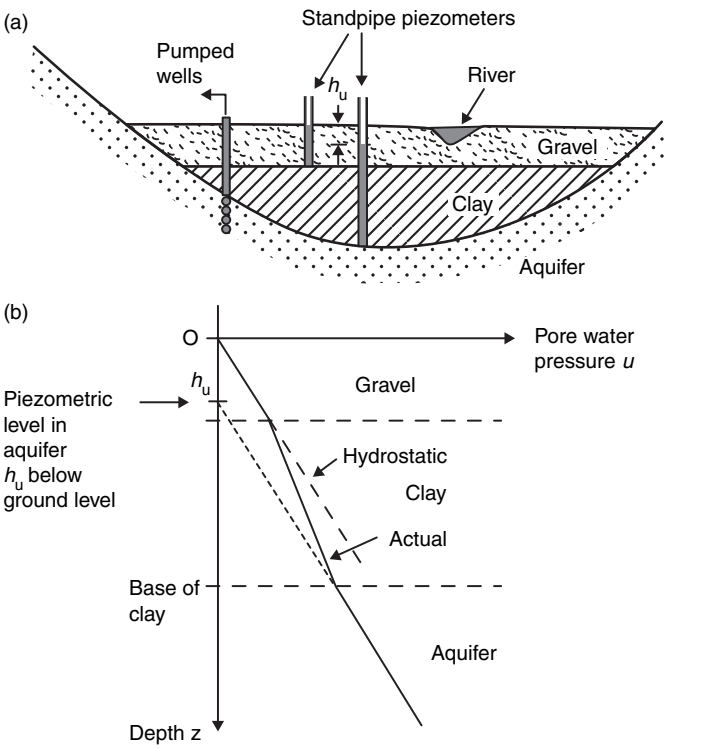


Figure 15.2 (a) Underdrainage and (b) associated pore water pressures in the intervening clay aquiclude
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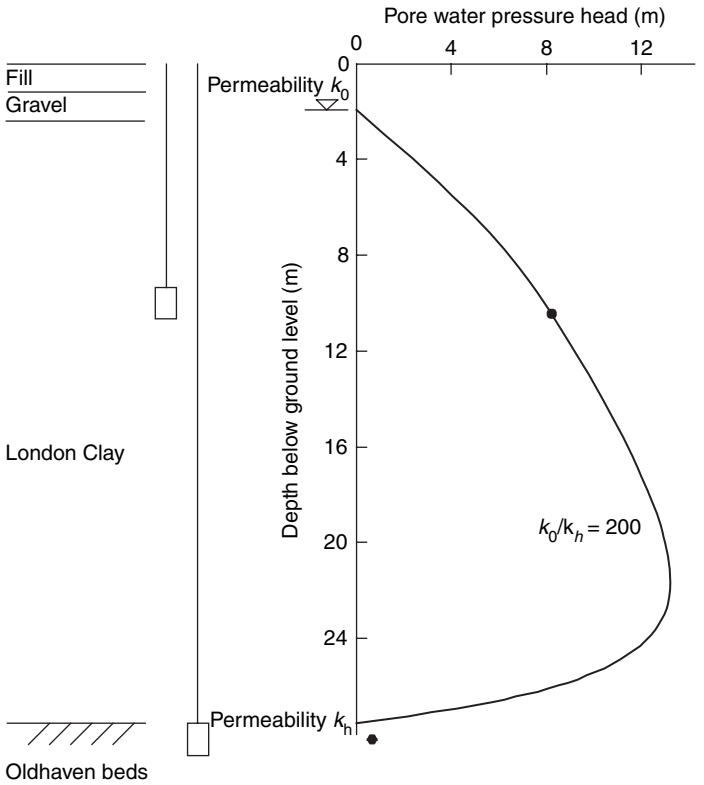


Figure 15.3 Variation of pore water pressure head with depth in a clay aquiclude in which the permeability decreases with depth
Modified from Vaughan (1994)

current conditions in London because the historic abstraction of groundwater by pumping from the chalk for industrial and domestic use has lowered the piezometric level within the chalk to below the water table in the alluvial gravels.

The water table in the upper aquifer is sometimes described as *perched*: this term is also applied to zones of higher pore water pressure resulting from extensive but discontinuous lenses of clay or other low permeability material.

It is important to realise that the simple linear distributions of pore water pressure with depth in the clay aquicludes indicated in **Figures 15.1(b)** and **15.2(b)** tacitly assume that the permeability of these layers is uniform with depth. For a clay, it is more likely that the permeability will decrease with vertical effective stress and hence with depth, giving an equilibrium pore pressure distribution more like the one shown in **Figure 15.3**.

15.6 Conditions above the water table

Above the water table in an unconfined aquifer, pore water will be retained in the soil by capillary action. Pore water pressures will fall with height above the water table (i.e. they will become negative) at the hydrostatic gradient, until a suction is reached at which sufficient air has entered the soil to disrupt the continuity of the water phase within the pores. This suction will depend on a variety of factors including the particle size and the particle size distribution, the density, and whether the soil is wetting or drying. Broadly, suction will increase dramatically with decreasing particle size. Thus gravel, with its large particle size, would become sufficiently unsaturated to disrupt the continuity of the liquid phase at a very low suction. A fine sand would hold a suction in the order of one to a few tens of kPa before becoming unsaturated; silt, several tens of kPa; and a clay, tens to a few hundred kPa. While soil suction can cause unwelcome and damaging shrinkage and settlements, in terms of the stability of a geotechnical structure (such as a slope) it is usually a good thing. However, because it is difficult to calculate and may be unreliable (for example, varying between seasons and with climate and vegetation cover), suctions are generally ignored in most engineering calculations and the pore pressures above the water table are often taken as zero. An exception may need to be made in the calculation of settlements when suctions can have a significant effect.

15.7 *In-situ* horizontal effective stresses

In-situ horizontal stresses are difficult to calculate, as they may lie anywhere between the active and passive limits for the soil, which is too wide a range to be useful (see Chapter 20 *Earth pressure theory*). One possibility is to calculate the horizontal effective stress using the theory of elasticity and the fact that during deposition of a soil, the lateral strains are zero. However, this requires estimates to be made of the elastic parameters. This is difficult for soils because of their dependence on stress history, stress state, stress path and stress change.

The most common approaches to estimating the *in-situ* horizontal stresses in a soil deposit are empirical. The *in-situ* horizontal effective stress is characterised by the earth pressure coefficient $K_0 = \sigma'_h / \sigma'_v$. For a normally consolidated soil (i.e. one that has not previously been subjected to a vertical effective stress greater than that which currently acts), Jaky's equation:

$$\frac{\sigma'_h}{\sigma'_v} = K_0 = (1 - \sin\phi') \tag{15.5}$$

is frequently used (Jaky, 1944).

For overconsolidated clays which have previously been subjected to a maximum vertical effective stress of $\sigma'_{v,max}$, some of the horizontal stress caused on first loading tends to remain 'locked in' on unloading, resulting in an increase in K_0. There are several empirical equations that can be used to capture this, for example Mayne and Kulhawy (1982) suggest

$$\frac{\sigma'_h}{\sigma'_v} = K_0 = (1 - \sin\varphi')(\mathrm{OCR})^{\sin\varphi'} \tag{15.6}$$

where OCR is the overconsolidation ratio, OCR = $\sigma'_{v,max} / \sigma'_{v,current}$.

15.8 Summary

- Profiles of pore water pressure and effective stress with depth are generally needed for use in geotechnical calculations.

- Vertical total stresses will normally increase geostatically with depth at a gradient equal to the bulk unit weight of the soil.
- If the groundwater is static, pore water pressures will increase hydrostatically with depth at a gradient equal to the unit weight of water, provided the water phase is continuous (e.g. the soil is saturated).
- Vertical effective stresses must be calculated by subtracting the pore water pressure from the vertical total stress.
- The horizontal effective stress profile may be estimated from the vertical effective stress profile, using an empirical relationship appropriate to the stress history of the deposit.

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Chapter 16
Groundwater flow

William Powrie University of Southampton, UK

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This chapter describes the fundamentals of steady-state groundwater flow in a saturated soil, based on Darcy's Law and the concept of soil permeability or hydraulic conductivity. Simple flow regimes including radial flow to single wells are described, and the use of plane flownets in plan and cross-section to calculate flowrates and pore water pressures is discussed. Flow in strata of anisotropic permeability and across a boundary between two soils having different permeabilities is briefly considered. The importance of controlling pore water pressures in the vicinity of an excavation is illustrated with reference to a case study, and some common methods of groundwater control are described. Finally, the concepts of transient flow and consolidation are summarised.

16.1 Darcy's Law

For almost all practical purposes, the flow of groundwater through a soil is governed by Darcy's Law,

$$q = Aki \tag{16.1}$$

where q is the volumetric flowrate in m^3/s; A is the gross area through which flow occurs (m^2); k is a parameter known as the *hydraulic conductivity* or the *permeability* (m/s), which is a measure of the ease with which water can flow through the soil; and i is the hydraulic gradient in the direction of flow being considered, which is defined as:

$$i = -\frac{\mathrm{d}h}{\mathrm{d}x}. \tag{16.2}$$

i.e. the rate of loss of total head, h (m), with distance x (m) along the direction of flow. The hydraulic gradient is therefore dimensionless. The head used is always the total head, measured above an arbitrary datum which, once chosen, remains fixed. In **Figure 16.1**, the total head for the point at the centre of the left-hand cross-section, A, above the datum indicated, is h_A. This is the level above datum to which water will rise in a standpipe piezometer with its tip at the point of interest A. Other types of head, which are not relevant for the purposes of Darcy's Law, are:

- *the elevational head*, i.e. the elevation of the point above the chosen datum, z_A;
- *the pressure head*, which is defined as the pore water pressure at the point of measurement divided by the unit weight of water, and is indicated by the height to which water in the piezometer rises above the measurement point – in the case of **Figure 16.1**, this would be $h_A - z_A$;
- *the velocity head*, defined as $v^2/2g$ where v is the velocity of groundwater flow, which is usually negligibly small in groundwater problems, and g is acceleration due to gravity.

Darcy's Law is illustrated schematically in **Figure 16.1**. This figure also shows the basis of the simplest form of hydraulic conductivity measurement, which involves subjecting a specimen of known cross-sectional area A to a known head drop h (from which the hydraulic gradient h/x may be calculated), and measuring the corresponding volumetric flowrate.

The main subtleties or potential pitfalls of Darcy's Law are that:

1. It applies only if the flow is laminar. However, unless the ground is very highly permeable, this will almost always be the case in groundwater applications.
2. The flowrate q may be divided by the cross-sectional area A to obtain an apparent, superficial or Darcy flow velocity $v_D = q/A$. This is not, however, the true average fluid flow velocity, because in reality flow takes place only through the soil pores. The pores occupy a proportion of the total area of nA, where n is the soil porosity defined as the volume of pores ÷

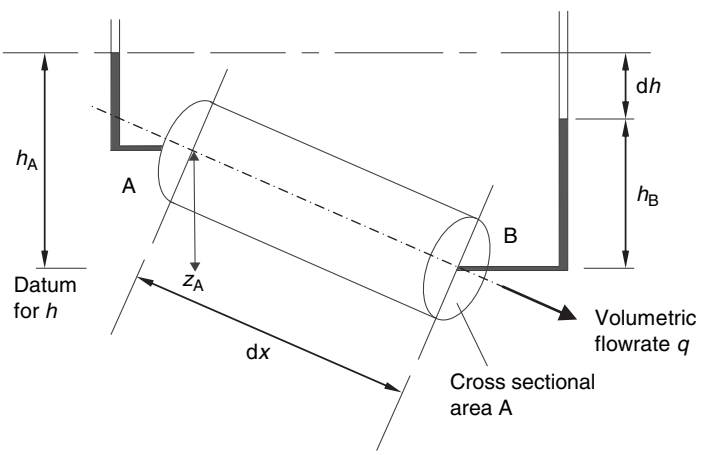


Figure 16.1 Schematic representation of hydraulic gradient and Darcy's Law

the total volume, or $n = e/(1 + e)$ where e is the void ratio. Thus the true average fluid flow velocity v_{true} is $(1 + e)/e$ times the superficial or Darcy flow velocity v_D,

$$v_{true} = \frac{(1+e)}{e} v_D \,. \tag{16.3}$$

3. The head loss h used in calculating the hydraulic gradient is in terms of total head, determined as the rise in water level above an arbitrary datum in an imaginary standpipe driven into the soil, with its tip at the point at which the head is to be measured.
4. The hydraulic conductivity depends on both the nature of the soil and the physical properties of the permeant fluid, in this case water.
5. Even in a uniform soil, the hydraulic conductivity depends hugely on the particle size ($k \propto D_{10}^2$) and to a lesser extent on the void ratio. In the ground it can vary massively from point to point because of natural inhomogeneities, and be strongly anisotropic (with horizontal k_h being much greater than vertical k_v) owing to soil fabric and structure. In finer grained and compressible materials such as clays and landfilled wastes, the hydraulic conductivity is especially likely to vary – often significantly – with stress and stress history, and hence with depth (Chapter 15 *Groundwater profiles and effective stresses*). In construction, what is usually sought is a reliable estimate of the effective bulk hydraulic conductivity of the volume of ground contributing to the flow of interest. This can be very difficult to estimate, even to within an order of magnitude, especially from small scale or laboratory element tests.
6. Darcy's Law forms the basis of nearly all groundwater flow calculations, whether they are carried out by hand or numerically. Darcy's Law can generally be applied to fissure flow in rocks (e.g. chalk) – in which case the hydraulic conductivity used is that representative of a volume of rock whose linear dimensions are large in relation to the typical fissure spacing.

Items (4), (5) and (6) are discussed in section 16.2.

16.2 Hydraulic conductivity (permeability)

The hydraulic conductivity, k, used in Darcy's Law depends on both the soil and the properties of the permeant fluid – in this case, water – according to the Equation (16.4):

$$k = \frac{K \gamma_f}{\eta_f} \tag{16.4}$$

where K (m^2) is the intrinsic permeability of the soil matrix, γ_f is the unit weight of the permeant fluid (kN/m^3), and η_f its dynamic viscosity (kNs/m^2). In geotechnical engineering, the hydraulic conductivity is often known simply as the permeability.

The practical implications of this are for the most part fairly limited. Obviously, the permeability will change if the permeant fluid is not water, but the dependence of the dynamic viscosity on temperature could also be significant in some cases (η_f reduces, and hence k increases by a factor of about 2 over a temperature rise from 20 to 60°C).

The hydraulic conductivity of a granular material is governed by the smallest 10% of the particles, and varies roughly with the square of the D_{10} particle size. Hazen's empirical formula:

$$k(\mathrm{m/s}) \approx 0.01\left[D_{10}(\mathrm{mm})\right]^2 \tag{16.5}$$

is often used to obtain a rough estimate of the hydraulic conductivity, even though it was originally intended for use only with clean filter sands. Thus the potential range of hydraulic conductivity even for uniform soils is very wide indeed, varying over perhaps ten orders of magnitude from 1 m/s for a clean, open gravel to 10^{-10} m/s for an unfissured clay (**Table 16.1**). This degree of variation is far greater than that of almost any other engineering parameter for natural materials – for example, the shear strength of a soft clay is a factor of about 10^5 less than that of high tensile steel.

Natural inhomogeneities such as lenses or zones of different materials (e.g. an infilled river channel) will cause the hydraulic conductivity in the ground to vary from point to point. Even in uniform soils, the *in situ* hydraulic conductivity may be anisotropic owing to the natural fabric or structure of the soil. In clays with no obvious layered structure, the hydraulic conductivity in the horizontal direction might be expected to be up to around ten times that in the vertical. If the soil displays an obvious layering, e.g. interlaminated bands of clay/silt and silt/sand, the ratio of horizontal to vertical hydraulic conductivity could easily be in the order of 10^2 or 10^3.

Some typical values of hydraulic conductivity are given in **Table 16.1**.

Indicative soil type	Degree of permeability	Typical hydraulic conductivity range m/s
Clean gravels	High	$>1\times10^{-3}$
Sand and gravel mixtures	Medium	1×10^{-3} to 1×10^{-5}
Very fine sands, silty sands	Low	1×10^{-4} to 1×10^{-7}
Silt and interlaminated silt/sand/clays	Very low	1×10^{-6} to 1×10^{-9}
Intact clays	Practically impermeable	$<1\times10^{-9}$

Table 16.1 Range of hydraulic conductivity for different soil and ground types
Data taken from Preene *et al.* (2000) (CIRIA C515)

In ground engineering, it is usual to use a fairly simple flow model based on a single value of hydraulic conductivity, at least in an initial calculation. It therefore becomes important that the value used is reasonably representative of the whole volume within which flow is taking place. Large scale, *in situ* tests that involve pumping on one or more wells, with an array of piezometers to monitor the resulting drawdown, are likely to give the most realistic estimates. Building a reliable picture from smaller scale tests can be difficult, not only because of the small volume of soil involved in the flow, but also because of the potential for disturbance during sample retrieval (for laboratory tests) or borehole installation (for rising or falling head tests in single boreholes). A review of the various methods of determining hydraulic conductivity for the purpose of large-scale groundwater control system design is given by Preene *et al.* (2000).

16.3 Calculation of simple flow regimes

Hand calculations based on Darcy's Law, combined with the condition of continuity of flow (i.e. the volumetric flow of water into an element of soil must be equal to the volumetric flow of water out, over a given increment of time, if the soil element is not changing in volume), can be carried out for simple flow regimes that are essentially two-dimensional in nature. Common situations include the following:

(i) Radial flow to a fully penetrating well in a confined aquifer (illustrated, with the terms defined, in **Figure 16.2**):

$$q = \frac{2\pi Dk(H-h)}{\ln\left(R_o / r_w\right)}. \qquad (16.6)$$

(ii) Radial flow to a fully penetrating well in an unconfined aquifer (illustrated in **Figure 16.3**):

$$q = \frac{\pi k\left(H^2 - h^2\right)}{\ln\left(R_o / r_w\right)}. \qquad (16.7)$$

Full details of the analyses leading to Equations (16.6) and (16.7) are given in Powrie (2004).

A large excavation of plan dimensions a (length) × b (breadth) surrounded by a ring of dewatering wells can be analysed approximately as a single well of equivalent radius r_e, provided that the aspect ratio of the excavation a/b is between 1 and 5, and the distance from the edge of the excavation to the recharge boundary, L_o, is greater than $3a$. The equivalent radius r_e is given by $(a + b)/\pi$, and the equivalent radius of influence R_o is $(L_o + a/2) \approx L_o$. Further details are given by Preene and Powrie (1992).

(iii) Plane flow: for excavations with close recharge boundaries, flowrates can be calculated by means of a flownet sketched within one half of the cross-sectional plane. The plane flownet is used to calculate the flowrate per metre run: this is then multiplied by the perimeter of the excavation to give the overall flowrate. If the excavation is long ($a/b > 10$) with close recharge boundaries ($L_o/a < 0.1$), plane flow to the long sides will dominate and end effects may be neglected. If the excavation is rectangular ($1 < a/b < 5$) with close recharge boundaries ($L_o/a < 0.3$), plane flow to the four sides dominates and corner effects may be neglected.

Plane flownet sketching is a useful and highly effective graphical technique for solving the Laplace Equation – governing the plane flow of groundwater through a soil of uniform or

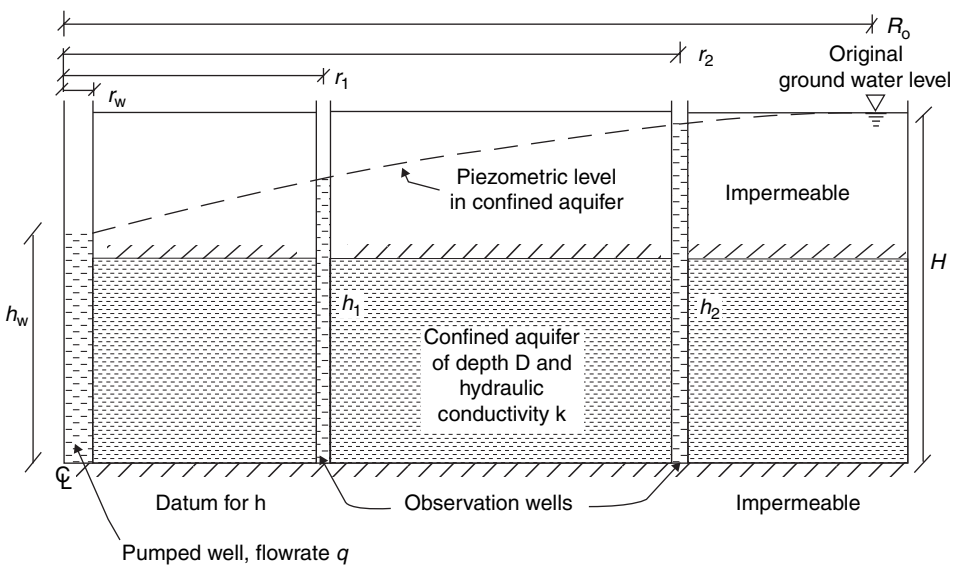


Figure 16.2 Radial flow to a fully penetrating well in a confined aquifer
Reproduced from Powrie (2004)

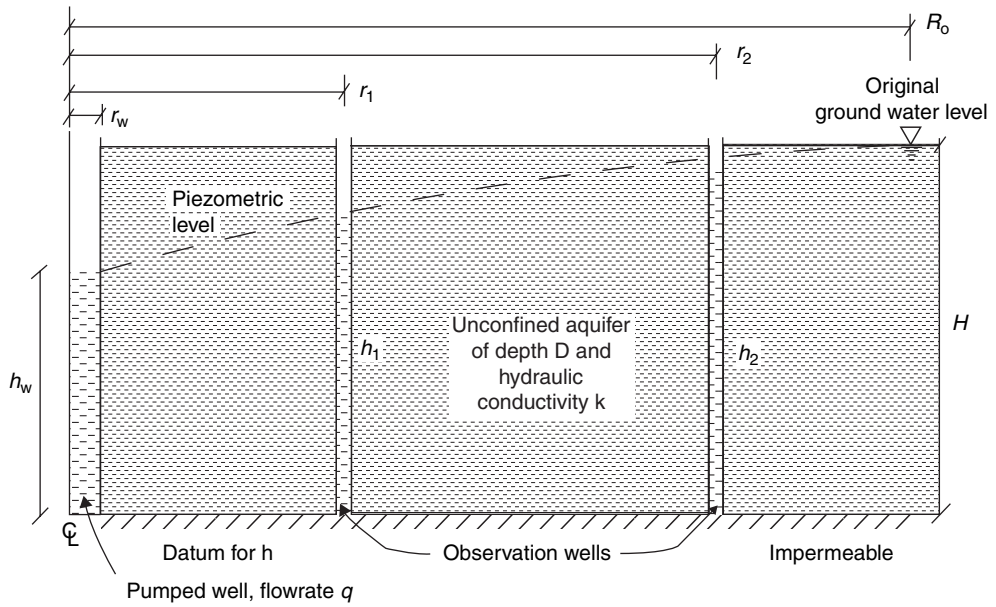


Figure 16.3 Radial flow to a fully penetrating well in an unconfined aquifer
Reproduced from Powrie (2004)

anisotropic hydraulic conductivity. The derivation of the equation, and a comprehensive introduction to flownet sketching, is given by Powrie (2004). In summary, plane flownet sketching involves determining the upper and lower equipotentials (i.e. lines of equal head) representing the source of water and the sink, and two bounding flowlines. The space so defined is then filled with a network of intermediate flowlines which intersect intermediate equipotentials at 90° to form elements known as 'curvilinear squares', whose length is equal to their breadth. (In practice, this means that a circle can be fitted inside each of them, although the circles will vary in diameter depending on the position of the element in the flownet.) In an unconfined aquifer, the need to determine the position of the top flowline (which represents the drawn down water table) as part of the general flownet sketching process represents a small additional complication. The flowrate per metre run is then calculated as:

$$q = kH \, N_F / N_H \qquad (16.8)$$

where H is the overall head drop between the upper and lower equipotentials (the source and the sink), N_F is the number of flowtubes (which is equal to the number of flowlines minus 1) and N_H is the number of equipotential drops (i.e. the number of equipotentials minus 1). A full description of the basic technique, which progresses by trial and error, is given by Powrie (2004).

An example of a plane flownet in the vertical plane is shown in **Figure 16.4**.

In cases where there is no similarity of vertical cross-sections and flow is primarily horizontal, flownets may be sketched in the horizontal plane as shown in **Figure 16.5**. The principles are the same, but the total flowrate is calculated by multiplying the flownet result (equation (16.8)) by the depth of the aquifer. An inherent assumption is that regions of vertical flow, for example up into the excavation, are small and do not significantly affect the basic premise of substantially horizontal flow.

Although the technique could be viewed as having been superseded by computer-based numerical methods, flownet sketching is quick and simple and has the great advantage of giving strong insights into the physicality of, and the main factors governing, the flow regime.

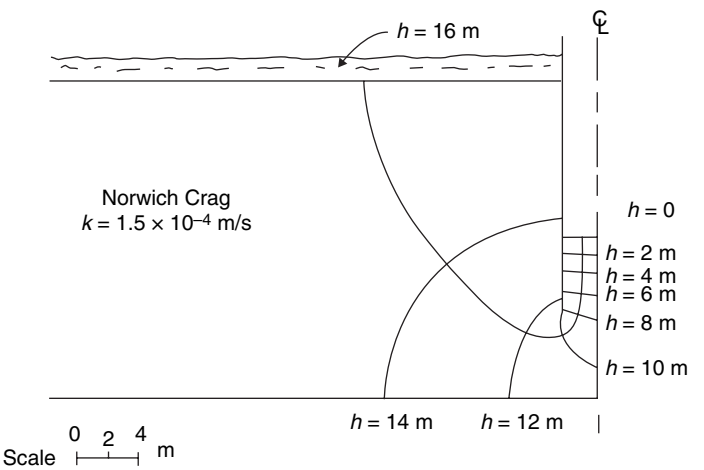


Figure 16.4 Example of a plane flownet in one half of the vertical cross-sectional plane for flow into a trench excavation
Reproduced from Powrie (2004)

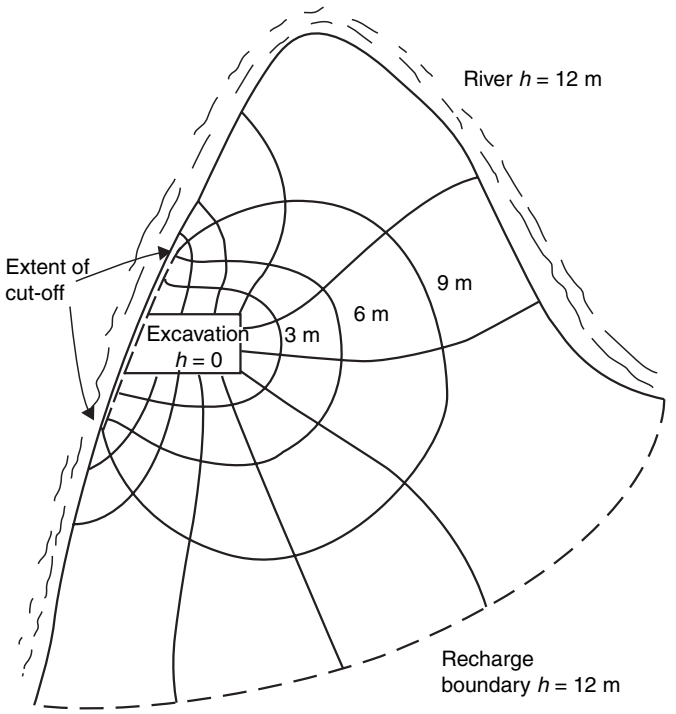


Figure 16.5 Example of a horizontal plane flownet, for flow to an excavation in a deep chalk aquifer on the Medway peninsula
Reproduced from Powrie (2004)

A finite difference solution to a plane flow problem can be obtained by establishing a grid of nodes and using Darcy's Law to write an expression for the flowrate into each node – in terms of the head difference between it and the adjacent nodes, the implied area of flow, and the soil hydraulic conductivity. At the steady state, the net flowrate into or out of each node is zero (except of course for the nodes on the constant head or equipotential boundaries). To obtain values of head that satisfy the condition of zero net flow into or out of each internal node, and give a sum of flowrates *into* the boundary nodes on the upper equipotential (the source) equal to the sum of the flowrates *out* of the boundary nodes on the lower equipotential (the sink), solution is by trial and error and is greatly facilitated by the use of a spreadsheet.

16.4 More complex flow regimes

Flownets can be sketched for anisotropic soils in the usual way, provided that the real geometry of the cross-section has first been 'transformed' to account for the difference between the vertical and the horizontal hydraulic conductivities. The true horizontal dimensions, x, must be multiplied by $\sqrt{(k_v / k_h)}$, where k_v and k_h are the hydraulic conductivities in the vertical and horizontal directions respectively. As k_v is usually less than k_h, this involves a shrinking of the horizontal distances on the transformed section. Equation (16.8) for the calculation of flowrates holds, except that the effective hydraulic conductivity of the transformed section, k_t, that must be used for this purpose is the geometric mean of k_v and k_h – in other words $\sqrt{(k_v k_h)}$. Derivations and further details are given in Powrie (2004).

Flownet sketching can also accommodate the presence of two or more strata of different hydraulic conductivities. If the ratio of hydraulic conductivities is in the order of hundreds, one will act as a reservoir compared with the other (i.e. any head drop within it will be negligible compared with that in the less permeable stratum), and this is the appropriate approximation to make. If the ratio of hydraulic conductivities is in the order of one to 100, the flownet should be sketched so that the flowlines deflect through an angle $(\beta_1 - \beta_2)$ as they pass from a soil with hydraulic conductivity k_1 into a soil with smaller hydraulic conductivity k_2, such that

$$\frac{\tan \beta_1}{\tan \beta_2} = \frac{k_1}{k_2}. \qquad (16.9)$$

This is analagous to the refraction of light as it passes between media of different optical densities. The derivation is given by Powrie (2004).

A comprehensive guide to flownet sketching for these more complex cases and even transient flow is given by Cedergren (1989), but given the ready availability of spreadsheets and low-cost personal computing it is probably easier to solve such problems using the finite difference approach and a spreadsheet, or one of the commercial computer programs available for modelling groundwater flow. The underlying principles, however, remain Darcy's Law and continuity of flow, and success still depends on understanding the physical constraints to the flow domain and the judicious selection of numerical values of soil hydraulic conductivity.

16.5 Groundwater control for stability of excavations

Whenever an excavation is made below the natural water table, at least in a non-clay soil, groundwater and pore water pressures must be controlled to prevent flooding and potential instability of the excavation. This can be achieved by physical means, e.g. grouting or a cut-off wall, pumping from dewatering wells placed in or around the excavation, or most usually a combination of both. Even when the sides of the excavation are supported by retaining walls, pumping may still be necessary to prevent instability of the base. Pumping will also often reduce the pore water pressures acting on the outside of the retaining walls, enabling them to be constructed more economically. In built-up areas, consideration should be given to mitigating the potential for settlement damage to buildings resulting from ground settlements associated with reducing the pore water pressures outside the excavation. Powrie and Roberts (1995) provide a case history on this subject.

As outlined earlier in this chapter, flow calculations are required (i) to estimate the likely flowrate that a dewatering

system will have to pump, and (ii) to calculate the reductions in pore water pressure that a dewatering system will achieve. The pore water pressures will feed in turn into calculations relating to the stability of sheet pile and other retaining walls, excavation side slopes and the excavation base. Most of these calculations are discussed elsewhere in this manual, but the importance of maintaining stability of the base will be discussed briefly here.

The base of an excavation will become unstable if the pore water pressures within a certain depth of the excavated surface become equal to the vertical total stress due to the weight of soil. The 'certain depth' will depend to some extent on the width of the excavation. If the sidewalls are of the embedded type, they will offer some additional frictional resistance to uplift, but the effect will diminish towards the centre of the excavation especially if the excavation is wide.. Ignoring any benefits of sidewall friction, it can easily be shown that instability or uplift will occur when the hydraulic gradient associated with upward flow to the base of the excavation is approximately 1, see Powrie (2004); or, when the pore water pressure on the base of a plug of low permeability soil exceeds the vertical total stress due to its weight.

The perils of ignoring the potential for unrelieved pore water pressures at depth to cause major damage, and possibly even loss of life, should not be underestimated and are illustrated by the case study summarised in **Figure 16.6**. This relates to an excavation for a pumping station near Weston-super-Mare which the contractor attempted to make without taking steps to reduce the pore water pressures in the underlying silty sand. As formation level was approached, the base became unstable and the excavation rapidly filled with water. This was quite predictable: at full depth, the pore water pressure at the point A on the base of the 6 m depth of soft clayey silt that remains below the excavation floor is 13 m × 10 kN/m^3 = 130 kPa, which is substantially in excess of the total vertical stress due to the weight of the clayey silt of 6 m × 18 kN/m^3 = 108 kPa. The remedial works necessitated by the failure were substantially more expensive than the cost of a pre-emptive groundwater control scheme, quite apart from the damage to plant and the potential for personal injury or even loss of life.

Methods of groundwater control by pumping are summarised in **Table 16.2**. Further details are given in Powrie (2004), Cashman and Preene (2001), Preene *et al.* (2000) and Powers (1992). The suitability of each method depends on the pumped flowrate, and hence the soil hydraulic conductivity and the drawdown, as shown schematically in **Figure 16.7**.

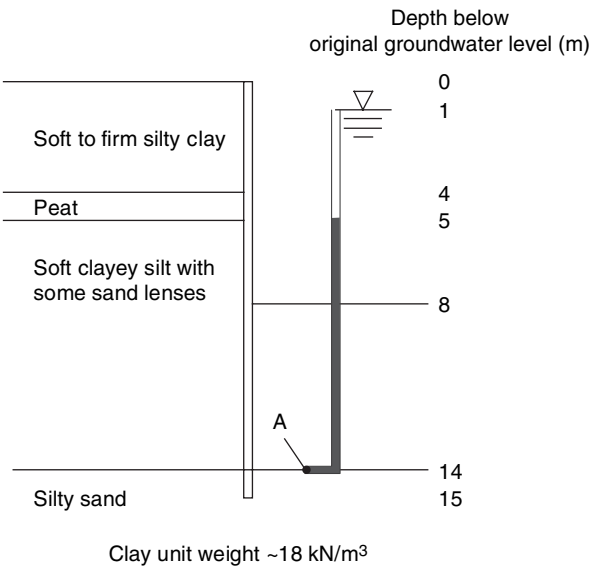


Figure 16.6 Case study demonstrating the importance of controlling pore water pressures to an adequate depth below the excavation floor

Method	Thumbnail description	Comments and case studies
Open sump	Literally an open sump into which groundwater is allowed to flow under gravity, and pumped away by a stand-alone diesel or electric site pump	Subject to limitations on space and accessibility, this method is generally only suitable for modest drawdowns (up to about 2 m) in well interlocked, open gravels
Vacuum wellpoints	Closely spaced small wells (typically in a line 1–3 m apart) approximately 50 mm diameter connected by risers to a common header main, pumped by a surface vacuum pump	Maximum drawdown (owing to limitations of vacuum lift) about 6 m, but can be used in multiple stages. Comfortable with flowrates typical of sandy soils
Deep wells	Large diameter wells (e.g. 200 mm diameter screens in 350 mm diameter bores) pumped individually by slimline electrical submersible pumps. Widely spaced so recovery of water table between wells is significant and well depth must allow for this	Can deal with higher flowrates and drawdowns than wellpoints. Restriction on depth of drawdown likely to be economic rather than physical. Vacuum can be applied via a separate system in lower permeability ground. Powrie and Roberts (1995); Bevan *et al.* (2010)
Ejectors (sometimes called eductors)	Nozzle and venturi devices installed at depth in deep but small diameter boreholes (minimum 50 mm), driven by high pressure water pumped from the surface	Will draw air as well as water so will create a vacuum automatically if the well is sealed. Particularly useful for lower permeability soils. Powrie and Roberts (1990); Roberts *et al.* (2007)

Table 16.2 Methods of groundwater control by pumping

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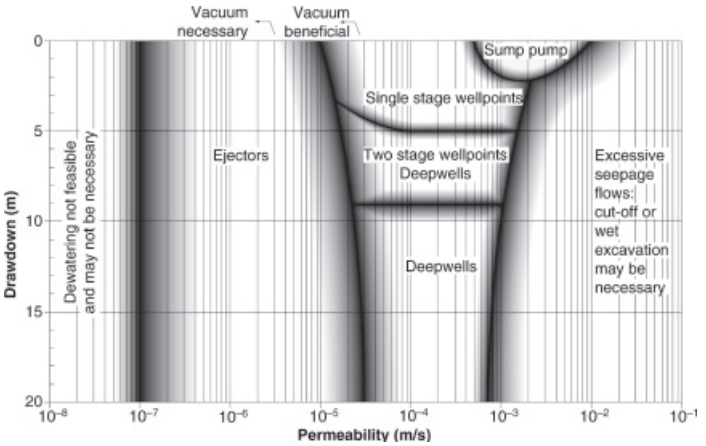


Figure 16.7 Range of application of dewatering techniques
Reproduced with permission from CIRIA C515, Preene *et al.* (2000), www.ciria.org

Soil parameter	Medium sand	Fine sand	Silt	Clay
Hydraulic conductivity k (m/s)	10^{-3}	10^{-4}	10^{-6}	10^{-8}
Stiffness in one-dimensional compression E'_o (MPa)	100	50	10	2
Time t for completion of transient flow with drainage path length d = 50 m	4 min	1.4 h	1 month	40 years

Table 16.3 Indicative times for completion of transient flow with a drainage path length of 50 m
Data taken from Preene *et al.* (2000) (CIRIA C515)

16.6 Transient flow

So far, we have considered steady-state flow, in which the volume of water flowing into a soil element in a given time is the same as the volume of water flowing out, i.e. there is no change in the volume of the soil element. This is not always the case; if the volume of the soil element is changing over time (for example in response to a change in the external loading regime), the flow into the element will be greater than the flow out if the soil volume is increasing (swelling), or less than the flow out if the soil volume is decreasing (compression). Non steady-state flow is known as *transient flow*, and the time-dependent process of volume change associated with it is known as *consolidation* (or possibly *swelling* if it is a volume increase).

For the effective stress to increase, the soil skeleton must compress: this requires water to flow from the pores, which cannot occur instantaneously because the rate of flow is limited by the hydraulic conductivity of the soil. Thus an increase in total stress results initially in an increase in the pore water pressure. The increase in pore water pressure causes local hydraulic gradients, in response to which water will flow out of the soil. As water flows from the pores, the pore water pressure falls back towards its equilibrium value, the soil compresses and the effective stress increases to make up for the reduction in pore water pressure. Eventually, the pore water pressures return to the initial equilibrium values, and transient flow stops. The soil has compressed, the effective stress has increased to mirror the increase in total stress, and consolidation is complete.

The rate of consolidation increases with the soil hydraulic conductivity k. The amount of compression associated with the eventual increase in effective stress decreases as the soil stiffness in compression, E'_o, increases. The parameters k and E'_o are combined to give the consolidation coefficient, c_v,

$$c_v = \frac{kE'_0}{\gamma_w} \qquad (16.10)$$

where γ_w is the unit weight of water. The consolidation coefficient c_v will obviously vary with soil stiffness and hydraulic conductivity, and hence with stress history, stress state and stress path. For stiff soils subjected to small changes in stress, it might be reasonable to assume an equivalent single value of c_v in analysis. However, for problems involving softer sediments and larger changes in stress, an approach which allows c_v to vary with both depth and stress will probably be required. The consolidation time also depends on the distance to the drainage boundary, d, and the flow geometry. For simple flow geometries approximating to one-dimensional flow, consolidation is usually considered to be substantially complete after a time t such that the dimensionless group T defined in Equation (16.11) is approximately equal to one:

$$T = \frac{c_v t}{d^2} = 1 \qquad (16.11)$$

In principle, all soils will undergo transient flow in response to a change in total stress or boundary pore water pressure. However, consolidation is normally only of practical importance in saturated, low permeability, fine grained soils. The reason for this, as illustrated in **Table 16.3**, is that sandy soils are so relatively stiff and more permeable that the timescale for transient flow is very short – perhaps a matter of minutes rather than months or even years.

It is important to realise that, while much practical soil mechanics theory is relatively straightforward, huge simplifications are involved both in idealising the complex behaviour of the soil and in choosing often a single value to characterise highly variable parameters such as hydraulic conductivity, soil stiffness and consolidation coefficient. This is why experience, judgement and a respect for reality are so important in translating theory into geotechnical engineering practice.

16.7 Summary

- The flow of water through the ground is governed by Darcy's Law, and controlled by a parameter known as the *hydraulic conductivity* or *permeability.*

- Hydraulic conductivity varies widely across the range of soils and can be difficult to determine with confidence, especially on the basis of tests involving small volumes of soil, owing to scale and fabric effects.
- Simple flow regimes can be analysed mathematically or by the use of plane flownet sketching.
- Anisotropy, and the deflection of flowlines as they pass between soil strata of differing hydraulic conductivity, can be taken into account if needed.
- Steps must be taken to lower groundwater levels and pore water pressures in the vicinity of an excavation below the natural water table, to prevent instability of the excavation sides or base. This is usually achieved by pumping from wells in a process known as construction detwatering.
- Transient flow occurs as a soil is changing in volume in response to a change in boundary stress. This leads to the process known as consolidation, which is usually only significant for low-permeability (clay) soils.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

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Chapter 17

Strength and deformation behaviour of soils

John B. Burland Imperial College London, UK

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This chapter explains the basic strength and stiffness properties of soils by frequent reference to their particulate nature. The results of simple shear box tests on sand and clay are presented and compared with the Coulomb strength criterion – $\tau_f = c' + \sigma'_n \tan \varphi'$. It is demonstrated that the angle of shearing resistance φ' depends on a number of factors, including the effective confining pressure, the initial relative density and the stress history. The key roles played by contractancy and dilatancy in controlling undrained strength are explained. The phenomena of critical state strength and residual strength are described, and the extension of the Coulomb strength criterion to the widely used Mohr–Coulomb strength criterion is explained, together with its limitations. The latter part of the chapter deals with the deformation properties of soil and its behaviour under one-dimensional compression. The important influence of inter-particle bonding is introduced, leading onto the concept of yield. In geotechnical engineering, extensive use is made of elasticity in the solution of boundary value problems – such as the settlement of foundations. The behaviour of an ideal isotropic porous elastic material is explored in detail and compared with real soils. The important phenomenon of non-linear stress–strain behaviour is introduced briefly.

17.1 Introduction

This chapter deals exclusively with the behaviour of fully saturated soils. Discussion of the behaviour of unsaturated soils may be found in Chapters 14 *Soils as particulate materials*, 32 *Collapsible soils*, 33 *Expansive soils*, 30 *Tropical soils* and 29 *Arid soils*. As stressed in Chapter 14, a saturated soil of whatever type may be considered as a skeleton of solid particles in contact with each other and with the surrounding voids – which are filled with water. Nevertheless, for the purposes of analysis, it is necessary to idealise the soil as a continuum in which the sizes of the particles are neglected in relation to the sizes of the elements of soil that are being considered. It is essential in making this idealisation to remember that it is the particulate nature of soil that controls its behaviour.

Some of the key controlling features of a particulate material listed in the conclusions to Chapter 14 are:

(i) the effective stress is that part of the total stress that is transmitted through the particle contact points and controls soil behaviour;

(ii) the closeness of pack of the particles (i.e. the void ratio e) plays an important role in determining the stiffness, strength and permeability of a soil;

(iii) the self-weight of the soil generates the effective confining pressures that govern its strength and stiffness;

(iv) when sheared, loose soils tend to contract and dense soils tend to dilate;

(v) particle grading, shape and micro-fabric are important in determining soil properties;

(vi) the presence of discontinuities through the mass of the soil, such as shear and bedding planes, can exert a key influence on the behaviour of the mass.

The element of soil shown in **Figure 17.1(a)**, with a pore pressure u acting in the voids, is idealised as an element of continuum in **Figure 17.1(b)**. When considering the normal stress σ and shear stress τ acting on any arbitrary plane through the continuum, it is also necessary to consider the pore pressure u acting within the plane. Therefore the effective stresses acting on the plane are $\sigma' = (\sigma - u)$ and τ. In this chapter, strength and stiffness properties of soil are introduced and explained in simple mechanistic terms by referring to their particulate nature. References are given to aid more advanced study.

17.2 Analysis of stress

The analysis of stress is thoroughly covered by most textbooks on applied mechanics and strength of materials, and will be dealt with only briefly here. It is important to appreciate the sign conventions that are adopted in soil mechanics. In particular, compressive stress is taken as positive as tensile stresses seldom occur in practical problems. **Figure 17.2(a)** shows a two-dimensional element referred to Cartesian coordinates (x, z) acted on by compressive stresses σ'_x, σ'_z and τ_{xz} $(= \tau_{zx})$. A convenient sign convention is that, when standing at the origin and looking at the two nearest faces of the element, positive stress is in the direction of increasing x and increasing z. Thus the positive stress σ'_x is acting normal to the constant x-plane directed towards increasing x. Similarly the positive shear stress τ_{xz} is acting parallel to the constant x-plane directed towards increasing z. Note that for equilibrium, $\tau_{xz} = \tau_{zx}$, which satisfies the above

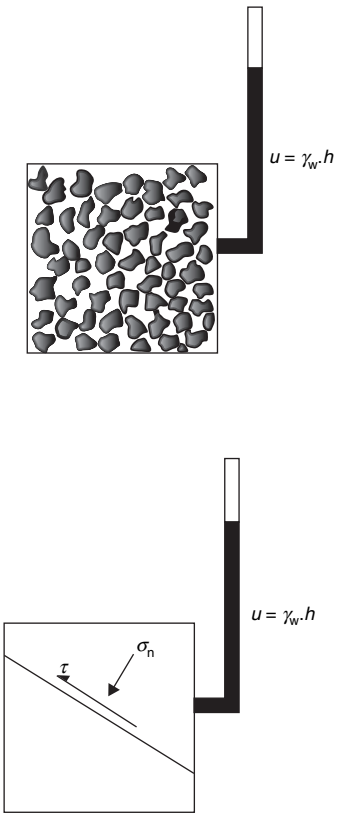


Figure 17.1 (a) An element of particulate material; (b) the element idealised as a continuum

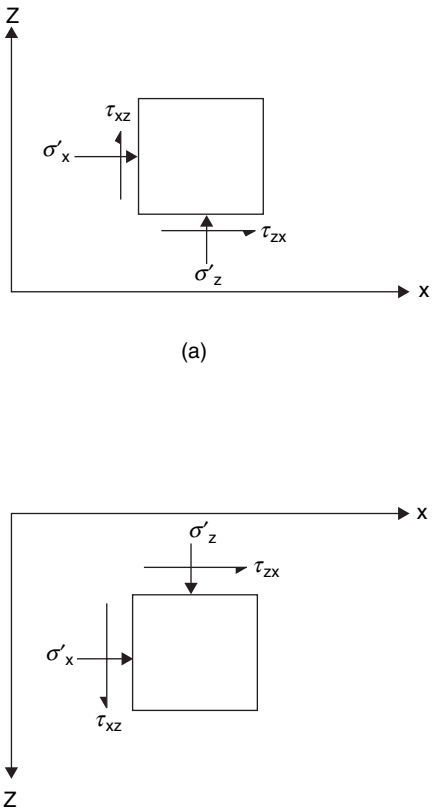


Figure 17.2 Sign convention for positive stresses (a) z positive upwards; (b) z positive downwards

sign convention. Also, for equilibrium, there are stresses acting on the other two faces in opposite directions from their counterparts. If z is taken as increasing downwards, which is common in geotechnical problems, then the direction of positive stresses acting on the element are as shown in **Figure 17.2(b)**.

17.2.1 Mohr's circle of stress

The stresses shown in **Figure 17.2** may be resolved to find the associated stresses acting on any plane through the element. The equations for such transformation of stress are given in most textbooks. In soil mechanics, extensive use is made of the well-known graphical procedure for transformation of stress by means of the Mohr's circle geometric construction – see, for example, Atkinson (1993). This is a very important tool in soil mechanics as it relates to the Mohr–Coulomb strength criterion which is universally used in soil mechanics. When using the Mohr's circle construction it is often not appreciated that it is necessary to make a small change to the sign convention for the shear stresses shown in **Figure 17.2**. For the geometric construction to work it is necessary to plot anti-clockwise shear stresses as positive and clockwise shear stresses as negative. Thus the shear stress τ_{zx} shown in **Figure 17.2(a)** is plotted as positive in a Mohr diagram, whereas the shear stress τ_{xz} is plotted as negative. It is important to stress that this sign convention only applies to Mohr's circles and not to stress analysis in general.

Figure 17.3(a) shows an element acted upon by major and minor principal stresses σ_I' and σ_{III}'. The stress state can be represented by a circle on a Mohr diagram of shear stress τ versus normal stress σ, as shown in **Figure 17.3(b)**. The points representing the major and minor principal stresses plot on the σ' axis since there are no shear stresses on the planes on which they act.

An important feature of a Mohr's circle is the pole (sometimes called the 'origin of planes'). If the pole can be located then the stresses acting on any plane can be found very simply. The pole is located by drawing a line parallel to the plane on which the stress acts through a known stress on the circle. Where the line intersects the circle again locates the pole. Thus, in **Figure 17.3(b)**, the line I–I is parallel to the principal plane on which σ_I' acts and passes through the point on the Mohr's circle representing σ_I'. Where this line intersects the circle again represents the pole.

Having found the pole, the stresses acting on any other plane may be found by drawing a line through the pole, parallel to the plane. Where the line intersects the circle again gives the required stress state. For the case shown in **Figure 17.3(b)**, the point E represents the stresses acting on the plane CD through the element. It can be seen that the shear stress is positive so

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that the shear stresses acting on either side of the plane *CD* are in an anti-clockwise direction as shown.

17.3 The drained strength of soils

17.3.1 Coulomb's equation

In 1773 Coulomb read a paper to the French Royal Academy of Science with the title 'On the application of the rules of maximum and minimum to some statical problems, relevant to architecture' (Heyman, 1997). Besides dealing with the bending of beams and the stability of arches, Coulomb discussed the strength of masonry columns and pressures of 'fresh earth' against retaining walls. This was a most remarkable contribution which is still widely used in geotechnical practice.

For this work, Coulomb assumed that sliding takes place along an arbitrary plane on which the material has both friction and cohesion. By varying the orientation of the sliding plane he was able to work out the minimum value of the column load to give failure and the maximum thrust against a gravity-retaining wall. Although he never explicitly stated it as such, the strength criterion adopted by Coulomb, expressed in terms of effective stresses, is given by the equation:

$$\tau_f = c' + \sigma'_n \tan \phi' \qquad (17.1)$$

where τ_f, is the shear stress at failure on the plane and σ'_n is the effective stress normal to the plane.

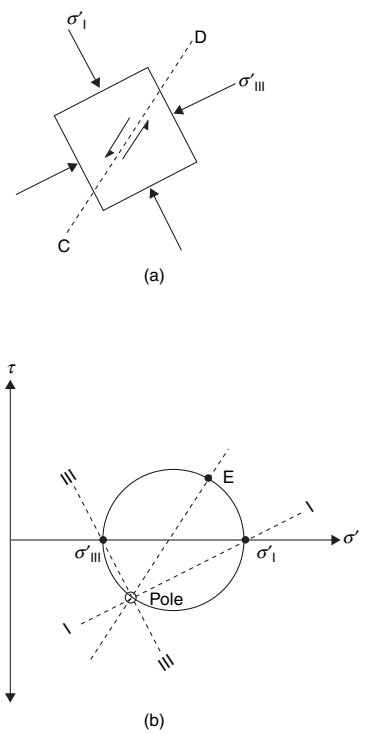


Figure 17.3 Application of Mohr's circle for the transformation of stress. For this construction anti-clockwise shear stresses are plotted as positive

Where c′ is the effective cohesion and φ' is the effective angle of shearing resistance (frequently referred to as the effective friction angle). However the author prefers the term 'effective angle of shearing resistance' as its value depends on other (more dominant) factors than inter-particle friction, such as particle shape and grading – see Chapter 14 *Soils as particulate materials*. Equation (17.1) is known as Coulomb's equation and it plots as two straight lines in a Mohr diagram as shown in **Figure 17.4** for positive and negative shear stress. In the following sections we will examine the results of laboratory tests using a simple shear box and compare them with the Coulomb equation.

17.3.2 Results of drained shear box tests on granular materials

For drained shearing, the shear stress is applied slowly so that the pore water can drain into or out of the soil sample as it dilates or contracts. Roscoe (1953), working at Cambridge University, developed a sophisticated shear box within which a soil sample is caused to deform uniformly in simple shear as shown in **Figure 17.5**. Drained tests were carried out on a variety of granular soils and clear patterns of behaviour emerged. **Figure 17.6** shows the results for some simple shear tests carried out on samples of sand at various initial relative densities ranging from very loose (best 1) to very dense (best 4). All the

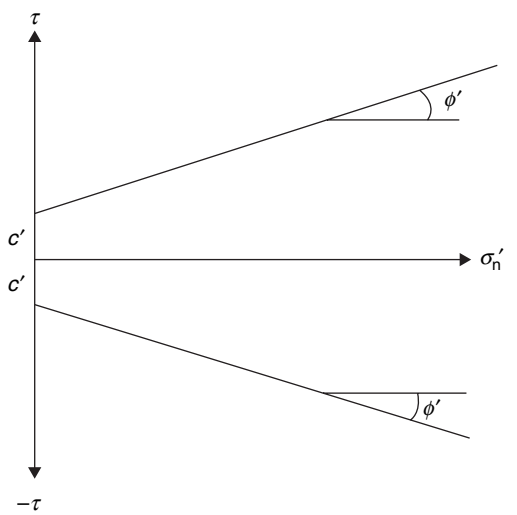


Figure 17.4 The Coulomb strength criterion

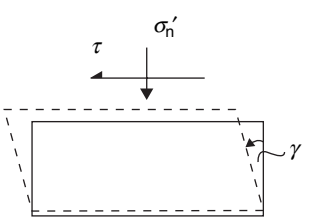


Figure 17.5 The mode of deformation of the Cambridge simple shear apparatus

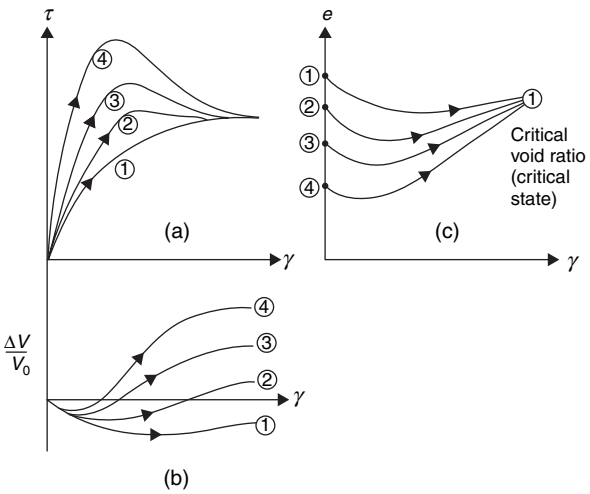


Figure 17.6 Results of drained tests on sand at various initial relative densities in the simple shear apparatus, carried out at the same value of σ'_n

tests were carried out at the same normal effective stress σ'_n. The results are plotted in **Figures 17.6(a–c)**.

Figure 17.6(a) shows graphs of shear stress τ against the shear strain γ. For the initially very loose sample it can be seen that the stress–strain curve rises at a decreasing rate until a maximum shear stress is reached, which then remains approximately constant as further deformation takes place. **Figure 17.6(b)** shows the corresponding volumetric strains during the test. It can be seen that as shearing commences the sample reduces in volume (contracts) initially, then increases very slightly and thereafter the volume remains constant for further deformation. Overall the sample can be seen to be contractant during shearing.

The above behaviour can be contrasted with that of an initially very dense sample. In **Figure 17.6(a)** it can be seen that the stress–strain curve rises steeply and reaches a maximum (peak) value, after which the strength reduces. Ultimately, with continued deformation, the shear stress reaches the same value of strength as the initially very loose sample. In **Figure 17.6(b)** it can be seen that after an initial small contraction, the very dense sample begins to dilate reaching a maximum rate of dilation at peak strength. Thereafter, the rate of dilation decreases until ultimately there is no further change in volume as shearing continues. Two samples of intermediate relative density show behaviour which lies between that of the initially very loose and very dense samples.

One of the key features of the Cambridge simple shear apparatus is that uniform deformations are imposed on the boundaries of the sample so that changes of void ratio can be measured throughout the test, right through to ultimate conditions at quite high shear strains. **Figure 17.6(c)** shows the changes of void ratio for each sample as the shear strains increase. The initially very loose sample starts at a high void ratio which reduces during most of the test. In contrast, the initially very dense sample starts at a low void ratio which then increases during the test. The important observation was made that all four samples ultimately ended up at the same void ratio, termed the 'critical void ratio'. It was further concluded that when a sample of soil reaches a condition in which it continues to shear at constant shear stress and constant void ratio, it has reached what is termed the 'critical state'.

All the tests described above were carried out at the same normal effective stress. **Figure 17.7** shows the results of tests at the four initial void ratios but with different values of vertical effective stress. The upper figure shows that the value of the critical void ratio reduces as the vertical effective stress increases, giving what has been termed the 'critical void ratio line'. In the lower figure, the peak strengths and ultimate strengths are plotted for each of the samples in a Mohr diagram. It can be seen that the ultimate strength line is straight and passes through the origin having a slope defined as φ'_{cs} – the critical state angle of shearing resistance. However the peak strength lines, particularly for the denser samples, are curved and lie above the critical state strength line. It can be shown that this upward displacement of the strength envelopes, together with their curvature, is due to the additional work done during shearing as a result of dilation.

The following broad conclusions can be drawn from these results:

(i) The effective confining pressure plays a dominant role in controlling the strength of a particulate material.

(ii) The initial density influences the peak strength. The greater the initial density, the steeper and more curved is the peak strength envelope in a Mohr diagram of τ versus σ'_n.

(iii) After a large shear strain, the shearing resistance of loose and dense samples under the same vertical effective stress converge to the same value. Similarly, the void ratios of each sample converge to the same value. This ultimate state is known as the 'critical state' in which a sample at a given value of vertical effective stress will continue to shear at constant shear stress and constant void ratio (i.e. at the critical state, the rate of dilation is zero).

(iv) On a Mohr diagram, the critical state strength line is straight and passes through the origin with an angle of shearing resistance φ'_{cs}.

(v) The initial density and the magnitude of σ'_n influence the amount of dilation during shear. The greater the initial density and the lower the magnitude of σ'_n, the greater is the dilation. It is noted that the maximum rate of dilation corresponds approximately to peak strength. The curvature of the strength envelope is related to the work done during dilation against the confining pressure.

(vi) At a given normal effective stress σ'_n, the peak strength τ_p is greater than the critical state strength τ_{cs}. The strength difference $(\tau_p - \tau_{cs})$ is due to the rate of dilation and is sometimes termed the 'enhanced strength'.

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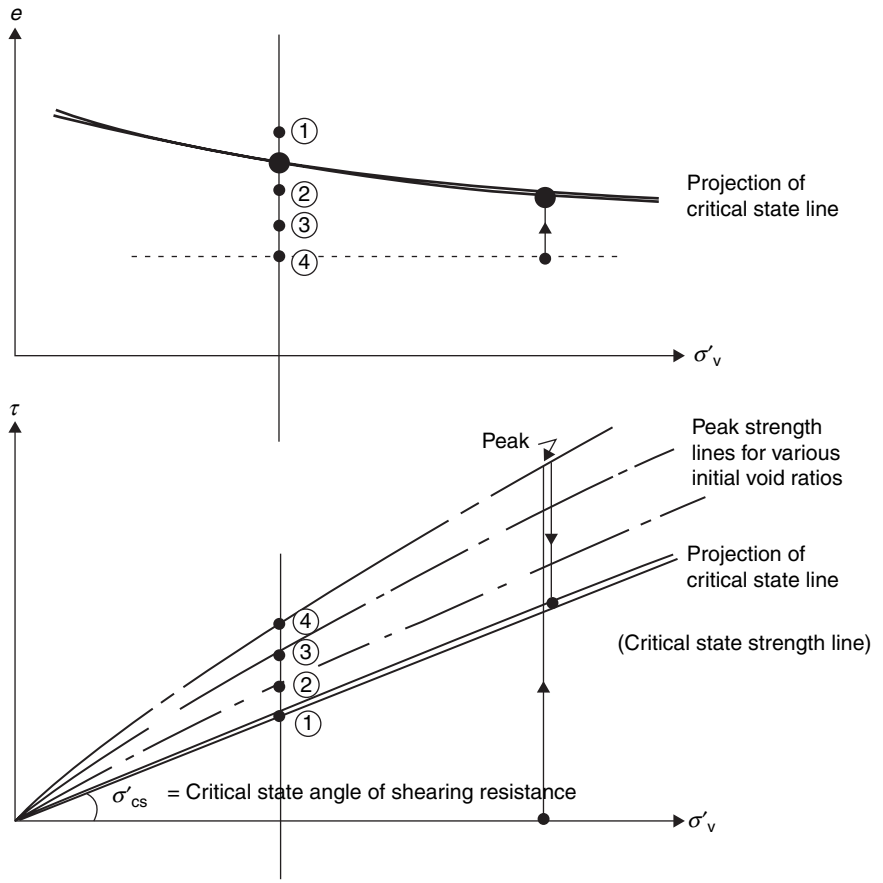


Figure 17.7 Results of drained tests on sand at various initial densities in the simple shear apparatus, carried out at various values of σ'_n

17.3.3 Results of drained shear box tests on clay soils

The basic strength properties of clay soils have been researched using samples that have first been formed with high moisture content and then consolidated to various values of effective normal stress in a shear box giving what is called 'the virgin compression line'. We describe such samples as remoulded or 'reconstituted'. **Figure 17.8(a)** shows the virgin compression line of void ratio e against σ'_v for a clay soil that has been formed in this way. At given values of effective normal stress σ'_v (points A and B), drained shear box tests are carried out. The behaviour is very similar to that of loose sand. **Figure 17.8(c)** shows the stress–strain curves of τ versus γ which rise at a decreasing rate until a maximum shear stress is reached, which then remains approximately constant as further deformation takes place (points A′ and B′). It can be seen from **Figure 17.8(c)** that as shearing commences, the samples contract at a rate that progressively decreases until at maximum strength there is no further volume reduction and void ratios remain constant (points A′ and B′ in **Figure 17.8(a)**). Thus the samples appear to reach a critical state condition. When plotted on a Mohr diagram of τ versus σ'_v, the strengths lie on a straight line through the origin, as shown in **Figure 17.8(b)**, having an angle of shearing resistance φ'_{cs} – the critical state strength line.

Overconsolidated reconstituted clay soils behave in a rather similar manner to dense sands. **Figure 17.9(a)** shows the relationship between void ratio e and σ'_v for a sample of reconstituted clay that has been consolidated to a high value of σ'_v (point B), after which the effective stress has been progressively decreased – causing the sample to swell and come to equilibrium at a much lower value of σ'_v (point C). If a drained shear box test is now carried out with σ'_v held constant, the stress–strain curve (see **Figure 17.9(c)**) will reach a peak value of τ at C′ which then gradually reduces as shearing continues. At the same time, the sample will tend to dilate so that the void ratio increases. When plotted on a Mohr diagram, the peak strength C′ lies above the corresponding critical state value and the strength envelope for the soil is curved as shown in **Figure 17.9(b)**.

17.3.4 Residual strength of clay soils

As described in section 17.3.2, for dense granular soils, tests show that after the strength reaches a peak value it gradually reduces to a steady state value corresponding to a critical state

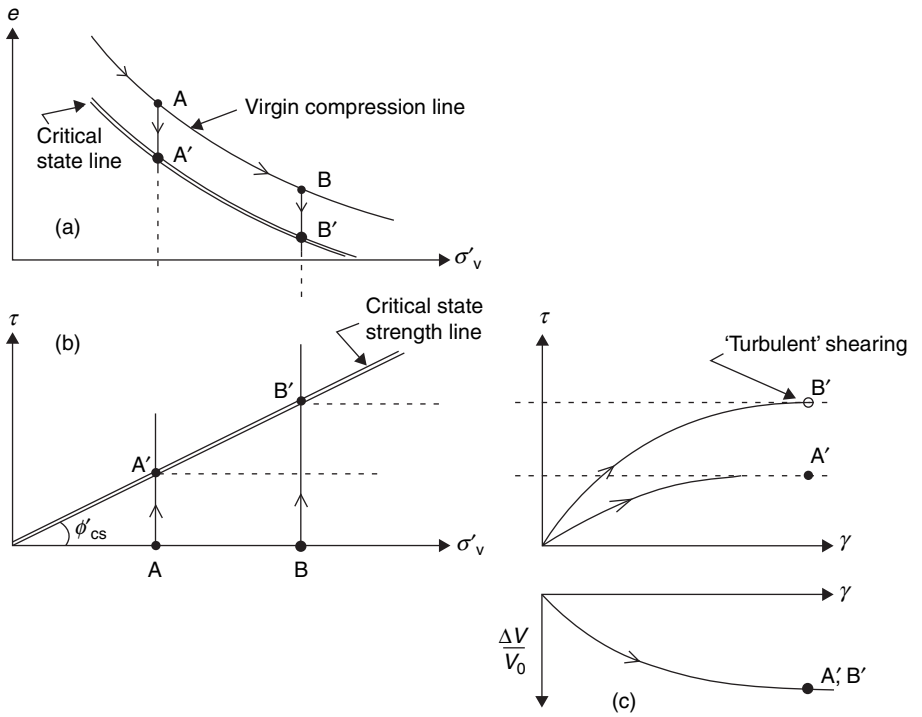


Figure 17.8 Results of drained simple shear tests on a reconstituted normally consolidated clay

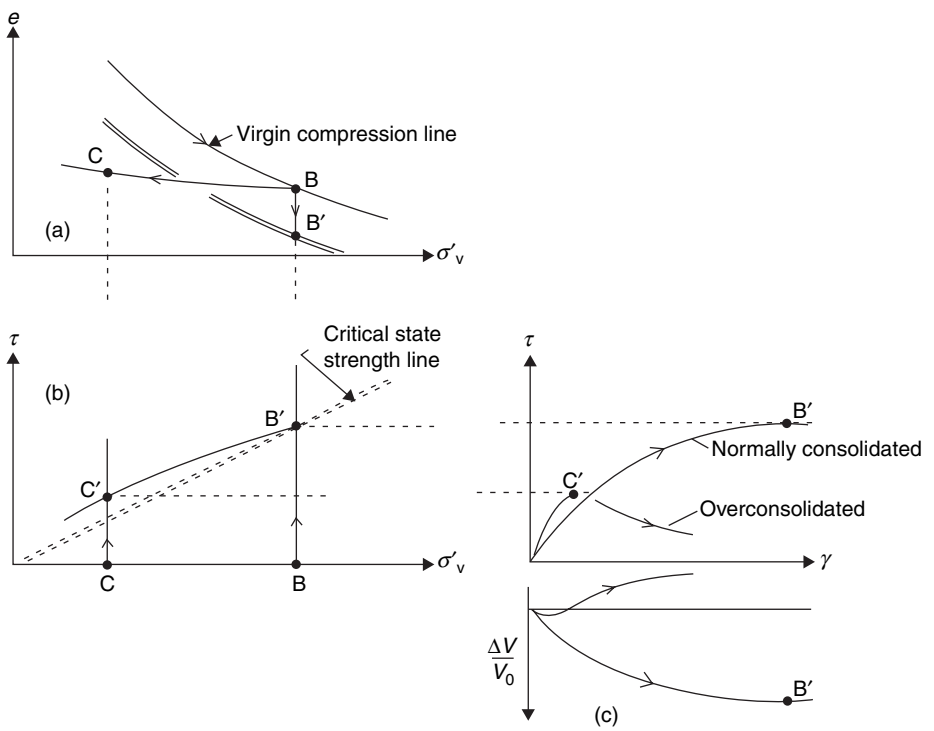


Figure 17.9 Results of drained simple shear tests on a reconstituted overconsolidated clay

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condition. It is often assumed that overconsolidated clay soils behave in a similar way. However, the behaviour of clay soils is complicated by the fact that the shearing process can dramatically change the fabric (arrangement of the particles). Ring shear box tests on soils with a high clay content show that after peak strength is reached, the clay particles progressively align themselves in the zone of shearing until a polished shear surface is formed. In so doing, the strength steadily decreases with increasing displacement across the shear plane until the clay particles are fully aligned and a low 'residual strength' is reached – see **Figure 17.10**. Lupini *et al*. (1981) termed this 'sliding shearing'. They contrasted this with shearing at the critical state which they termed 'turbulent shearing' to convey the idea that during continued shearing at the critical state, the particles are tumbling over one another in a turbulent manner.

Figure 17.11 shows the comparison between the critical state angle of shearing resistance φ'_{cs} and the residual friction angle φ'_r for a sand–bentonite mixture in which the bentonite content was steadily increased. It can be seen that for a clay fraction of less than 20% there is little difference between φ'_{cs} and φ'_r. However, as the clay fraction increases the values of φ'_r become progressively less than φ'_{cs}. For an overconsolidated clay tested in a ring shear apparatus, the strength drops smoothly from peak to residual as shown in **Figure 17.10** and it is not possible to distinguish an obvious critical state strength. It seems that the simplest way of measuring the critical state strength is on normally consolidated reconstituted samples of the soil which contract during shearing.

17.3.5 The influence of bonding on the drained strength of clay soils

In the previous section, the influence of changes in microfabric on the drained shearing resistance of clay soils was described. Another important influence is the effect of inter-particle bonding which most natural clays exhibit to a greater or lesser extent. Bonding has the effect of increasing the strength and stiffness of the material. Tests on initially intact, natural, stiff, overconsolidated clays show that the breaking of the bonds results in a very rapid post-peak reduction of strength with the formation of well defined rupture surfaces – see **Figure 17.12**. The strength along the rupture surface, known as the post-rupture strength, has an effective angle of shearing resistance φ'_{pr}. This is remarkably similar to the critical state value for the reconstituted material, but often with a small effective cohesion c'. In view of the very brittle post-peak behaviour of such soils it would be unwise for the design to rely on strengths exceeding the post-rupture values.

17.4 The undrained strength of clay soils

When a shear stress is applied to a clayey soil of relatively low permeability, it takes a considerable time for the pore water to drain into or out of the pores. When shearing takes place with little or no drainage, the process is defined as 'undrained shearing', and the measured strength is defined as the 'undrained strength' S_u. The tendency for a soil to contract or dilate during drained shearing has a profound effect on the undrained strength of the material. This behaviour can be illustrated by considering elements of soil sheared undrained in a shear box, in which the applied normal stress σ_n is kept constant during the test.

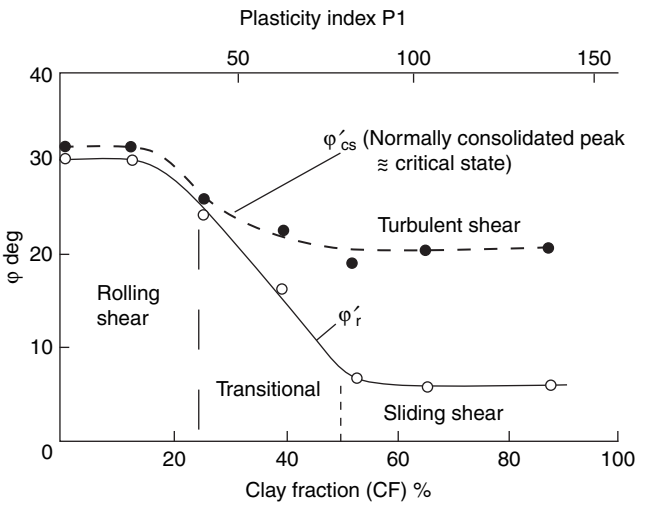


Figure 17.11 Ring shear tests on sand–bentonite mixtures. Normally consolidated at $\sigma' = 350$ kPa; PI/CF = 1.55
Data taken from Lupini *et al*. (1981)

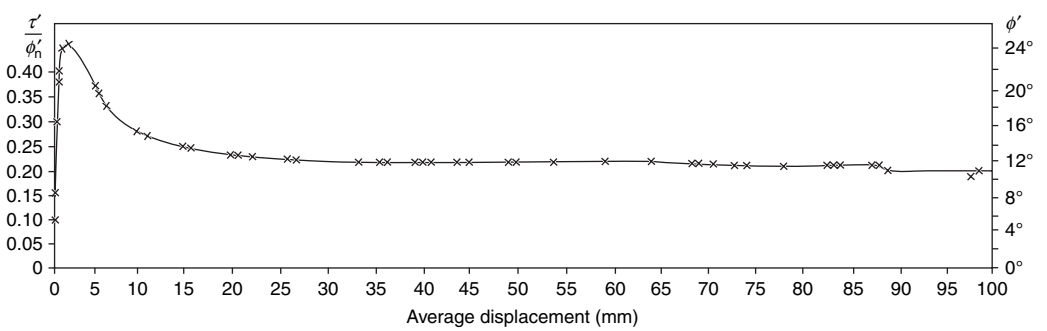


Figure 17.10 Results of drained ring-shear tests on an undisturbed sample of undisturbed Blue London Clay. Nominal normal stress $\sigma'_n = 207$ kPa; nominal shearing rate 0.0076 mm/min
Redrawn with data taken from Bishop *et al*. (1971)

For a loose (normally or lightly overconsolidated) sample, as τ is increased, the element will try to contract – but is prevented from doing so. As a consequence, stress is thrown onto the pore water so that the pore pressure u increases and, since σ_v is constant, σ'_v decreases. The resulting effective stress path and stress–strain behaviour are shown by the full lines in **Figure 17.13**. It can be seen that the undrained strength S_u (given by point A) is less than the corresponding drained strength (given by point B). If, at a given value of τ, drainage now takes place with τ and σ_v kept constant, the positive

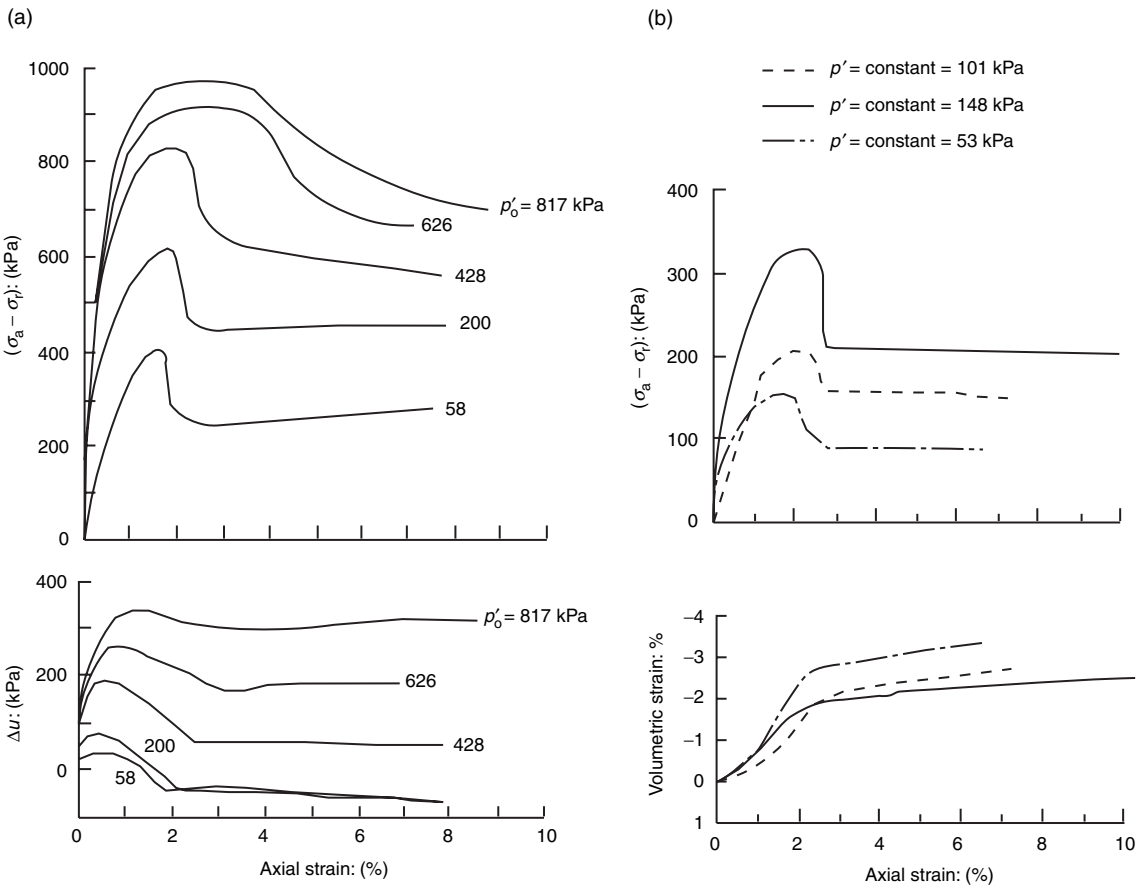


Figure 17.12 Results of (a) drained and (b) undrained triaxial tests on intact Vallerica Clay showing very brittle post-peak strength reductions
Reproduced from Burland *et al.* (1996)

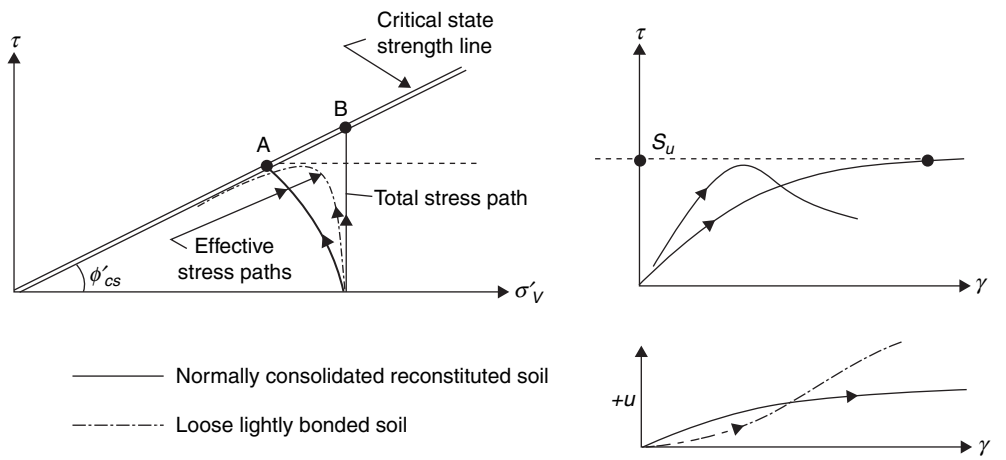


Figure 17.13 Undrained shearing of a normally consolidated clay

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pore pressure dissipates, the volume decreases and the strength of the sample will increase. *Hence contractant soils tend to get stronger as they drain after the undrained application of a shear stress*.

For an initially dense (overconsolidated) sample, as τ is increased the element will try to dilate but is prevented from doing so. In contrast to the loose sample, the pore pressure u decreases as the soil tries to dilate and hence σ'_v increases. The resulting stress path and stress–strain behaviour are shown by the full lines in **Figure 17.14**. It can be seen that the undrained strength S_u (point A) is greater than the drained strength (point B). If, at a given value of τ, drainage now takes place with τ and σ_n kept constant, the negative pore pressure dissipates, the volume increases and the strength of the sample will decrease. Clearly if the drained strength is less than the applied shear stress τ, failure will occur as drainage is taking place. This is the cause of long-term (or delayed) slope failures in overconsolidated cut slopes – see Chapter 23 *Slope stability*. Hence dilatant soils tend to get weaker as they drain after the undrained application of a shear stress.

The chain-dotted lines in **Figure 17.13** represent the undrained behaviour of an initially very loose soil with some bonding between the particles. The material is highly contractive and positive pore pressures continue to develop after peak undrained strength is reached, and the bonds are progressively broken down. As a consequence, the strength reduces and the effective stress path continues to travel to the left moving down the critical state strength line. Hence loose soils, such as normally and lightly overconsolidated natural clays, can exhibit very brittle undrained shearing behaviour with significant loss of strength after the peak has been reached, particularly if they are lightly bonded or cemented.

The degradation of undrained strength upon remoulding is termed 'sensitivity' which is defined as the undrained strength of a clay divided by its remoulded strength at the same moisture content.

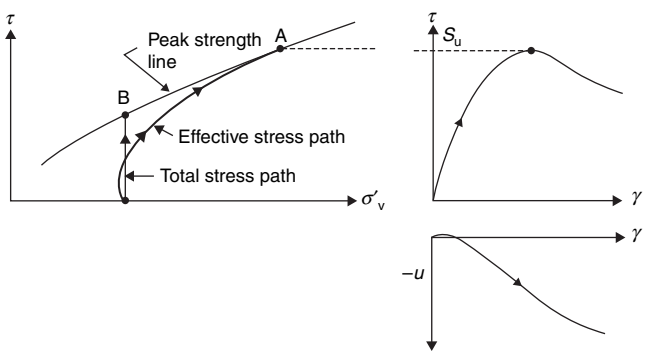


Figure 17.14 Undrained shearing of an overconsolidated clay

17.5 The Mohr–Coulomb strength criterion

So far we have only considered the strength of a mass of soil in terms of the classical Coulomb equation (17.1) applied to a shear box in which the normal effective stress σ'_n and the shear stress τ are known. This equation gives no information about the principal stresses or their orientation. Mohr showed that when a material fails according to the Coulomb criterion, the Mohr's circle of stress corresponding to the state of failure is tangential to the Coulomb strength line as shown in **Figure 17.15**.

Figure 17.16 shows the Mohr's circle of failure for a drained triaxial compression test for which the axial effective stress σ'_a is the major principal effective stress ($= \sigma'_I$) and the radial effective stress σ'_r is the minor principal effective stress ($= \sigma'_{III}$). From simple geometry, it can be shown that σ'_{III} is related to σ'_I by the expression:

$$\sigma'_{III} = \sigma'_I \tan^2(\pi/4 - \varphi'/2) - 2c' \tan(\pi/4 - \varphi'/2) \qquad (17.2)$$

which is known as the Mohr–Coulomb strength criterion.

In **Figure 17.16**, since σ'_a ($= \sigma'_I$) acts normal to a horizontal plane, the pole coincides with the σ'_{III} point as shown. The lines connecting the pole to the points of tangency of the Mohr's circle of failure are often termed 'stress characteristics'

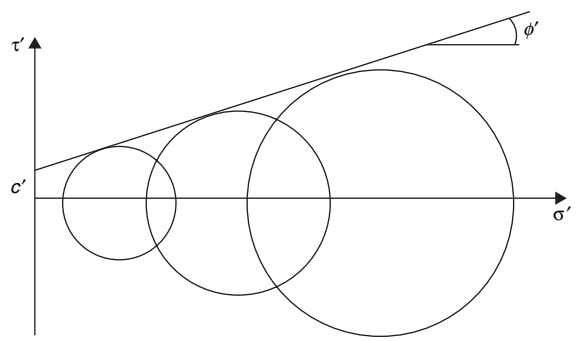


Figure 17.15 The Mohr–Coulomb strength criterion

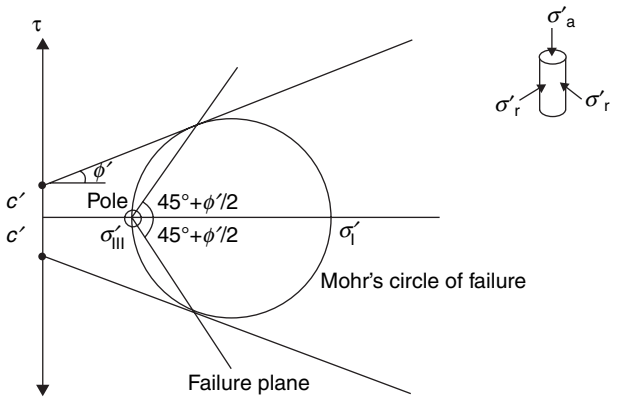


Figure 17.16 Failure in a triaxial compression test

or 'failure planes'. It can be seen that the failure planes make an angle of $\sigma'_n \pm (\pi/4 + \varphi'/2)$ to the major principal plane.

We saw that the measured strength envelope from a series of shear box tests is often curved. In this situation the Mohr's circles are still tangential to the envelope. By carrying out a number of drained or undrained triaxial tests with pore pressure measurements it is possible to construct the strength envelope by constructing the Mohr's circles of effective stress corresponding to failure. The best fit failure envelope is then sketched in tangential to these circles as shown in **Figure 17.17**. In this particular case, an effective cohesion intercept c' is indicated which will only exist if there is some inter-particle bonding at very low confining pressures.

17.6 Choice of strength parameters for analysis and design

In the previous sections we identified at least three common definitions of strength: peak strength, critical state strength and residual strength. Peak and critical state strength may be applied to either drained or undrained conditions. Residual strength is usually a drained condition.

We saw that the Coulomb equation is a simplification. Firstly, the peak value of φ' is not unique to a given soil, but depends on its initial void ratio or density. Secondly, the strength envelope is often curved and therefore does not strictly satisfy the Coulomb equation. As a consequence of these observations, which effective strength parameters to use in an analysis that makes use of the Mohr–Coulomb strength criterion is not straightforward. A simple practical approach is to choose a linear failure envelope defined by values of φ' and c' that are appropriate to the range of normal effective stresses encountered in the problem. In these circumstances it is important to appreciate that the chosen design value of c' is an analytical convenience rather than a true measure of the cohesion of the material.

The choice of a design value of undrained strength S_u is also not straightforward. The key point to remember about undrained strength is that it is not a unique quantity for a given deposit of soil or soil sample. Its value depends on at least three factors: (1) the rate at which the sample is tested (i.e. S_u is rate dependent – the higher the rate the greater the strength); (2) the undrained strength of most soils is anisotropic and depends on the orientation of the principal stresses; and (3) for stiff fissured clays, the undrained strength depends on the size of the sample being tested – the mean strength of 100 mm diameter samples of stiff fissured clays will usually be significantly lower than that of 35 mm diameter samples. The important subject of choice of strength parameters is covered in Chapter 27 *Geotechnical parameters and safety factors*.

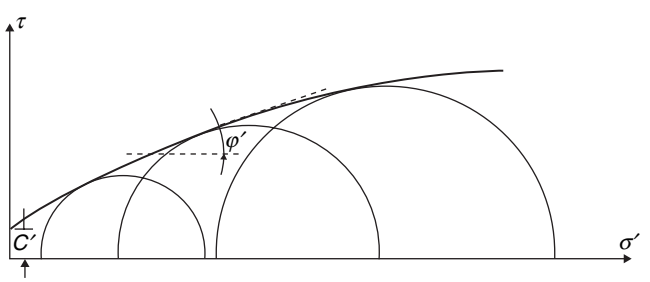


Figure 17.17 Curved strength envelope drawn tangential to Mohr's circles of failure

17.7 The compressibility of soils

We now move from discussing the strength of soils to their deformability. Because of their particulate nature, soils are compressible – especially clay soils. As discussed in Chapter 14 *Soils as particulate materials*, increases in all-round effective stress cause slip to take place between particles which then take on a closer pack and, as a consequence, water is squeezed out of the voids. The time taken for the water to be expelled is a function of the permeability and compressibility of the soil.

Perhaps the earliest, and still most widely used, apparatus for measuring the compressibility of soils is the oedometer, in which a sample of soil is compressed one-dimensionally as shown in **Figure 17.18**. Detailed descriptions of the test and its application are given in most textbooks – e.g. Craig (2004), Powrie (2004).

17.7.1 The coefficient of volume compressibility (mv)

We now consider the one-dimensional (confined) compression characteristics of an initially very loose sample of soil that is placed in an oedometer, subjected to a number of increments of increasing vertical effective stress with full drainage between increments, and then subsequently unloaded. **Figure 17.19(a)** shows a graph of void ratio e versus vertical effective stress σ'_v for the sample which is termed the 'virgin compression line'. It can be seen that this compression line ABC is concave upwards, showing that as σ'_v increases the soil becomes increasingly less compressible. The coefficient of volume compressibility m_v is defined as the increment in volumetric strain $\Delta V/V_0$ divided by the corresponding increment in vertical effective stress $\Delta\sigma'_v$. We consider a state B on the compression line for the sample corresponding to a void ratio e_0 and a vertical effective stress σ'_{v0}. An increment $\Delta\sigma'_v$ causes a decrease in void ratio Δe. Since the initial volume of the sample per unit volume of solid is $(1 + e_0)$ and the corresponding change of volume is equal to Δe, it follows that

$$m_v = \frac{\Delta e/(1+e_0)}{\Delta\sigma'_v}. \qquad (17.3)$$

Clearly m_v is not a constant, but it has to be determined for the appropriate stress range. It is usually expressed in the units m^2/MN – the inverse of stress.

17.7.2 Simple mechanistic explanation of compression and swelling

The soil sample is loaded to a vertical effective stress σ'_{vp} (point C in **Figure 17.19(a)**) and then unloaded in increments. It can be seen that the sample increases in volume as unloading

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takes place. The unloading (or swelling) curve is much flatter than the initial compression line. A simple mechanistic explanation of this behaviour is that during initial loading, much of the volume reduction results from slip at the particle contact points as the particles take on a closer pack. On unloading, little 'reverse' slip occurs and much of the recovery results from elastic unloading of the particles and, in the case of clays, absorption of water into the clay particle lattice structure.

The sample is now re-loaded to an effective stress corresponding to point D which is greater than σ'_{vp}. Initially, the re-loading compression line lies a little above the unloading curve – until the stress approaches the maximum previous stress σ'_{vp} where it crosses the unloading curve and re-joins the virgin compression line. The simple mechanistic explanation of this behaviour is that initially, on re-loading, the material is responding approximately reversibly and the particles deform with little slip at the grain contact points. However, once the effective stress approaches the previous maximum value, slip at the contact point begins to re-commence and the re-loading compression line re-joins the original 'virgin compression line' as shown. Thus the maximum previous effective stress acts as a kind of 'pre-loading' such that, at lower stresses, the compressibility of the sample is much less than on first loading. Once the 'pre-loading' is exceeded, significant slip at the particle contact points re-commences. This concept of the onset of non-reversible particle contact slip is very important and is analogous to 'yield' of a ductile metal which signals the onset of irrecoverable (plastic) strains.

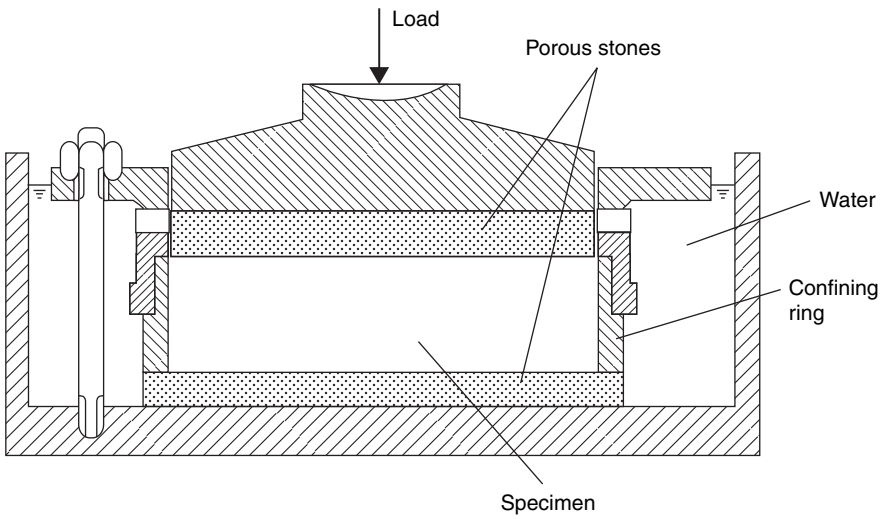


Figure 17.18 The oedometer

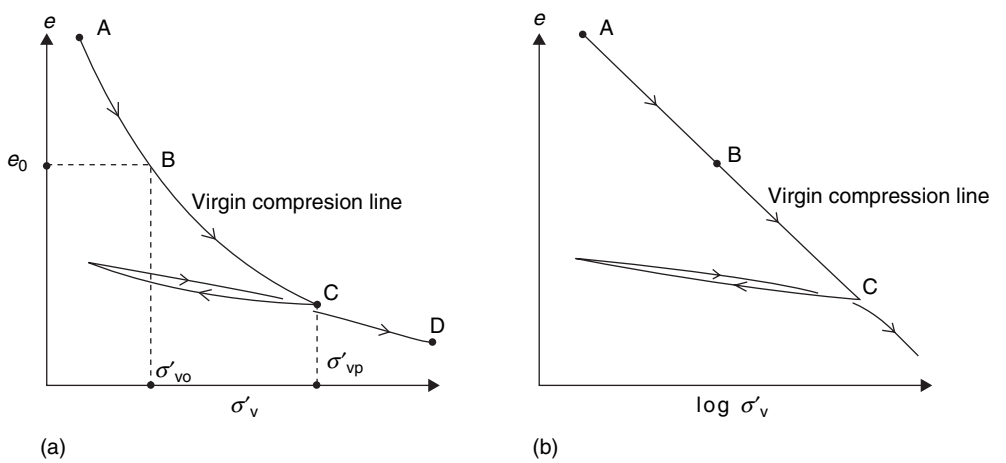


Figure 17.19 One-dimensional (confined) compression characteristics of an initially very loose soil subject to loading, unloading and re-loading; (a) σ'_v plotted to a natural scale; (b) σ'_v plotted to a logarithmic scale

17.7.3 The compression curve plotted on semi-logarithmic axes

It is common practice to plot the compression and swelling lines illustrated in **Figure 17.19(a)** using a log scale for the stress axis, as shown in **Figure 17.19(b)**. When plotted in this manner, the virgin compression and swelling lines approximate to straight lines. The slope of the virgin compression line in a semi-logarithmic plot is defined as the compression index, C_c, and for any two points on the approximately linear portion, it is given by:

$$C_c = \frac{e_0 - e_1}{\log(\sigma'_{v1} / \sigma'_{v2})}. \quad (17.4)$$

The slope of the unloading part of the $e - \log \sigma'_v$ plot is referred to as the swelling index C_s.

For the reloading line, the sharp downward curvature as it rejoins the virgin compression line has traditionally been used to determine the maximum past pressure σ'_{vp}. As will be explained in the following sections, some care is needed in interpreting oedometer curves in this way – as the sharp downward curvature also indicates the breaking of inter-particle bonds.

17.7.4 The influence of inter-particle bonding on compression and yield

Most natural soils have some inter-particle bonding (due to cementation or physico-chemical effects) which has an important influence on the compression characteristics. Physico-chemical bonds can develop over quite a short period of time in clay soils. Their influence was demonstrated by Leonards and Ramiah (1959) in some classic experiments, as reported by Burland (1990). A clay soil was reconstituted and compressed in an oedometer for which special precautions were taken to minimise wall friction. At a given value of vertical effective stress, the load was held constant for 12 weeks. After this ageing period, the loading process was then continued.

The results are illustrated schematically in **Figure 17.20** on a graph of e versus log σ'_v. As σ'_v was incremented daily, the soil compressed along the virgin compression line. At a stress σ'_{v0}, the load was maintained constant for 12 weeks, during which time some creep compression took place and the void ratio of the soil dropped below the virgin compression line. When loading was resumed, the resulting compression curve crossed to the right of the virgin compression line, then bent downwards sharply and rejoined the virgin compression line from above. Some additional tests were carried out in which creep was prevented during the ageing process. Once again, the subsequent compression line moved out to the right of the virgin compression line before bending down sharply and rejoining it as before. The downward 'kink' in the compression curve results from the breaking of the particle bonds and the onset of inter-particle slip. In the case of Leonards and Ramiah's experiments this 'kink' occurred at a stress equal to about 1.5 times the value of σ'_{v0}.

The important conclusion from the above work is that care is needed in adopting the classical interpretation of the point of maximum curvature of an oedometer compression curve as indicating the maximum past effective overburden pressure. What it actually indicates is the stress at which significant particle slip begins to take place, often due to the breaking of inter-particle bonds. For this reason, Burland (1990) recommended that this point be referred to as the vertical yield stress σ'_{vy} rather than the pre-consolidation pressure. The latter term should be reserved for situations in which the magnitude of such a pre-consolidation pressure can be established by geological means. It is the norm rather than the exception that normally consolidated natural clays have values of $\sigma'_{vy}/\sigma'_{v0}$ (termed the yield stress ratio) as high as 1.5. In cases where cementation has taken place, the yield stress ratio can be very much higher.

The accurate determination of σ'_{vy} is of great practical importance – if vertical effective stresses can be kept below this value, then settlements will generally be small. If, for soft clays, the vertical yield stress is exceeded, very large movements can take place. The vertical yield stress is also useful as an empirical indicator of undrained compressive strength S_{uc}. It has been found that the ratio S_{uc} / σ'_{vy} is roughly in the range 0.28–0.32 for a wide variety of natural clays (Burland, 1990).

Sometimes the use of an e versus log σ'_v plot can be misleading when attempting to evaluate σ'_{vy}. For example, a linear line on an e versus σ'_v plot becomes a convex upwards curve on an e versus log σ'_v plot. It is not unknown for such a line to be interpreted as exhibiting a pre-consolidation pressure or yield stress! A more reliable approach is to plot the data on double log axes as suggested by Butterfield (1979). **Figure 17.21** is taken from Butterfield (1979) who plotted $\ln(1 + e)$ versus log σ'_v for four sets of data given by Taylor (1948). It is evident that the double logarithmic plots give a clearer indication of the vertical yield stress than do the semi-logarithmic plots.

17.8 The stress–strain behaviour of soils

Section 17.3 began with a description of the most commonly used idealisation of the strength of soils *viz.* the Coulomb equation. Thereafter, the results of soil tests were examined to assess the equation's accuracy and limitations. A similar

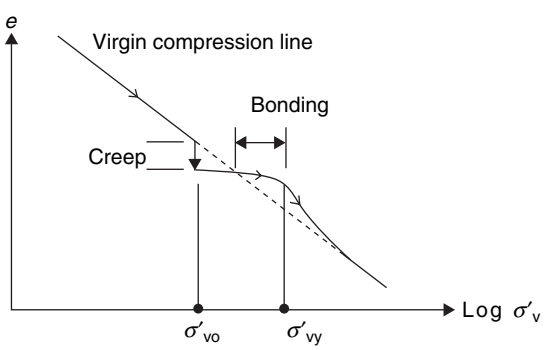


Figure 17.20 Influence of ageing and inter-particle bonding on the compression characteristics of a reconstituted normally consolidated clay

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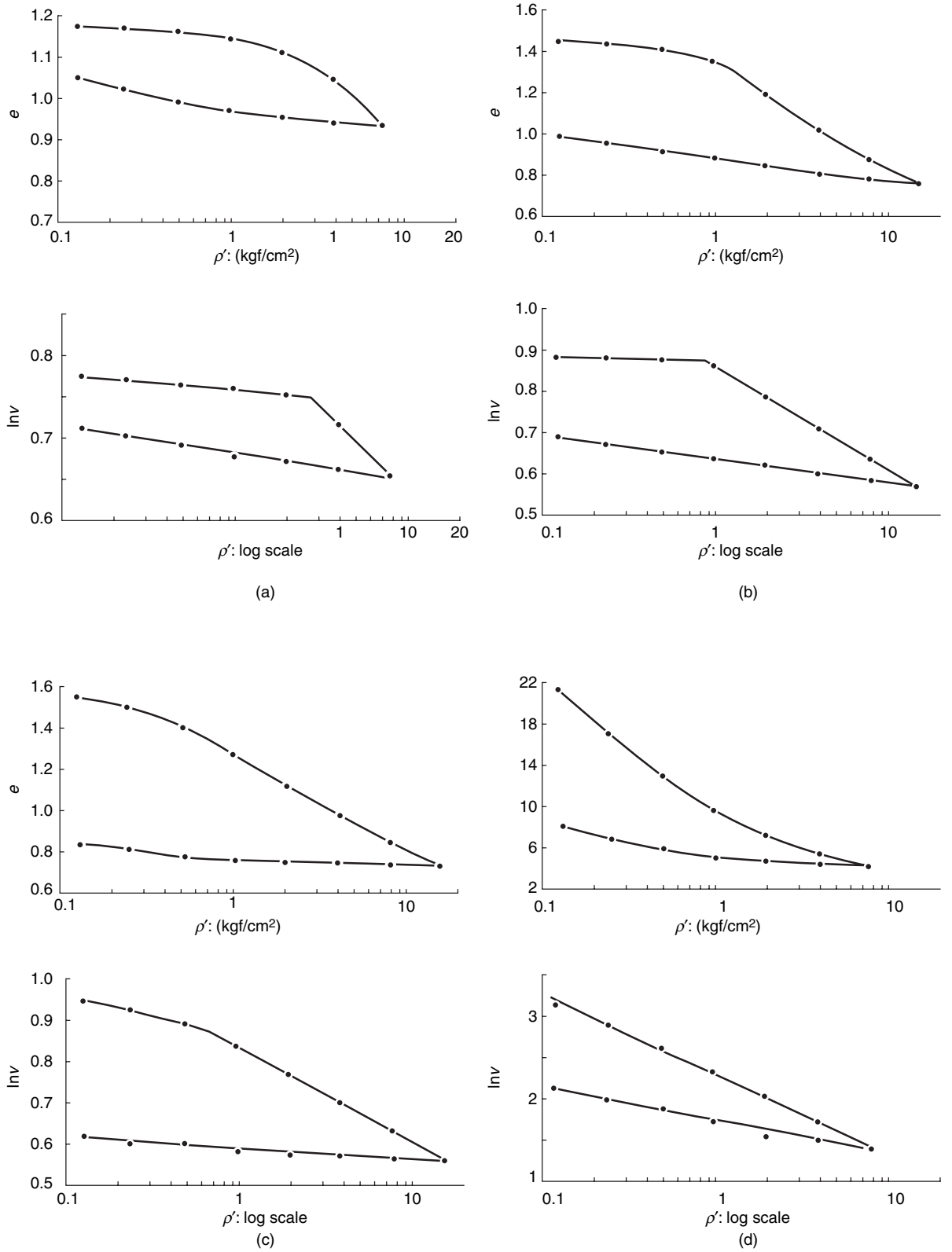


Figure 17.21 The use of double logarithmic plots to identify σ'_{vy} for various soils (a) Boston Blue Clay; (b) Chicago Clay; (c) Newfoundland silt; (d) Newfoundland peat

Reproduced from Butterfield (1979)

approach is adopted for the study of the stress–strain behaviour of soils. The most commonly used model of the stress–strain behaviour of soil is that of an ideal porous elastic solid.

17.8.1 Ideal isotropic porous elastic solid

A basic property of an elastic material is that when it is subjected to a closed cycle of stress, the deformations are reversible and fully recoverable. It is not necessary for the stress–strain behaviour to be linear. It is helpful to have a physical image of an isotropic porous elastic material: imagine a material which consists of very fine metal filings (elongated particles) arranged in a random manner and spot welded at their contact points (to ensure that the assemblage is elastic). The particles are so fine and the pores so small that the material has a low permeability. Hence, when loaded quickly, pore pressure changes can take place within the voids.

An isotropic elastic material is fully defined by two parameters. The most commonly used ones are Young's modulus E and Poisson's ratio υ. We are interested in the properties of the *skeleton* of this ideal soil. These are the properties that control its drained behaviour; we call them the 'effective' properties and denote them by a prime. The two most commonly used effective parameters are:

The effective Young's modulus E'

The effective Poisson's ratio υ'

The corresponding undrained parameters are denoted as E_u and υ_u.

17.8.2 The elastic equations

As is usually the case in soil mechanics, compressive stresses are taken as positive – so reductions in length or volume give rise to positive strains. In developing the elastic equations we deal with *stress changes* and their corresponding *strain changes*. Consider a vertical rectangular prism of isotropic porous elastic material as shown in **Figure 17.22**. An increment of principal vertical effective stress $\Delta\sigma'_1$ is applied allowing full drainage. The resulting increment in vertical strain $\Delta\varepsilon_1$ is $\Delta\sigma'_1/E'$. It also follows that the increments in horizontal strain are given by $\Delta\varepsilon_2 = \Delta\varepsilon_3 = -\upsilon'\Delta\sigma'_1/E'$.

Independent applications of $\Delta\sigma'_2$ and $\Delta\sigma'_3$ in the horizontal plane will cause increments of strain in the vertical direction as follows:

$$\Delta\varepsilon_1 = -\upsilon'\Delta\sigma'_2/E' \tag{17.5}$$

and

$$\Delta\varepsilon_1 = -\upsilon'\Delta\sigma'_3/E'. \tag{17.6}$$

By superposition, we find that for the general case of changes in all three principal effective stresses, the increment of vertical strain is given by

$$\Delta\varepsilon_1 = \{\Delta\sigma'_1 - \upsilon'\Delta\sigma'_2 - \upsilon'\Delta\sigma'_3\}/E'. \tag{17.7}$$

Similarly:

$$\Delta\varepsilon_2 = \{\Delta\sigma'_2 - \upsilon'\Delta\sigma'_1 - \upsilon'\Delta\sigma'_3\}/E' \tag{17.8}$$

and

$$\Delta\varepsilon_3 = \{\Delta\sigma'_3 - \upsilon'\Delta\sigma'_1 - \upsilon'\Delta\sigma'_2\}/E'. \tag{17.9}$$

These three simple equations can be used to derive a number of important properties for an ideal isotropic porous elastic material.

17.8.3 Volumetric strain

An increment in volumetric strain Δv $(= \Delta V/V_0)$ is equal to the sum of the three principal increments of strain (neglecting second-order terms). Adding equations (17.7)–(17. 9) gives

$$\Delta v = \frac{1-2\upsilon'}{E'}\{\Delta\sigma'_1 + \Delta\sigma'_2 + \Delta\sigma'_3\}. \tag{17.10}$$

Therefore,

$$\Delta v = \frac{3(1-2\upsilon')}{E'}\Delta p' = \frac{\Delta p'}{K'} \tag{17.11}$$

where $\Delta p' = (\Delta\sigma'_1 + \Delta\sigma'_2 + \Delta\sigma'_3)/3$, the effective mean normal stress and $K' = E'/3(1 - 2\upsilon')$, the effective bulk modulus.

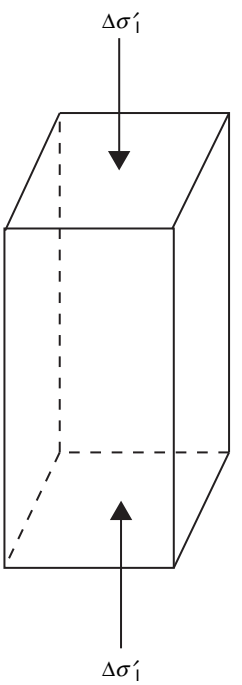


Figure 17.22 Element of porous isotropic elastic material subjected to an increment in vertical effective stress

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It follows from equation (17.11) that if there is no volume change during the application of a general stress increment, then there will be no change in mean normal effective stress, i.e. if $\Delta v = 0$, then $\Delta p' = 0$, and vice versa. This result is an inevitable consequence of the idealisation of isotropic porous elastic behaviour. It has important consequences for undrained behaviour, as discussed later.

17.8.4 Shear strain

By definition, an increment in maximum shear strain $\Delta\gamma_{max} = \Delta\varepsilon_1 - \Delta\varepsilon_3$. Substituting from equations (17.7) and (17.9), and noting that $\Delta\tau_{max} = (\Delta\sigma'_1 - \Delta\sigma'_3)/2$, it follows that

$$\Delta\gamma_{max} = \frac{2(1+\upsilon')}{E'}\Delta\tau_{max} = \frac{\Delta\tau_{max}}{G'} \qquad (17.12)$$

where G' is the effective shear modulus $= E'/2(1+\upsilon')$. It follows from equation (17.12) that the application of an increment, in shear stress only, causes no volume change. Therefore this ideal material is non-dilatant. Moreover, we see that the effects of applying $\Delta p'$ and $\Delta\tau_{max}$ are entirely separate, in that the former only causes volume change and the latter only causes shear distortion. This is termed 'uncoupled' behaviour.

17.8.5 One-dimensional compression

We saw from section 17.7 that the coefficient of one-dimensional volume compressibility $m_v = \Delta v/\Delta\sigma'_v$ where $\Delta\varepsilon_2 = \Delta\varepsilon_3 = 0$. It can be shown from the elastic equations that

$$m_v = \frac{1 - \dfrac{2\upsilon'^2}{1-\upsilon'}}{E'}. \qquad (17.13)$$

It should also be noted that if $\Delta\varepsilon_3 = 0$ and $\Delta\sigma'_2 = \Delta\sigma'_3$ in equation (17.9),

$$\frac{\Delta\sigma'_3}{\Delta\sigma'_1} = \frac{\upsilon'}{1-\upsilon'} \qquad (17.14)$$

which gives the value of the coefficient of earth pressure at rest K_0 for the one-dimensional compression of an isotropic porous elastic material (See Chapter 15 *Groundwater profiles and effective stresses* for a discussion on the important topic of the coefficient of earth pressure at rest, K_0). The following limiting values of m_v from equation (17.13) should be noted:

- when $\upsilon' = 0$; $m_v = 1/E'$, i.e. m_v is the reciprocal of E';
- when $\upsilon' = 0.5$; $m_v = 0$, i.e. the material is incompressible.

17.8.6 The measurement of E′ and υ′ in an ideal drained triaxial test

For a standard drained triaxial test, increments of axial effective stress are applied without change in effective radial stress, i.e. $\Delta\sigma'_a = \Delta\sigma'_1 > 0$ and $\Delta\sigma'_r = \Delta\sigma'_3 = 0$. Both the axial strain $\Delta\varepsilon_a = \Delta\varepsilon_1$ and the volumetric strain $\Delta V/V_0$ are measured and plotted as shown in **Figure 17.23**.

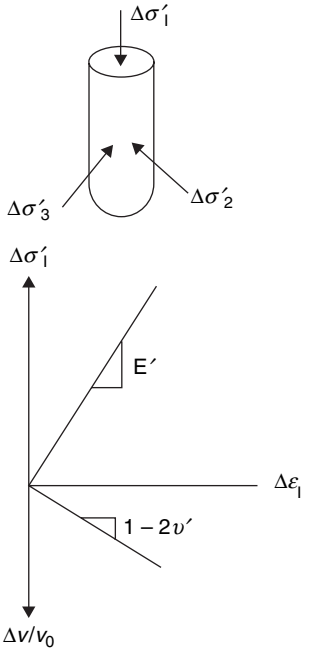


Figure 17.23 Determination of E' and υ' from a drained triaxial test on an ideal isotropic porous elastic material

From elastic equation (17.7):

$$E' = \Delta\sigma'_1/\Delta\varepsilon_1. \qquad (17.15)$$

By definition $\upsilon' = -\Delta\varepsilon_3/\Delta\varepsilon_1$ and $\Delta V/V_0 = \Delta\varepsilon_1 + 2\Delta\varepsilon_3$. Therefore

$$\Delta\varepsilon_3 = \frac{\left[\dfrac{\Delta V}{V_0} - \Delta\varepsilon_1\right]}{2}$$

and hence

$$\upsilon' = \frac{\left[1 - \dfrac{\Delta V/V_0}{\Delta\varepsilon_1}\right]}{2}. \qquad (17.16)$$

Note from **Figure 17.23** that the slope of the $\Delta\sigma'_1$ versus $\Delta\varepsilon_1$ line is equal to E' and the downward slope of the $\Delta V/V_0$ versus $\Delta\varepsilon_1$ line is equal to $(1–2\upsilon')$.

17.8.7 Ideal undrained triaxial test

For an undrained test, the pore water cannot drain out of or into the sample. Hence 'excess' pore pressures develop which can be positive or negative. Unless the pore pressures are measured, we only know the applied total stresses. The undrained Young's modulus E_u is given by:

$$E_u = \Delta\sigma_1/\Delta\varepsilon_1. \qquad (17.17)$$

As shown in the next section, the undrained Young's modulus is not usually equal to the drained Young's modulus E'. For an undrained test, full saturation is usually ensured so that there is no volume change – the water can be assumed to be incompressible. Hence υ_u (the undrained Poisson's ratio) is 0.5. When the excess pore pressure Δu is measured, as is usually the case for consolidated undrained tests, its value may be used to calculate the pore pressure coefficient A, which is referred to later.

17.8.8 Relationship between drained and undrained Young's modulus

Since water cannot carry shear, an increment in shear stress $\Delta\tau$ is an effective stress, irrespective of drainage. We have seen that for an ideal isotropic porous elastic material the application of $\Delta\tau_{max}$ will not cause any volume change in a drained test (or any pore pressure change in an undrained test). Hence the only deformation is a shear strain $\Delta\gamma_{max}$ which must be identical for both drained and undrained conditions. Hence the effective shear modulus G' is identical to the undrained shear modulus G_u. From equation (17.12) and its undrained equivalent:

$$G' = \frac{E'}{2(1+\upsilon')} = G_u = \frac{E_u}{2(1+\upsilon_u)}. \tag{17.18}$$

Setting $\upsilon_u = 0.5$ and solving for E_u:

$$E_u = \frac{3}{2(1+\upsilon')}E'. \tag{17.19}$$

This result is an inevitable consequence of the idealisation of isotropic elastic behaviour. It has particular significance when calculating undrained and drained settlements of foundations.

17.8.9 Pore pressure changes during undrained loading

We saw from equation (17.11) that, for an isotropic porous elastic material, if there is no volume change during general loading $(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3)/3$ there will be no change in mean normal effective stress p', i.e. $\Delta p' = 0 = (\Delta\sigma'_1 + \Delta\sigma'_2 + \Delta\sigma'_3)/3$.

Since, from the effective stress principle $\Delta\sigma'_1 = \Delta\sigma_1 - \Delta u$, etc., it follows that:

$$0 = (\Delta\sigma_1 - \Delta u + \Delta\sigma_2 - \Delta u + \Delta\sigma_3 - \Delta u)/3 \tag{17.20}$$

and

$$0 = \Delta p = \Delta u \tag{17.21}$$

i.e. during undrained loading of a fully saturated porous isotropic elastic material, $\Delta u = \Delta p$ (assuming the pore fluid to be incompressible in comparison with the skeleton). For an undrained triaxial compression test on a fully saturated sample of the ideal material in which both the axial and the radial total stresses are changed, we take $\Delta\sigma_a = \Delta\sigma_1$ and $\Delta\sigma_r = \Delta\sigma_2 = \Delta\sigma_3$. Hence from equation (17.21):

$$\Delta u = \Delta p = \frac{(\Delta\sigma_1 + 2\Delta\sigma_3)}{3} = \Delta\sigma_3 + \frac{(\Delta\sigma_1 - \Delta\sigma_3)}{3}. \tag{17.22}$$

Equation (17.22) is written in the form of Skempton's pore pressure equation:

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)] \tag{17.23}$$

where A and B are experimentally determined coefficients. When the soil is fully saturated, $B = 1$. It can be seen by inspection of equations (17.22) and (17.23) that for an isotropic porous elastic soil, the value of A for an undrained triaxial test is fixed at 1/3. (It can be shown that under plain strain conditions the value of $A = 1/2$. Hence A is dependent on the total stress changes and is not a fundamental property, even for the ideal case of a porous isotropic elastic material.)

Further details on the determination and application of Skempton's pore pressure coefficients may be found in most basic soil mechanics textbooks, for example Craig (2004) and Powrie (2004). Note that the coefficient B can be measured by applying an increment of equal all-round stress $\Delta\sigma_3$ in an undrained triaxial test and measuring the change of pore pressure Δu. If B is less than unity then the sample is not fully saturated.

17.8.10 Summary of the properties of an ideal isotropic porous elastic material

Many soil mechanics problems are modelled using an ideal isotropic porous elastic material. We have shown in this section that this idealised material possesses some important and rather restrictive results which practising engineers should be aware of:

- Only changes in stress and strain are considered.
- Volume change only takes place if there is a change in mean normal effective stress, i.e. $\Delta v = \Delta p'/K'$.
- A change in shear stress only causes distortion and not volume change, i.e. $\Delta\gamma_{max} = \Delta\tau_{max}/G'$.
- The undrained Young's modulus E_u is uniquely related to the effective Young's modulus E' and the effective Poisson's ratio υ'.
- In undrained triaxial compression, the pore pressure coefficient $A = 1/3$.
- E' and υ' can be measured in the standard drained triaxial test.
- The one-dimensional volumetric compressibility m_v can be expressed in terms of E' and υ'.

17.8.11 The stress–strain behaviour of real soils

There are many useful solutions available for practical geotechnical problems based on ideal isotropic elastic materials – see, for example, Poulos and Davis (1974). In making use of such

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solutions, the engineer has to make allowances for the fact that the stress–strain behaviour of real soils differs from that of the ideal isotropic elasticity in a number of ways. The three most obvious differences are non-linear behaviour, the dependence of stiffness on confining pressure, and the effects of anisotropy. All three stem directly from the particulate nature of soils.

It was shown in section 17.7 that the compressibility of a soil reduces with increasing pressure. Similarly, research has shown that, in the same way that strength is significantly dependent on the initial effective confining pressure p'_0, so is the stiffness – as shown in **Figure 17.24**. Since p'_0 increases with depth, it can be anticipated that the value of E' will usually increase with depth.

In Chapter 14 *Soils as particulate materials*, it was shown that the process of deposition of a sediment leads to preferred arrangements of the particles so that their stiffness and strength properties will be anisotropic. The geotechnical engineer needs to be aware of this departure from the ideal of isotropic behaviour. It is relatively straightforward to develop the elastic equations for a cross-anisotropic porous elastic material in which the properties in the horizontal direction differ from those in the vertical direction. For such a material, five elastic constants are required: E'_v, υ'_{vh}, E'_h, υ'_{hh} and G_{vh}. The determination of all these constants is far from straightforward experimentally and the application of a cross-anisotropic model should be carried out by an expert.

It is evident from the above that real soils depart from ideal isotropic elastic behaviour in a number of respects. The application of simple elastic theory can be very instructive in solving practical geotechnical problems. However, the key to its successful application is to understand clearly the properties of such a material and how the real material departs from these properties. The application of elastic theory to the calculation of foundation settlement and stress distributions is discussed in more detail in Chapter 19 *Settlement and stress distributions*.

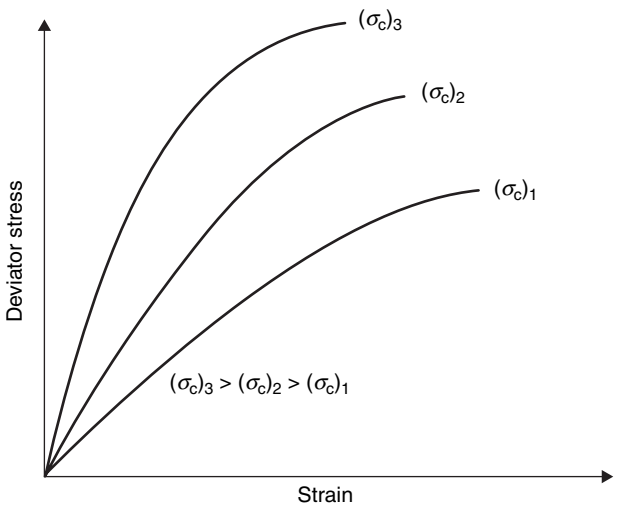


Figure 17.24 Influence of confining pressure on the drained stress–strain behaviour of soils

Most soils exhibit non-linear stress–strain behaviour of the form shown in **Figure 17.25**. A value of Young's modulus E' or E_u has to be selected which is appropriate for the range of stress changes involved. Frequently a value is chosen corresponding to about 0.1% axial strain. In sophisticated applications, numerical analysis is carried out using non-linear elastic formulations. The equations used are the same as those given in this chapter, but the elastic constants (E', G, K', etc.) are altered as the analysis progresses to reflect the reduction in stiffness of the soil as the strains increase. More advanced non-linear models make use of elastic–plastic formulations. A large number of non-linear constitutive models are available for undertaking the numerical analysis of geotechnical problems. When embarking on such complex analysis, it is important to remember that such models are idealisations and always carry with them a number of assumptions and limitations. Every effort should be made to understand these. For a detailed discussion of the benefits and limitations of the various soil models that are used in practice, reference should be made to Potts and Zdravković (1999). Non-linear analysis should only be undertaken with the advice and guidance of a specialist geotechnical engineer with expertise in numerical analysis.

17.9 Conclusions

The purpose of this chapter is to summarise the basic strength and stiffness properties of soils, emphasising the particulate nature of the material. In doing so, it has been necessary to relate the behaviour to simple idealised frameworks. Reference to the geotechnical triangle in Figure 4.1 of Chapter 4 *The geotechnical triangle* shows that there is a link between 'measured behaviour' in the bottom left-hand corner of the triangle and 'appropriate model' in the bottom right-hand corner. Simple models and idealisations are necessary to aid in prediction, but they need to be tempered with a clear understanding of real behaviour.

In the case of strength, the Coulomb criterion is universally used and the results of experiments on real soils have been compared with it. It is apparent that at least three measures of strength are used: peak strength, critical state strength and residual strength. In the case of peak strength, it has been shown that for dense or over-consolidated soils, the strength envelope is curved and depends on the initial relative density of the material. The engineer has to decide what measure of strength is relevant for a given problem and what values of φ' and c' to adopt in an analysis. This subject is covered in Chapter 27 *Geotechnical parameters and safety factors*; it is essential to have a clear understanding of the idealisations that are being made and their limitations.

Most clayey soils are very compressible compared with other structural materials and classical soil mechanics has focused particularly on the one-dimensional (confined) compressibility. In this chapter, discussion of the one-dimensional

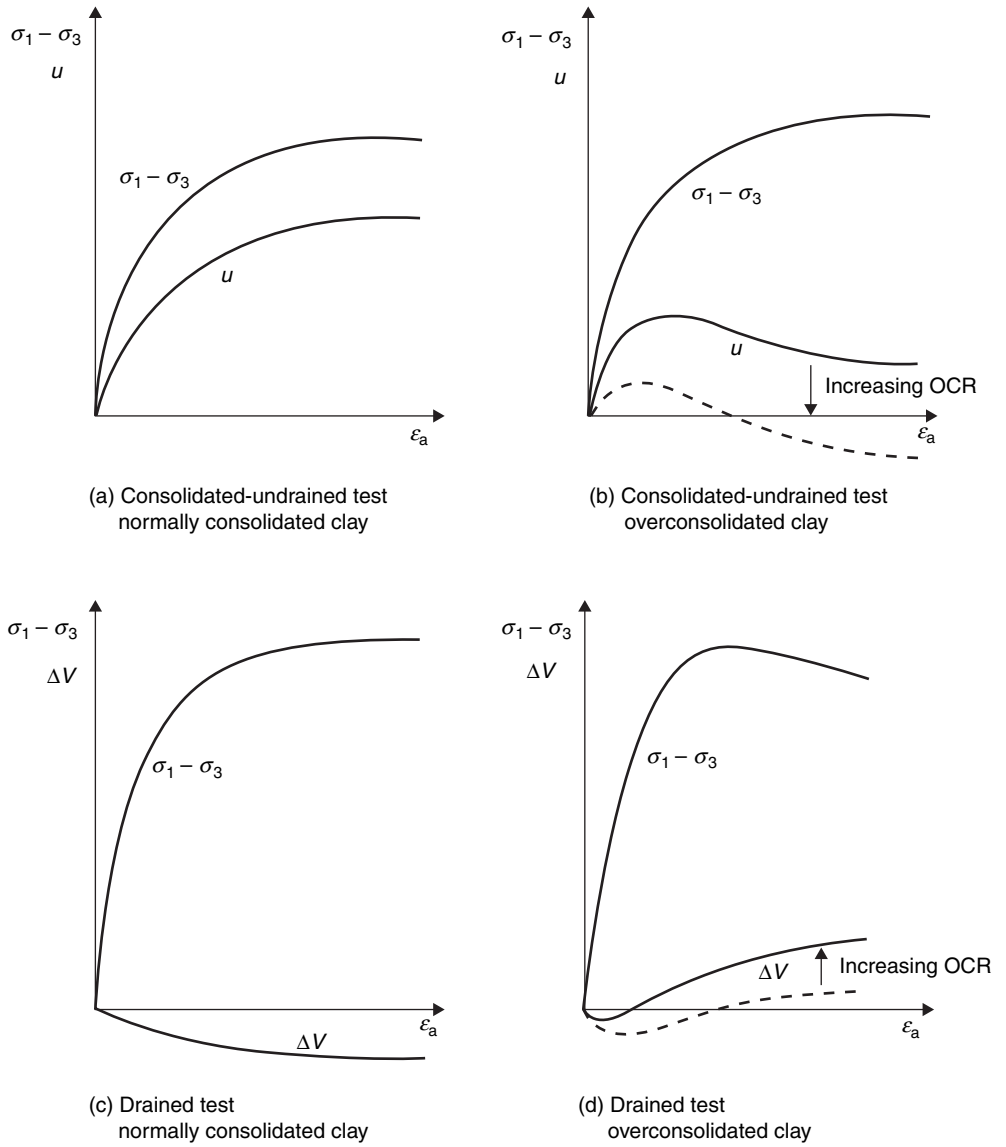


Figure 17.25 Typical stress–strain curves from consolidated undrained and drained triaxial tests

compressibility of soils follows traditional approaches. Particular emphasis is placed on the importance of inter-particle bonding which is present in most natural soils. In particular, the existence of the vertical yield pressure σ'_{vy} is introduced. This yield pressure represents the vertical effective stress at which the inter-particle bonds begin to break down and slip between the grains. In the past this pressure has been confused with the pre-consolidation pressure, but this can be misleading as most natural normally consolidated soils exhibit a value of σ'_{vy} which is greater than the effective overburden pressure σ'_{v0}. The ratio $\sigma'_{vy}/\sigma'_{v0}$ is known as the 'yield stress ratio'. A most useful empirical finding is that the undrained strength in triaxial compression S_{uc} is closely related to σ'_{vy} such that the ratio S_{uc}/σ'_{vy} usually lies roughly in the range 0.28–0.32.

Many practical solutions exist for deformation problems using simple isotropic elasticity. In this chapter, the properties of an ideal isotropic porous elastic material have been described in detail. This ideal behaviour has then been compared with the known stress–strain behaviour of real soils. It is hoped that this discussion will assist the engineer both in the choice of appropriate stiffness parameters and in assessing the limitations of predictions based on the simple elastic models. If a more complex analysis is required employing non-linear constitutive relations, it is important to remember that such models are idealisations and always carry with them a number of assumptions and limitations. Such an analysis should only be undertaken with the advice and guidance of a specialist geotechnical engineer with expertise in numerical analysis.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

Chapter 18
Rock behaviour

David J. Sanderson University of Southampton, Southampton, UK

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Rocks are naturally occurring, polycrystalline materials that play a wide range of roles in civil engineering. Their behaviour can be understood in terms of their intrinsic textures (grains, cement, voids and discontinuities) and through the application of standardised testing methods. Rocks and soils display a wide range of rheological properties, even under near-surface conditions. The void space in rocks is generally filled with water, at some pore pressure. For many engineering applications rocks may be considered as poroelastic, with a proportion of the applied load being supported by the pore pressure. Thus, their deformation is related to the effective stress (= total stress – pore pressure) as is generally found for soils. Rocks have a wide range of stiffness, strength and permeability, all of which will determine their suitability for different engineering applications. Most importantly, rocks are heterogeneous, displaying various forms of layering or grain fabrics, and are almost ubiquitously fractured. Thus the rock mass properties vary over a range of scales (heterogeneity) and orientations (anisotropy).

18.1 Rocks

Rocks are naturally occurring, polycrystalline materials that play a wide range of roles in civil engineering, ranging from their support for structures, to their use as construction materials.

All rocks can be considered as being made up of the following components (**Figure 18.1**):

Grains – discrete elements of either individual crystals or aggregates of crystals, usually having a distinct composition and shape;
Cement – usually crystalline material that binds the grains together;
Voids – spaces between grains, usually in the form of interconnected pores, often filled with water.
Discontinuities – macroscopic surfaces that separate the rock mass into blocks or layers. They may take the form of fractures (or joints), movement surfaces (faults), bedding planes or other surfaces. The discontinuities usually have markedly different physical properties from the rest of the rock material.

The composition and arrangement of these components gives rise to a variety of different rock types (that geologists describe with a bewildering range of names), which have a wide range of engineering properties. *The nature of the grains, cement, voids and discontinuities is important in evaluating the behaviour of rocks and rock masses.* Some understanding of the nature of rocks is necessary in order to discuss their physical behaviour.

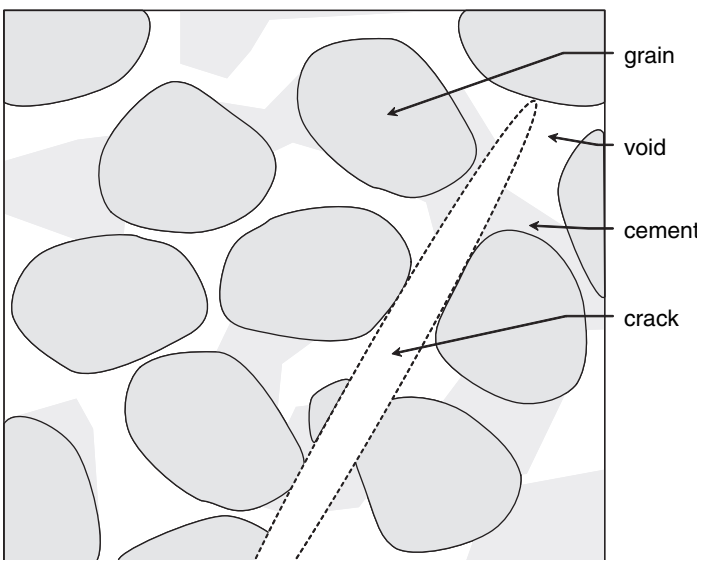


Figure 18.1 Diagrammatic representation of the components that make up a typical rock

18.2 Classification of rocks

Geologists recognise three broad categories of rocks, based on their mode of formation:

Sedimentary rocks – that form at or near the surface of the Earth from grains, usually eroded from previously formed

rock. The grains are then transported across the surface and deposited to form new ***sediments***. The main agents for these processes are water (in the form of rivers and seas), ice and wind. We can regard soils as a form of sediment.
Igneous rocks – that form by crystallisation of grains from molten rock either deep within the Earth (magma) or at the Earth's surface (lava).
Metamorphic rocks – that form deep within the Earth due to solid-state recrystallisation of other rock types.

Igneous and metamorphic rocks share many characteristics and consist of an interlocking aggregate of mineral grains, usually resulting in strong and stiff materials. The boundaries between grains are usually very narrow, with no cement, and it is the interlocking nature of these boundaries and the lattice-scale forces between grains that impart their strength. Void space is small, with the porosity (ratio of void space to total volume) generally being < 1%.

Sedimentary rocks on the other hand are generally formed as loose aggregates of grains, initially with a large amount of void space (porosities of 30% or greater). In the Earth, this porosity is usually saturated with water (and occasionally other fluids). The water may contain dissolved materials that can precipitate in the pore space to form cement.

At an initial stage, when porosities are high, the sediments resemble soils in their mechanical behaviour (see Chapter 17 *Strength and deformation behaviour of soils*). As the sediments are buried they experience elevated pressures and temperatures, leading to consolidation (usually by expulsion of water from the void space) and cementation. These processes essentially convert the 'soil' to rock. Near-surface chemical processes, generally termed diagenesis by geologists, accelerate the conversion of sediment to rock.

There is no clear boundary between rocks and soils, and many of the methods used to describe their behaviour are based on the same principles of continuum and granular mechanics. Both materials can exhibit considerable heterogeneity, often an important factor in assessing their engineering behaviour, but some important differences exist between the two.

Soils are essentially particulate (or granular) materials, with particle sizes that are usually many orders of magnitude less than the length scales of the imposed engineering loads. Thus they are generally treated as continuum materials, their micro-scale granularity being approximated by macroscopic parameters. On the other hand, the block sizes of rocks are often of a similar length scale to the applied loads and the discrete nature of rock masses is usually of greater importance in rocks than for soils (e.g. Hudson and Harrison, 1997).

Generally, both rocks and soils contain water, the void space often being saturated. Thus in both materials the effects of fluid pressure and the principles of effective stress are of great importance. However, the fundamental behaviour of soil changes dramatically with water content, changing from solid → plastic → liquid at the plastic and liquid limits, respectively. Rock properties are also affected by water, but not to anything like the same extent; for example, the strengths of dry and saturated rocks rarely differ by more than a factor of two.

18.3 Rock composition

An important distinction between the crystalline (igneous and metamorphic) and sedimentary rocks is the composition of the grains. The bulk composition of the Earth's crust is dominated by the elements silicon (Si) and oxygen (O) that readily combine to form the silica ion (SiO_4^{4-}). The next most abundant elements are the metals aluminium (Al), iron (Fe), magnesium (Mg), calcium (Ca), potassium (K) and sodium (Na), which form positively charged ions and combine with the SiO_4^{4-} to form a group of minerals known as rock-forming silicates. These include the common minerals quartz and feldspar, together with a complex array of other silicates (pyroxenes, amphiboles, micas, etc.). Most igneous and metamorphic rocks consist of a small number, typically three or four, of these rock-forming silicates, the composition of which depends on the chemistry of the patent magma or rock, and the pressure–temperature conditions under which the rock formed. The details of these minerals need not concern us here; what is important is that they generally have moderate to high strengths, contributing to the strength of igneous and metamorphic rocks.

Many of these rock-forming silicates weather on exposure to the Earth's surface, and are converted to various types of clay minerals and other salts; the latter often dissolve in water. An exception is the mineral quartz, which is highly resistant to weathering. Weathering leads to a reduction in strength of igneous and metamorphic rocks, and is facilitated by fracturing of the rock mass. Thus a key issue in the behaviour of such rocks is the rock mass quality.

Weathering leads to the development of sediment – hence, we would expect this to be composed mainly of quartz and clay minerals. Sediment is also generated by the precipitation of dissolved salts, either through evaporation (e.g. rock salt) or more commonly by biochemical action, mainly involving fixing of dissolved salts in the shells of invertebrate organisms. The nature of the sediment determines the type of sedimentary rock that is subsequently produced:

Quartz sand	→	sandstone
Clay minerals	→	claystone (also called mudstone or shale)
Calcite shells	→	limestone
Dissolved salts	→	rock salt (and other 'evaporates')
Organic matter	→	coal, oil, etc.

18.4 Porosity, saturation and unit weight

From the previous description of rock in terms of grains, cement and voids, we can consider the grains and cement together as a solid phase (**Figure 18.2**), with the voids being either liquid (pore water) and/or gas (air).

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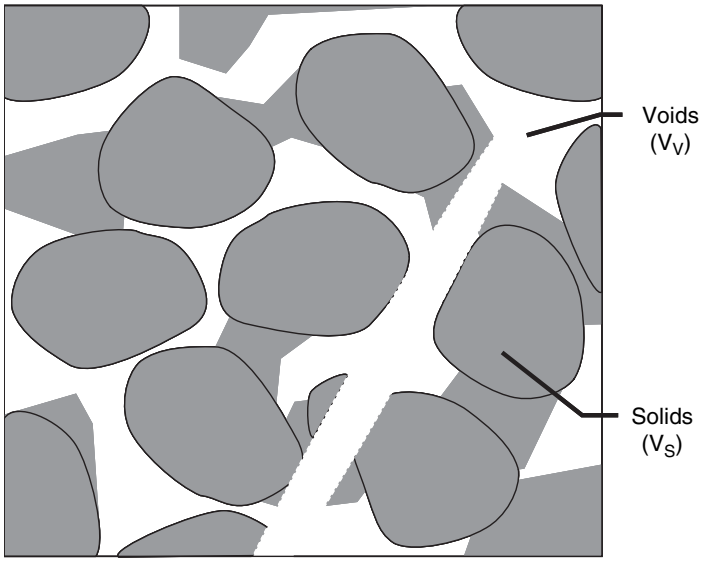


Figure 18.2 Rock components (as Figure 18.1) classified into solids (V_s) and voids (V_V)

The volume of the rock (V_R) is made up of solids (V_S) and voids (V_V), where $V_R = V_S + V_V$. The relative proportions of these define two parameters:

$$\textit{Porosity}\ (n) = V_V / V_R \text{ or } V_V / (V_S + V_V) \qquad (18.1a)$$

$$\textit{Voids ratio}\ (e) = V_V / V_S \qquad (18.1b)$$

where $$e = n / (1 + n) \text{ and } n = e / (1 + e). \qquad (18.1c)$$

Since the volume of voids can be filled with water and/or air: $V_V = V_W + V_A$, which we can express in terms of the **saturation** S, where:

$$S = V_W / V_V. \qquad (18.2)$$

Porosity, voids ratio and saturation are usually expressed as a fraction (0 – 1), but can be expressed as a percentage.

Knowing the densities (ρ) of the solid, water and air phases, and using g = 9.81, the weights of the phases may be calculated, where $W = \rho g$.

For example, $$\textit{unit weight}\ (\gamma) = (W_S + W_W + W_A) / (V_S + V_W + V_A) \qquad (18.3a)$$

$$\textit{unit dry weight}\ (\gamma_d) = W_S / (V_S + V_W + V_A). \qquad (18.3b)$$

These parameters are discussed more fully in Chapter 17 *Strength and deformation behaviour of soils*, but can provide a useful basis for characterising the multi-component nature of rocks and, hence, assessing rock behaviour. For example the uniaxial compressive strength (UCS) is related empirically to the porosity for different rock types, which provides a useful guide to rock strength in the absence of tests.

18.5 Stresses and loads

Stress exists throughout the Earth's crust and may be changed by surface and subsurface construction. There are four universal components responsible for this stress:

(1) The weight of the overlying column of rock, which is a relatively simple function of depth and rock density (unit weight), and is usually referred to as **overburden stress (σ_V)**.

(2) **Fluid (or pore) pressure (P)**, which arises from the fact that most rocks are basically two-phase materials consisting of mineral grains (which may or may not be cemented together) and fluid-filled pores and cracks.

(3) **Thermal stresses** arising from heating or cooling of rock, which tends to cause rocks to expand or contract.

(4) **Externally applied loads** that may be imposed by geological processes (tectonics, topography, etc.) or by construction.

These four components interact in different ways, but their combined effect is to act on or load materials to induce strain (change in shape and/or volume). In civil engineering we are mainly concerned with how the ground responds to externally applied loads, but we should not overlook the possible effects of other sources of stress.

Most rocks contain pores and cracks that are generally saturated by water. The externally applied loads create both a stress in the framework of grains and cement, and a pressure in the fluid, which we refer to as fluid pressure or pore presure. Terzaghi (1943) suggested that the applied loads may be supported by both the stress in the solids and the fluid pressure in the pores. At the grain boundaries, these two tractions oppose one another and create an **effective stress**, such that:

$$\text{Effective stress} = \text{Total stress} - \text{Pore pressure} \qquad (18.4a)$$

$$\sigma' = \sigma - P_f. \qquad (18.4b)$$

In a granular material, it is this effective stress that promotes deformation, with the relationship between effective stress and strain being determined by the rheology and properties of the material. The effective-stress principle is applied almost universally in soil mechanics and widely in rock mechanics. We will discuss this further in the section on poroelasticity (section 18.8).

18.6 Rock rheology

There are three basic responses of materials to applied stress that are easily recognised from plots of stress against strain or strain rate (**Figure 18.3**):

Elasticity – where the strain is linearly proportional to the stress (**Figure 18.3(a)**). This typifies the behaviour of solid materials and the ratio of stress to strain is referred to as the stiffness (Young's modulus, rigidity, etc.). In the ideal case, the

deformation is completely recovered on removal of the stress and the rock exhibits no significant change in structure. Many crystalline rocks approximate this behaviour and are fairly rigid, i.e. they exhibit a high stiffness, with Young's modulus being measured in GPa. This is one reason for their widespread use as construction materials.

Viscosity – is where the material flows and is the basic characteristic of liquids. In rock mechanics we often perform ***creep*** experiments where a sample is allowed to deform under a constant applied stress and the strain plotted against time, the slope being the strain rate (**Figure 18.3(c)**). If the strain rate is proportional to stress, the material exhibits linear or Newtonian viscosity, where the viscosity is the ratio of stress to strain rate. Some rocks exhibit Newtonian viscosity, but more generally the behaviour is more complex (nonlinear viscosity).

Yield – occurs when a material behaves elastically at low stress, but is ductile (i.e. flows) at higher stresses. The stress at which this transition takes place is the yield stress (**Figure 18.3(b)**). This behaviour is typical of plastic materials.

Rocks, in common with most other materials, exhibit all of these basic rheologies, the stiffness, viscosity and yield stress often being complex (nonlinear) and dependent on the temperature, confining pressure and strain-rate.

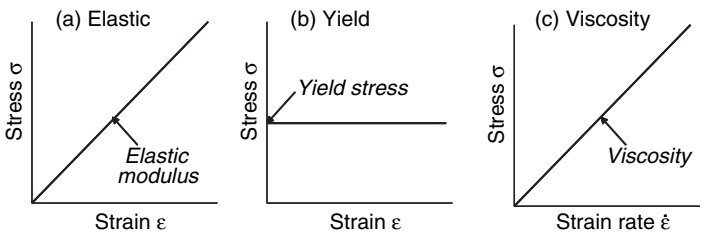


Figure 18.3 Idealised rheological behaviour showing (a) linear elasticity, (b) plastic yield and (c) Newtonian viscosity

It is useful, at least from a conceptual basis, to model rocks by combining the three basic rheological elements in various ways. For example, the stress–strain curves for rock (**Figure 18.4(a)**) at room temperature are elastic, whereas high confining pressures show a close correspondence to an elastic-plastic rheology. Confining pressure effectively inhibits failure and the rock exhibits a reasonably clearly defined yield stress at ~500 MPa (**Figure 18.4 (a)**). Yield stress and the stiffness decrease with increasing temperature (**Figure 18.4(b)**), as does the viscosity (not shown in **Figure 18.4**).

The rheology of a material is described by a ***constitutive law*** – an equation relating the deformation (strain) induced in the rock to the applied stress (or *vice versa*). We have encountered simple examples of these in the previous section. However, to define a constitutive law more rigorously we need to consider the relationships between different components of stress and strain. In general, the deformation can be described by a second-order strain tensor, ε_{ij}, which is related to the displacements (u):

$$\varepsilon_{ij} = \tfrac{1}{2}\left(\frac{\delta u_i}{\delta x_j} + \frac{\delta u_j}{\delta x_i}\right). \qquad (18.5)$$

Three commonly encountered deformation types (unconfined compression, simple shear and volumetric strain) are illustrated in **Figure 18.5**.

18.7 Elasticity and rock stiffness

For elastic behaviour, the relationship between stress and strain is linear and, for small deformations, can be described by a series of stiffness (or elastic) constants, defined by the ratio of different stress-to-strain components. The following elastic constants are widely used and relate to the three common deformation types (**Figure 18.5**) as follows:

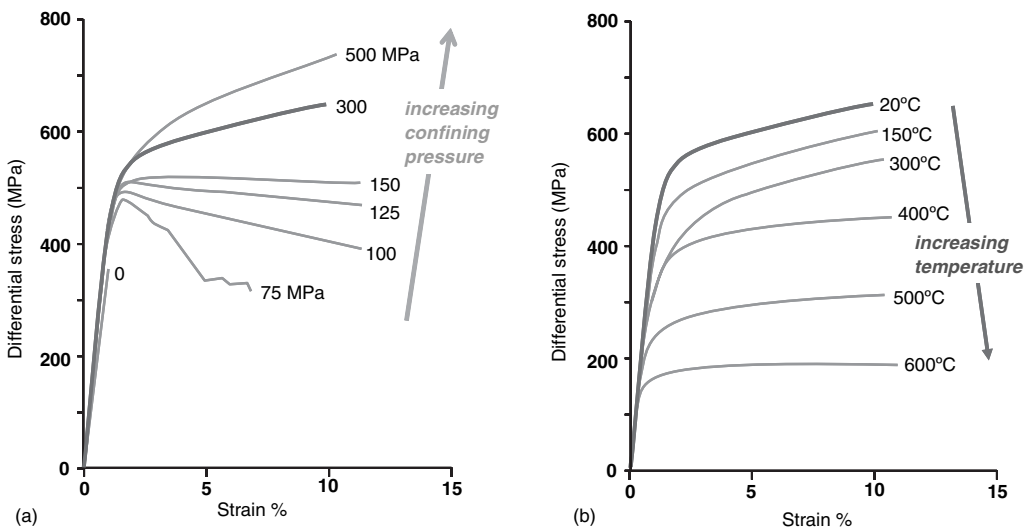


Figure 18.4 Stress–strain curves from triaxial tests on Solenhofen Limestone (after Heard, 1960). All tests at strain rate of 2×10^{-4} s^{-1}; (a) changing confining pressure at constant temperature of 25°C; (b) changing temperature at 300 MPa confining pressure

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Young's modulus (***E***) where $E = \sigma_{33} / \varepsilon_{33}$. This is the stiffness in unconfined (uniaxial) compression (**Figure 18.5(a)**), where σ_{33} is the only non-zero stress although there will be lateral strains in the other principal directions (i.e. $\varepsilon_{11} = \varepsilon_{22} \neq 0$).
Poisson's ratio (ν) where $\nu = -\varepsilon_{33} / \varepsilon_{11}$, is the ratio of lateral to axial strains in uniaxial compression.
Shear modulus (***G***) where $G = \frac{1}{2}(\sigma_{31} / \varepsilon_{31})$; it is easily determined from simple shear experiments (**Figure 18.5(b)**).
Bulk modulus (***K***) where $K = \sigma_{00} / \varepsilon_{00}$ and σ_{00} is the mean stress or uniform confining pressure (**Figure 18.5(c)**). The **compressibility** (c) is simply $1/K$.

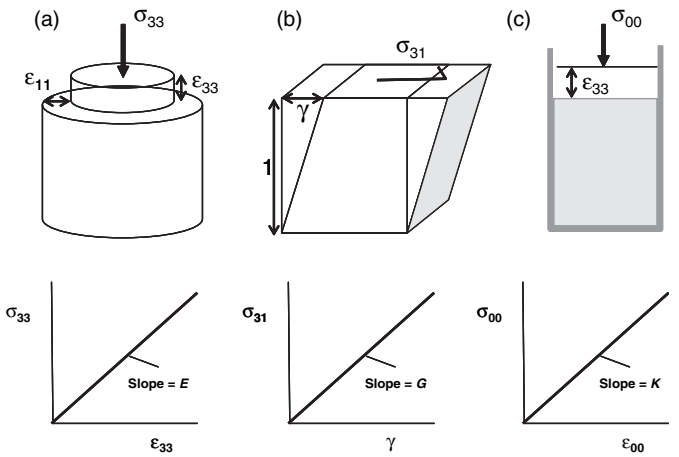


Figure 18.5 Three types of deformation involving different components of stress and strain and leading to different descriptions of rock stiffness

More generally the **constitutive equation** for linear elasticity is given by the equation:

$$\sigma_{ij} = \lambda \delta_{ij} \varepsilon_{00} + 2G \varepsilon_{ij} \tag{18.6}$$

where δ_{ij} is the Kronecker delta, which has a value 1 if i = j and 0 if i ≠ j. The term λ is a fifth elastic constant know as **Lame's constant** and, together with G, provides a more concise mathematical formulation of the constitutive equation in the theory of elasticity. The other, more practical stiffness constants can be related to λ and G, as follows

$$K = \lambda + 2G/3 \tag{18.7a}$$

$$E = (3\lambda + 2G) / (\lambda/G + 1) \tag{18.7b}$$

$$\nu = \lambda / [2(\lambda + G)]. \tag{18.7c}$$

Any two elastic constants can be used to define the material and, hence, all other constants (**Table 18.1**). The range of elastic properties in natural materials is very large, especially if we include water, as illustrated in **Table 18.2**.

The discussion of elasticity presented above assumes that rocks are homogeneous and isotropic. In reality a rock mass is inhomogeneous (properties vary with location) and anisotropic (properties vary with direction). A major contribution to anisotropy comes from the grain fabric due to depositional processes (commonly parallel to bedding) and ductile deformation. Fractures also contribute significantly to anisotropy, both on a macroscopic scale and due to microcracks (e.g. Goodman, 1989; Hudson and Harrison, 1997). This elastic anisotropy is

	Young's modulus E	Poisson's ratio ν	Bulk modulus K	Rigidity (Shear modulus) G	Lame constant λ
E, ν			$\frac{E}{3(1-2\nu)}$	$\frac{E}{2(1+\nu)}$	$\frac{E\nu}{(1+\nu)(1-2\nu)}$
E, G		$\frac{E}{2G}-1$	$\frac{EG}{3(3G-E)}$		$G\frac{E-2G}{3G-E}$
E, K		$\frac{3K-E}{6K}$		$\frac{3KE}{9K-E}$	$\frac{3K(3K-E)}{9K-E}$
ν, G	$2G(1+\nu)$		$\frac{2G(1+\nu)}{3(1-2\nu)}$		$\frac{2G\nu}{(1-2\nu)}$
ν, K	$3K(1-2\nu)$			$\frac{3K(1-2\nu)}{2(1+\nu)}$	$\frac{3K\nu}{(1-\nu)}$
ν, λ	$\frac{\lambda(1+\nu)(1-2\nu)}{\nu}$		$\frac{\lambda(1+\nu)}{3\nu}$	$\frac{\lambda(1-2\nu)}{2\nu}$	
K, G	$\frac{9KG}{(3K+G)}$	$\frac{3K-2G}{2(3K+G)}$			$K-\frac{2G}{3}$
λ, K	$9K\frac{K-\lambda}{3K-\lambda}$	$\frac{\lambda}{3K-\lambda}$		$3\frac{K-\lambda}{2}$	
λ, G	$G\frac{3\lambda+2G}{\lambda+G}$	$\frac{\lambda}{2(\lambda+G)}$	$\lambda+\frac{2G}{3}$		

Table 18.1 Conversion of elastic constants

	Crystalline rock	Soil	Water
Youngs's modulus (MPa)	4×10^4	$10^1 - 10^2$	0
Bulk modulus (MPa)	2×10^4	10^{-1}	2.2×10^3
Rigidity (MPa)	2×104	$10^1 - 10^2$	0
Yield stress (MPa)	$\sim 2 \times 10^2$	$10^{-3} - 10^{-1}$	0
Poisson's ratio	0.1 – 0.25	0.2 – 0.45	0.5
Viscosity (Pas)	$\sim 10^{19}$	$\sim 10^4$	10^{-4}

Table 18.2 Simplified material properties of rock, soil and water (liquid)

manifest in directional variation in seismic wave velocities, but discussion of this is beyond the scope of this study. We will return to anisotropy of rock mass strength in a later section.

18.8 Poroelasticity

Poroelasticity is an attempt to describe the deformation of a solid material with connected voids (pores) that are saturated in fluid, and is a useful description for many soils or rocks (e.g. Biot, 1941; Wang, 2000).

Consider a rock under an isotropic confining pressure σ_C, with pores that are subject to a fluid pressure P. The volumetric strain e will be affected by both pressures σ_C and P. We can conceptualise the material as comprising a solid 'framework' and a series of pores, shown schematically in **Figure 18.6(a)**. Based on the law of superposition, this stress system can be considered as the addition of (a) a confining stress (σ_C -P) acting on the outer boundary, with no pore pressure (**Figure 18.6(b)**), and (b) the pore pressure P acting on all boundaries (**Figure 18.6(c)**).

The total volumetric strain e_A is simply:

$$e_A = e_B + e_C \tag{18.8}$$

but:

$$e_B = 1/K\ [\sigma_C - P] \text{ and } e_C = 1/K_G\ [P] \tag{18.9}$$

where K and K_G are the bulk modulii of the rock and mineral grains, respectively.

Combining equations (18.8) and (18.9) and rearranging gives:

$$e_A = 1/K\ [\sigma_C - \alpha\ \Delta P]. \tag{18.10}$$

Thus, deformation is produced by an effective stress:

$$\sigma' = K\ e_A = \sigma_C - \alpha\ P \tag{18.11}$$

where $\alpha = (1 - K/K_G)$ is a dimensionless constant, usually referred to as the Biot constant. Note that equation (18.11) is identical to (18.4b), and hence Terzaghi's effective stress principle, for the case $\alpha = 1$.

For unconsolidated materials, where $K << K_G$, the Biot constant is $\alpha \approx 1$, as was originally proposed for soils (Terzaghi, 1943). For unfractured, crystalline rock the grain framework

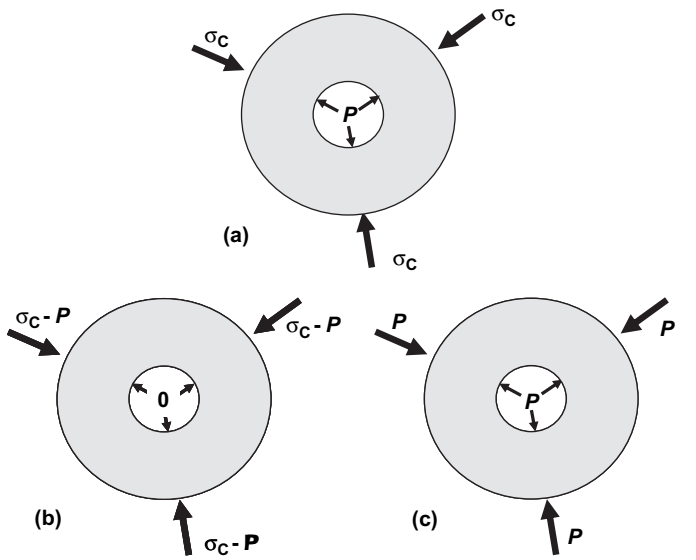


Figure 18.6 Decomposition of confining stress (σ_C) and pore pressure (P) – for explanation see text

may be stiffened by cement and/or interlocking grains, thus $K \rightarrow K_G$ and $\alpha \rightarrow 0$. Partially cemented sediments and fractured rock would be expected to have $0 < \alpha < 1$. For example, the bulk modulus for quartz is ~38 GPa, but that of sandstones is around 5–10 GPa; therefore we would expect $\alpha = 0.75$–0.9.

18.9 Failure and rock strength

In simple terms, we can recognise two broad types of failure in rocks:

(a) Brittle failure is where the rock undergoes some sort of fracture, usually accompanied by a volume increase (as cracks develop) and preceded by an essentially elastic behaviour of the rock mass.

(b) Ductile failure is where the rock undergoes some form of plastic yielding, usually accompanied by a volume decrease (collapse of pore structure) and preceded by some form of elastic compression or compaction.

Traditionally rock mechanics has focused on the brittle type of failure and soil mechanics on ductile yielding, but in reality both types of failure are seen in both rocks and soils.

The strength of a rock can be considered as the stress at which some sort of failure occurs. In rocks, strength is a complex concept and will depend not only on the properties of the rock components (grains, cement, voids and discontinuities) and their interactions, but on the type of failure and the conditions under which it occurs. Thus any specification of rock strength must include a careful description of the test conditions, or have been obtained under standard (i.e. pre-specified) conditions.

A simple view of rock failure is represented in **Figure 18.7**, which incorporates three widely recognised types of failure criteria.

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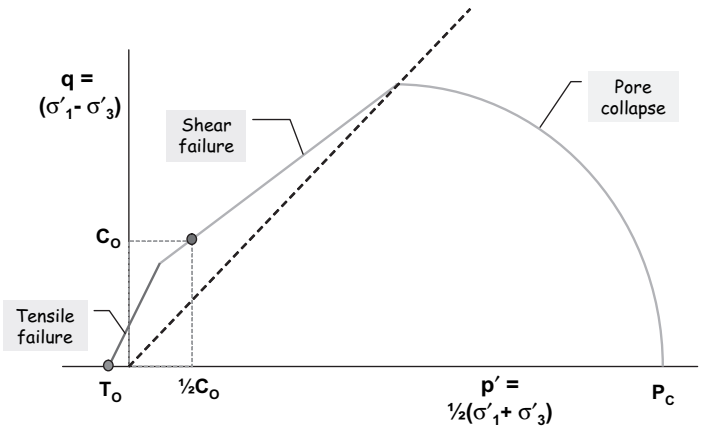


Figure 18.7 Plot of differential stress (q) against mean effective stress (p′) showing the three main types of failure mechanism T_O – tensile strength, C_O – uniaxial compressional stress (UCS), P_C – preconsolidation pressure

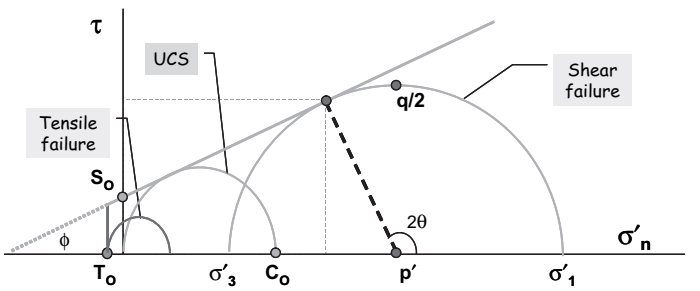


Figure 18.8 Mohr diagram (plot of shear stress, τ, against normal effective stress, σ'_n) showing conditions for tensile and shear failure T_O – tensile strength, C_O – uniaxial compressional stress (UCS), S_O – cohesion, φ – angle of friction, p' mean effective stress, $q/2$ – maximum shear stress = ½ (differential stress)

Tensile failure occurs when one of the principal effective stresses is negative (tensile) and cracks form normal to the minimum principal stress (σ_3), the magnitude of which is the tensile strength (T_O) i.e.

$$T_O = -\sigma_3. \tag{18.12}$$

The concept is simple, but direct tensile testing is difficult to achieve and interpret, and most tensile strengths are measured indirectly, e.g. from a Brazilian test. For most rocks tensile strengths are low, generally 0–30 MPa.

Following the work of Griffith (1921), we now appreciate that materials fail in tension due to the growth of micro-cracks. For a uniaxial remote tensile stress (σ_r), failure will occur when the stress concentration at the crack tip attains a critical value (K_C – the critical stress intensity factor or fracture toughness).

$$K = Y\,\sigma_r(\pi a)_{½} \tag{18.13}$$

where a is the half-length of the crack and Y is a factor (usually ~1) that depends on the geometry of the crack and sample.

Combining equations (18.12) and (18.13) and using $Y = 1$ gives:

$$T_O \approx K_C\,(\pi a)_{-½}. \tag{18.14}$$

Thus the tensile strength is a function of fracture toughness (a material property) and crack length (a textural property). The micro-cracks and other defects in an unfractured rock generally approximate to the size of the grains or pores (10^{-3} to 10^{-4} m) and K_C is in the range 0.3–3 MPa $m^{½}$. Thus, a tensile strength of 5–170 MPa would be expected for most rocks, as is observed.

Shear failure occurs on planes oblique to the principal stresses that, hence, experience a shear stress. A simple and widely used criterion was developed by Coulomb, which states that the shear stress (τ) must exceed a linear function of the normal stress (σ_n), such that:

$$\tau \geq S_O + \mu\,\sigma_n \tag{18.15}$$

where S_O is the cohesion – the shear stress required for failure in the absence of a normal stress, and μ is the coefficient of internal friction. This behaviour is analogous to frictional sliding (see section 18.11).

Coulomb failure is represented by a straight line on a plot of shear stress against normal stress – the Mohr diagram (**Figure 18.8**). Failure will occur on planes where 2θ is normal to the failure envelope, i.e. $2\theta = 90 + \varphi$, where φ is the angle of friction and $\mu = \tan(\varphi)$. For most rocks $\varphi = 20°–50°$, hence $\theta = 55°–70°$, with shear fractures forming at 20°–35° to the maximum compressive stress (σ_1) – a commonly observed orientation in tests and nature.

When triaxial test data are plotted on either the Mohr diagram or on a plot of σ_1 against σ_3 they tend to show a nonlinear relationship. Hoek (1968) originally proposed an empirical failure criterion for rocks, which was modified by Hoek and Brown (1980) to:

$$\sigma_1 = \sigma_3 + [m\,C_O\,\sigma_1 + s\,C_O^2]^{½} \tag{18.16}$$

where C_O is the uniaxial compressive strength, and the parameters m and s are chosen to best fit the data. For more discussion of this criterion see Chapter 49 *Sampling and laboratory testing*.

Tensile and shear failure both involve the development of cracks and are generally accompanied by small volume increases (dilatency). At high confining pressures, many porous rocks show a form of plastic yielding associated with localised **pore collapse** and compaction. This behaviour is typical of soils (Chapter 17 *Strength and deformation behaviour of soils*). This is shown schematically in **Figure 18.7** by the addition of an 'end cap' which meets the p′ axis at a pressure equivalent to the pre-consolidation pressure in soils. Stress paths, such as that produced by hydrostatic compression (q = 0), that lead to failure by pore collapse have a lower ratio of q/p than that for

shear failure. Such deformation is best analysed using the techniques of critical state soil mechanics (see Chapter 17 *Strength and deformation behaviour of soils*).

18.10 Strength testing

The field and laboratory testing of rock samples is described in detail elsewhere (Chapters 47 *Field geotechnical testing* and 49 *Sampling and laboratory testing*). Tests are usually performed on rock cylinders of standard shape and size, and some are briefly introduced below.

- **Uniaxial (unconfined) compressive test –** A simple and widely used test in which a cylinder or cube of rock is compressed between platens with no confining stress. The stress producing failure is the unconfined compressive strength (UCS).
- **Brazilian test** – A cylinder of rock is loaded between two platens transverse to its axis. This test is used to determine the tensile strength (T).
- **'Triaxial' test** – Axial compression (σ_1) of a cylinder under a radial confining stress ($\sigma_2 = \sigma_3$) – geometry similar to UCS. This test is usually run at several confining stresses and the failure envelope constructed, usually by plotting on a Mohr diagram (as in **Figure 18.8**).
- **Hydrostatic compression test** – This is usually carried out in a triaxial rig and involves increasing the confining pressure, in the absence of an axial load, until there is a volumetric collapse of the pore space. This is similar to the pre-consolidation pressure in soil mechanics and is used to define the plastic yield surface (**Figure 18.7**).
- **Shear tests** – The direct shear test involves a simple shear loading of a rectangular prismatic sample in a shear box. It is widely used to measure the shear strength (S_S) of soils, but is not suitable for anything but the weakest of rocks. A **torsional ring-shear test** has been developed that involves the twisting of a hollow disk between rigid end-disks to impart a shear on the specimen, which can be applied to a wider range of rock strengths.
- **Point load test** – This is a widely used test involving transverse loading of a cylinder of rock between two conical 'points' of standard shape (60º conical angle and tip radius of 5 mm). It can be performed with portable apparatus and applied to borehole core and irregular rock samples.

Other, field-based methods of measuring rock strength have been developed, which usually rely on an empirical calibration of some strength measure (usually UCS) to some physical response measured in the apparatus. A good example of this approach is the widely used **Schmidt hammer**, which measures the rebound of a spring-loaded rod propelled against a rock surface to estimate UCS. A **scratch test** determines the normal and tangential forces required to attain a constant depth of scratch on a rock surface, which can be related empirically to UCS.

A scheme for characterising rock strength based on simple field classification is outlined in **Table 18.3**. Essentially this is a subjective scheme based on the response of the rock to a series of simple physical tests using one's hand or a hammer. The scheme conforms broadly with the Working Party Report (Geological Society of London 1977) and found in Clayton *et al*. (1995) and Waltham (2009).

In terms of using these tests to describe rock behaviour it is important to remember that they are usually carried out on small pieces of the rock mass, typically at the cm-scale. Such tests may be useful in the characterisation of pieces of aggregate and building stone, but do not directly characterise the resultant structures (concrete, wall or foundation). Nor do the tests relate directly to the behaviour of the rock mass as a whole in groundworks, slopes, tunnels, etc. The following considerations are important in utilising laboratory estimates of rock strength.

- **Sampling –** Is the small specimen used in a test representative? All rocks are heterogeneous, usually at a wide range of scales – does the sampling capture this heterogeneity? This is particularly important in evaluating many sedimentary rock units that are made up of layers of different rock types and textures, with samples of a suitable size often being much easier to obtain from the thicker and stronger layers. These effects may be less of a problem in igneous and metamorphic rocks, although the fabrics in the latter produce anisotropy, which requires careful treatment (see section 18.16).

Description	UCS (MPa)	φ	Cohesion (MPa)	Field test	Rock type(s)
Very strong rock	300	50	20	repeated hammering to break	most igneous rocks
Strong rock	100	45	9	breaks with hammer	greywacke, quartzite, gneiss
Average rock	30	40	3.3	dented by hammer	sandstone, limestone
Moderately weak rock	10	35	1.35	cannot be broken by hand	shale, claystone
Weak rock	3	32	0.46	crumbles under hammer blows	soft chalk
Very stiff soil / Very weak rock	1	30	0.17	easily broken by hand	sand
Stiff soil	0.3	28	0.054	indented by finger nail	marl
Firm soil	0.1	25	0.020	moulded by fingers	clay
Soft soil	0.03	22	0.007	easily moulded by fingers	clay

Table 18.3 Field estimation of rock strength (UCS, internal friction angle for intact rock (φ) and cohesion)

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- ***In situ* conditions** are difficult to replicate in a laboratory test for a number of reasons. Changes in boundary stress, and fluid pressure and saturation occur during the extraction, transport and storage of samples that can lead to permanent physical and chemical damage, which cannot be reversed under test conditions. This is common in soils and poorly consolidated rocks, but is less important in most rocks.
- Virtually all routine tests are on intact rock and *do not measure the contribution of the rock fractures.*

18.11 Behaviour of discontinuities

Discontinuities are macroscopic surfaces that separate the rock mass into blocks or layers. They include fractures (or joints), movement surfaces (faults) and bedding planes or other surfaces. The discontinuities usually have markedly different physical properties from the rest of the rock matrix and may fail in tension or shear.

Experiments of sliding on planar surfaces cut in rock show a simple linear relationship between the shear stress (τ) and normal effective stress (σ_n') at low normal stresses (**Figure 18.9**), such that:

$$\tau = \mu_S \, \sigma_n' \tag{18.17}$$

where μ_S is the coefficient of sliding friction. Byerlee (1978) proposed an average of $\mu_S = 0.85$, but at higher normal effective stresses ($\sigma_n > 200$ MPa), he found:

$$\tau \approx 100 + 0.65 \, \sigma_n' \text{ (MPa)}. \tag{18.18}$$

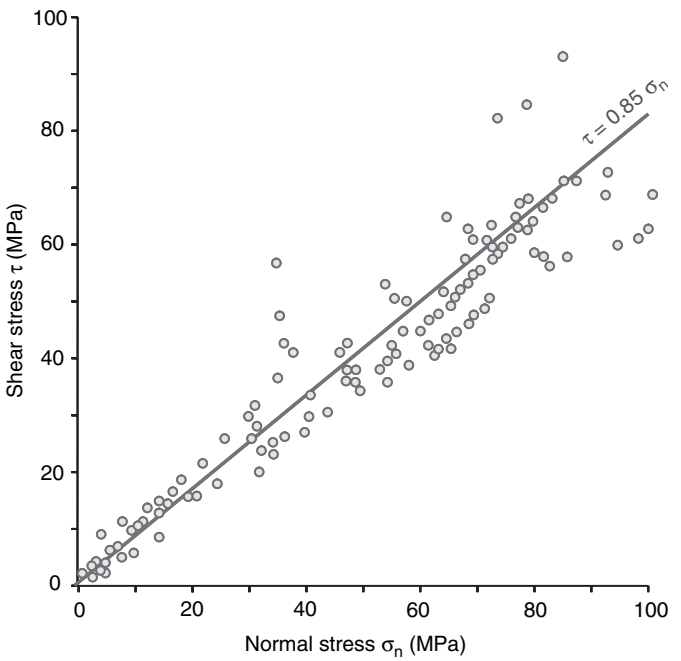


Figure 18.9 Data obtained by Byerlee (1978) for frictional sliding of planar rock surfaces at low normal stress

These results appear to be largely independent of rock type (at least for well consolidated and crystalline rocks). Based on Byerlee's results, it is widely assumed that cohesionless friction characterises many fractured rocks, with coefficients of sliding friction generally being between 0.65 and 0.85 – this has become known as 'Byerlee's Law'. The presence of weak material (e.g. clay fault gouge) in the fault plane can considerably reduce the coefficient of sliding friction. Water in the fractures mainly supports a pore pressure that controls the effective stress.

Since both shear and normal stress depend on the orientation of the surface in relation to the principal stress axes, failure by frictional sliding on rock fractures will be strongly controlled by the orientation of the fractures. If, however, sufficient variation in fracture orientation exists in a rock mass, then shear failure on optimally oriented fractures may be the dominant failure mechanism (e.g. Zhang and Sanderson, 2001).

18.12 Permeability

Rocks consist of solid phases (grains and cement) separated by voids. Flow of fluid between connected voids creates permeability in the porous medium. Flow is possible only through the connected void fraction, but for many rocks this is most of the void space. An exception is a rock such as pumice, which is a volcanic rock with isolated, unconnected gas bubbles; these make the rock light enough to float in water, but the rock is virtually impermeable – hence does not get 'waterlogged' and sink.

For most porous rocks, the flow is laminar (Reynolds number R_e<<1000) and the flow rate is linearly proportional to the pressure gradient. This was originally demonstrated by Henry Darcy in 1856, who showed that the flux (Q – m^3s^{-1}) of water in a pipe full of sand and subjected to a constant pressure drop (ΔP) or head (h) was proportional to its cross-sectional area (A) and inversely proportional to its length (L). Thus:

$$Q = -K_H \, A \, \Delta P / L. \tag{18.19}$$

The negative sign indicates that flow is from high to low pressure and the constant (K_H) is known as the hydraulic conductivity. K_H has SI units of ms^{-1} and depends on the nature of both the rock and the fluid.

$$K_H = k \, \rho \, g \, / \, \mu \tag{18.20}$$

where ρ and μ are the density and viscosity of the fluid, g is the gravitational acceleration and k the intrinsic permeability of the material. The intrinsic permeability has units m^2 with rocks typically having values in the range 10^{-18} to 10^{-10} m^2. The Darcy ($\approx 10^{-12}$ m^2) is often a more convenient unit as many rocks have permeabilities in the Darcy to milliDarcy range.

In civil engineering we are normally concerned only with groundwater, whose properties change relatively little. For water, at room temperature, $\rho g/\mu \approx 10^7$ in SI units; hence, $K_H \approx 10^7 \, k$. Again because of the low values K_H is often measured in

m/day or m/year. Another common practice in civil engineering is to refer to K_H as a 'coefficient of permeability' (or even just 'permeability'), a situation that can lead to confusion, and one that is both unnecessary and unhelpful.

A porous medium can be modelled as a system of pores, with porosity (n), connected by much finer tubes or throats of radius (r). Models can be used to relate the pore structure to the intrinsic permeability. For example, a simple model of a bundle of cylindrical tubes gives:

$$k \approx r^2 n / 8. \quad (18.21)$$

Thus for $r = 10^{-5}$ m and $n = 0.1$ this yields an intrinsic permeability of ~1 Darcy, equivalent to a hydraulic conductivity of a little less than 1 m/day.

The size and distribution of rock components (grains, voids, cement and fractures) have a major influence on the hydraulic conductivity of rocks.

18.13 Fracture-controlled permeability

Many rocks have very low porosities, such as most igneous and metamorphic rocks, and many cemented limestones. Others have very small grain size (and hence pore-throat radius), such as claystones, and this results in very low intrinsic permeability. Small specimens when tested in the laboratory confirm this. On the other hand when tested in the field, many of these rocks have permeability in the milliDarcy to Darcy range that can be attributed to flow in fractures. Thus it is the rock mass, rather than grain-scale, properties that control flow.

Fracture flow can be modelled by laminar flow between two parallel plates (**Figure 18.10(a)**) where:

$$Q = -(Wh^3/12) \cdot (\rho g/\mu)\, \Delta P/L. \quad (18.22)$$

This is the well-known 'cubic flow law'. Since the cross-sectional area A = Wh, it follows that the intrinsic permeability of the fracture is $k = h^2/12$; this would be the permeability of a layer assigned to model the fracture.

For a set of fractures oriented parallel to the pressure gradient (**Figure 18.10(b)**), with aperture (h) and density (d = number per unit length), then from equation (18.22), it is clear that the intrinsic permeability of the rock mass as a whole is proportional to the fracture density (d) and the cube of the fracture aperture (h) and is

$$k = d\, h^3 / 12 \quad (18.23)$$

since d has SI units m^{-1}, it follows that k has units m^2.

Fractures are an important component of the permeability of many rocks. A single fracture of aperture $h = 100$ microns (10^{-4} m) in a cubic metre, of rock (i.e. $d = 1$ m^{-1}) would provide a permeability of ~10^{-13} m^2 (or 100 milliDarcys), from equation (18.23). This is equivalent to the permeability of many sandstones used as oil or water reservoirs and would represent a hydraulic conductivity $K_H \approx 1$ m/day.

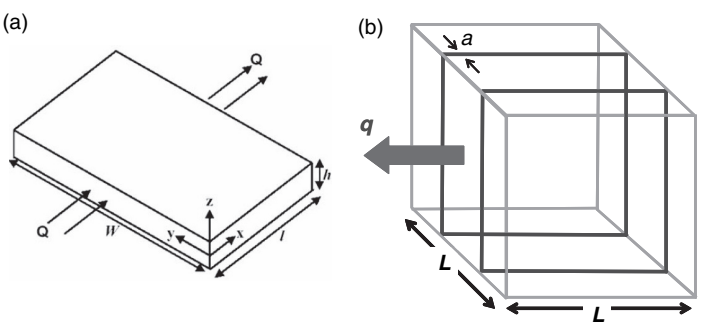


Figure 18.10 Flow of fluid through set of parallel fractures

18.14 Rock mass characterisation

Rock mass classification schemes have been around for over a century. They have generally been developed for assessment of behaviour in different engineering environments (tunneling, slope stability, etc.). Early schemes were simple and qualitative, e.g. Terzaghi (1946); more recent schemes use standardised measures and an algorithmic approach that can be implemented using spreadsheets.

Terzaghi's descriptions (simplified from his original paper) are:

- *Intact* rock contains no discontinuities and, hence, breaks across the rock matrix, either across grains or along grain boundaries.
- *Stratified* rock consists of layers separated by boundaries that may represent discontinuities across which the rock may have less resistance to separation and/or shear.
- *Moderately jointed* rock contains a network of joints that generally separate the rock into blocks, which interact along a large proportion of their surface and may be intimately interlocked.
- *Blocky and seamy* rock consists of grains and other rock fragments that are imperfectly interlocked and interact through small areas of contact.
- *Crushed* rock consists of small fragments and grains with no re-cementation that generally are surrounded by voids and interact at point contacts. The voids may be saturated with fluid.
- *Squeezing* rock flows into large voids (tunnel and other excavations) without perceptible volume increase. Such rocks typically have microscopic particles of mica or clay minerals with a low swelling capacity.
- *Swelling* rock flows into large voids on account of expansion. Such rocks contain clay minerals such as montmorillonite, with a high swelling capacity.

This scheme is entirely qualitative, but has the advantage of focusing attention on the main characteristic of the mass behaviour. It is rarely used in modern engineering design.

Rock quality designation (RQD) is a single parameter used widely to describe the degree of rock fracturing in borehole core and was developed by Deere *et al.* (1967). RQD is simply the percentage of intact core pieces longer than 0.1 m (4 inches) in the total length of core. The core should be at least

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50 mm or 2 inches in diameter. RQD is dependent on the spacing and direction of fractures relative to the borehole axis, and should exclude any fractures induced by drilling or handling of the core.

RQD clearly depends on spacing of fractures – the inverse of the number of fractures per unit length (λ). For a negative exponential distribution of fracture, i.e. one obtained from random placement of fractures, Priest and Hudson (1976) showed that:

$$\text{RQD} = 100\, e^{-\lambda t}\,(1 + \lambda t) \qquad (18.22)$$

where t is the threshold length used to determine RQD. Using the conventional value t = 0.1 m, RQD varies between 5% and 95% in a range of 3 to 50 fractures/m and thus is insensitive to variations outside this range.

Rock mass rating (RMR) was designed for the estimation of rock strength (Bieniawski, 1973, 1989) and it uses six quantitative measures to classify a rock mass:

1. Uniaxial compressive strength of rock material
2. Rock quality designation (RQD)
3. Spacing of discontinuities
4. Condition of discontinuities
5. Groundwater conditions
6. Orientation of discontinuities.

Values of each of these measures are used to define 'ratings'. For measures 1–5 in the above list these ratings are numbers usually between 0 and 15–30, which sum to a total between 0 (good rock) and 100 (poor rock). The final factor (6) is expressed as a negative rating reflecting the favourable (low –ve value) or unfavourable (high –ve value) of the orientation of fractures to the structure (tunnel wall, slope, etc.). The magnitude of the –ve value also varies with the type of excavation, being greater for slopes than tunnels.

Factors 2 and 3 above are closely related, and together account for 40% of the RMR, indicating that the 'degree of fracturing' is a key factor in this rock mass classification. RMR also incorporates the condition of the fractures (alteration and water content as well as the strength of the rock matrix.

The classification system is generally applied to different regions within the rock mass, usually based on the distribution of different rock types and major discontinuities.

18.15 Rock tunnelling quality index, *Q*

This was developed by the Norwegian Geotechnical Institute for underground excavations (Barton *et al*., 1974) and is based on six quantitative measures:

1. RQD (as defined above)
2. Jn – the number of joint sets
3. Jr – the joint roughness number
4. Ja – the joint alteration number
5. Jw – the joint water reduction factor
6. SRF – a stress reduction factor.

These parameters are used to define three ratios:

1. RQD/Jn represents the structure of the rock mass. As RQD is assessed relative to a 10 cm length of intact core, the ratio is crudely related to the size of fracture blocks.
2. Jr/Ja is a ratio that accounts for the roughness and frictional characteristics of the fracture walls and/or filling materials.
3. Jw/SRF is a ratio that consists of two stress parameters and attempts to describe the *in situ* loading conditions within the rock mass. Jw is a measure of water pressure, which reduces the effective normal stress, hence the shear strength of fractures. SRF is a measure of the total stress or load applied to the rock mass, but also incorporates effects such as loosening due to excavation and squeezing loads in plastic materials.

For a simple introduction to the Q system see Waltham (2009, pp. 86–87).

Both the RMR and Q schemes use geological and engineering parameters to provide a quantitative assessment of rock mass quality. Many of the parameters used are similar, but the calculations and weightings differ. Both schemes consider the degree of fracturing and the conditions of the fractures (groundwater, roughness, alteration, etc.). A significant difference is that RMR uses compressive strength directly, whereas Q only considers this in relation to *in situ* stress (through the SRF parameter). The RMR also incorporates the orientation of the fractures relative to the structure. Some estimate of orientation can be incorporated into Q using guidelines presented by Barton *et al*. (1974).

Both the RMR and Q schemes are best implemented using standardised procedures, which can be easily and consistently executed using spreadsheets, and details of the precise implementation of the methods are not given here.

18.16 Anisotropy

As well as the presence of discontinuities, there are often marked differences in rock properties with orientation within the rock mass. The major causes of such anisotropy are related to depositional layering in sedimentary rocks and flow fabrics in metamorphic and some igneous rocks; generally the latter have the greatest homogeneity. The anisotropy of rocks can be of three fundamentally different types:

1. Intrinsic anisotropy at the grain-scale, caused primarily by the preferred orientation of mineral grains. The mineral lattice properties may be strongly isotropic, hence alignment produces a macroscopic anisotropy in the rock sample. Alternatively the mineral grains may have a shape fabric and, hence, even for isotropic minerals, the alignment of

grain boundaries produces anisotropy. Preferred orientation of micro-cracks can produce similar anisotropy.

2. Many rock masses consist of layers of different rock materials and these produce a composite material, whose properties are not simply the average of the two (or more) materials involved. An example of this sort would be thin clay layers in sandstones, where the lower stiffness, strength and permeability of the clay may dominate the physical behaviour of the material. In such cases there is usually a strong directional dependence on the material properties.
3. The presence of fractures (see section 18.11).

Recognition of one or other of these types of anisotropy necessitates careful design of procedures to characterise the material – sampling different layers, testing large representative samples, testing in different orientations, etc. Conversely, one must be very careful in applying test results to construction in such anisotropic and/or composite materials.

For example, anisotropy is a major consideration in many claystones (shales). Stiffness is generally greater for compression normal to layering. Strength (e.g. UCS) is generally greater for compression normal to layering, but for shear failure in fissile shales the strength may be less when loaded in directions oblique to layering.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

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Chapter 19

Settlement and stress distributions

John B. Burland Imperial College London, UK

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This chapter describes the basic steps involved in calculating both the undrained and drained settlement of foundations on clay soils. It is demonstrated that the vertical stress changes beneath loaded areas, derived from simple elasticity, can be used with confidence for this purpose. In contrast, the changes in horizontal and shearing stresses are very sensitive to the assumed stress–strain behaviour and the theory of elasticity is much less reliable for calculating these. The four most common methods for calculating the final settlement are described. The somewhat surprising conclusion is reached that the simple one-dimensional method of analysis is even better than the more sophisticated methods, provided the appropriate soil stiffness parameters have been obtained. Settlements on granular soils are usually small and much of it takes place during construction. Numerous empirical methods have been proposed for predicting the settlement of foundations on granular materials, which lead to a wide range of estimates. A straightforward analysis of the measured settlements of over 200 records has led to a remarkably simple approach relating settlement to bearing pressure, breadth and average standard penetration test blow count, or cone resistance, over the depth of influence.

19.1 Introduction

This chapter describes in simple terms the most widely used methods of settlement analysis. These range from the simple classical one-dimensional (1D) method through to methods that make use of numerical methods of analysis. It is demonstrated that the simple traditional methods of analysis are usually adequate for practical purposes, provided that the appropriate soil stiffness parameters have been measured. Thus the main emphasis should be placed on high quality sampling and testing rather than sophisticated analysis. For granular soils, these procedures are not usually possible because of the difficulties of sampling them. Thus for granular soils a completely different approach is usually adopted.

19.2 Total, undrained and consolidation settlement

When a load is applied to a foundation and then kept constant, a time–settlement graph of the form shown in **Figure 19.1** is observed. Some settlement takes place almost immediately during loading. If the soil is saturated and of low permeability, there will be no time for drainage; this is termed the undrained (or immediate) settlement ρ_u.

As the excess pore water pressures (set up during loading) drain away under approximately constant load, consolidation (or time-dependent compression) of the soil takes place until all the excess pore water pressure has been dissipated. The settlement occurring during this process is termed the consolidation settlement and is termed ρ_c as shown in **Figure 19.1**.

For some soils, settlement continues after the excess pore water pressure has been dissipated due to creep of the soil skeleton. This is termed 'secondary' or 'creep' settlement and is termed ρ_s.

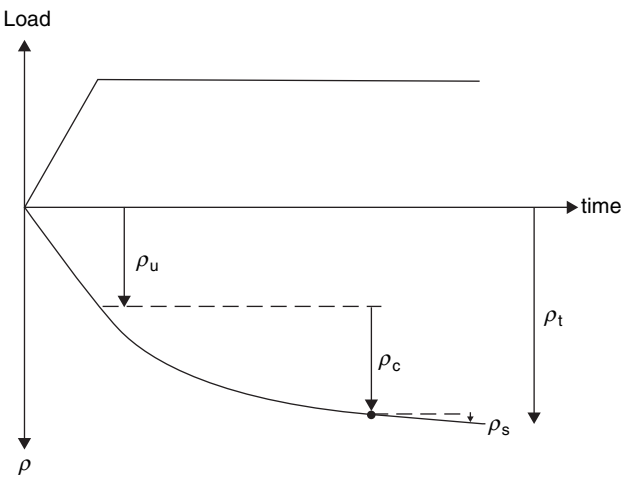


Figure 19.1 Idealised time–settlement curve for a foundation showing the undrained (ρ_u), consolidation (ρ_c) and secondary (ρ_s) components

Thus the total settlement ρ_t at a given time following completion of consolidation settlement is made up of three components:

$$\rho_t = \rho_u + \rho_c + \rho_s. \tag{19.1}$$

For rapidly draining or partly saturated soils, ρ_u and ρ_c may, for all practical purposes, be indistinguishable. Little attention will be devoted to ρ_s as it is not well understood and is frequently, though not invariably, small – see Simons and Menzies (2000) for a more detailed discussion on this topic.

Figure 19.2 shows schematically the ground surface movements that take place owing to (a) undrained settlement and (b)

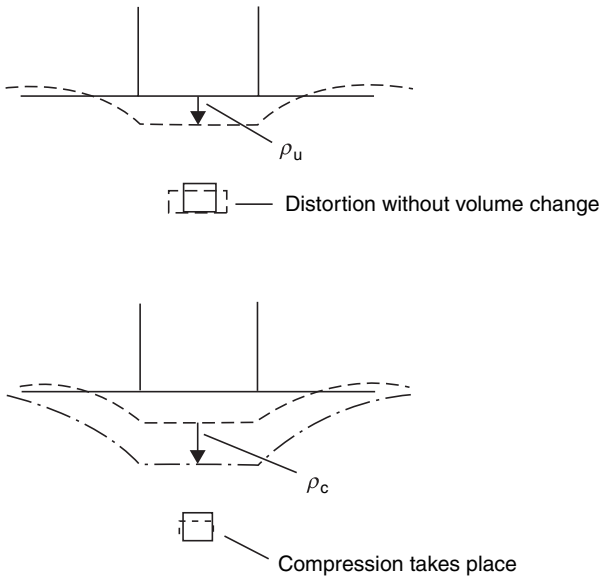


Figure 19.2 Schematic deformations of a soil element beneath a foundation during undrained loading and subsequent consolidation

subsequent consolidation settlement. The associated deformations of an element of soil beneath the centre line of the foundation are shown. It is evident that during undrained settlement only distortion without volume change occurs, whereas during consolidation, volume reduction of the soil skeleton takes place.

19.3 Stress changes beneath loaded areas

To carry out a settlement calculation it is necessary to derive the vertical strains ε_v at various depths beneath the foundation. The vertical compressions of the various layers of thickness Δz are then summed up over the depth of influence such that:

$$\rho = \sum \varepsilon_v \Delta z. \tag{19.2}$$

The calculation of ε_v at any depth requires knowledge of two things: (1) the stiffness or deformability of the soil at that depth, and (2) the stress changes at that depth induced by the loading of the foundation.

The theory of elasticity is invariably used for calculating the stress changes. Most textbooks outline methods of obtaining the vertical stress changes beneath simple loaded areas on an elastic, isotropic homogeneous half-space. For some particularly simple problems other stress changes, such as the principal stress changes or the horizontal stress changes, are given as well. All these results are obtained by integrating Boussinesq's famous solution for a point load on the surface and are often referred to as 'Boussinesq stresses'.

The chart given in **Figure 19.3** is the well-known one for deriving the vertical stress change at depth z beneath the corner of a rectangular uniformly loaded area. Superposition can be used to calculate the vertical stress change at any location

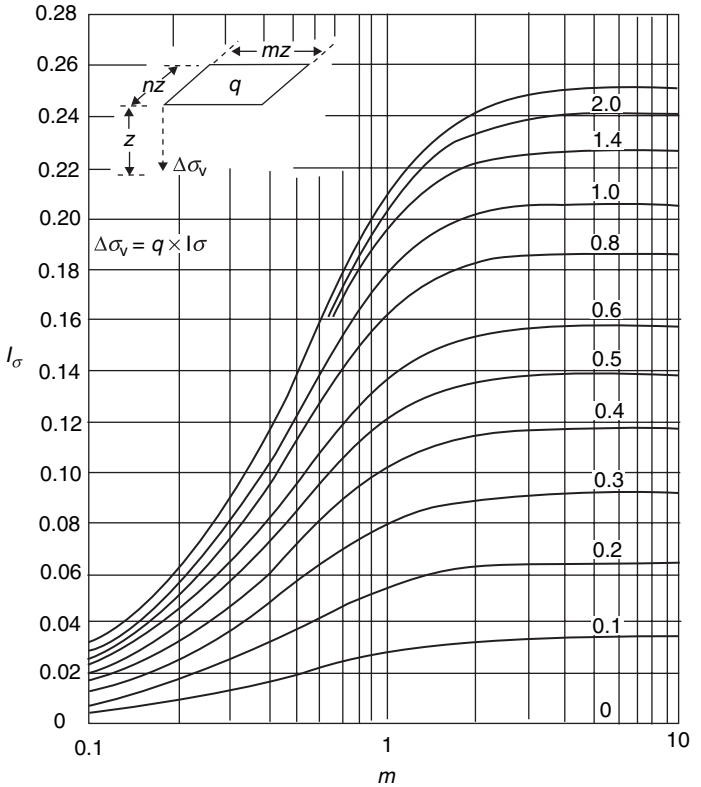


Figure 19.3 Vertical stress under the corner of a rectangular area carrying a uniform pressure

Reproduced from Simons and Menzies (2000); data originally taken from Fadum (1948)

beneath or outside the rectangle, as explained in most textbooks. The expressions for the vertical and horizontal stress changes beneath the centre of a uniformly loaded circular area of radius r are as follows:

$$\Delta\sigma_v = q\left\{1 - \left[\frac{1}{1+(r/z)^2}\right]^{3/2}\right\} \tag{19.3}$$

$$\Delta\sigma_h = \frac{q}{2}\left\{(1+2\nu) - \frac{2(1+\nu)}{[1+(r/z)^2]^{1/2}} + \frac{1}{[1+(r/z)^2]^{3/2}}\right\}, \tag{19.4}$$

where ν is Poisson's ratio. **Figure 19.4** is derived from equation (19.3) and shows the contours of increase in vertical stress Δq_v beneath a uniformly loaded circular area expressed as a proportion of the applied pressure at the surface Δq_s. The contour of $\Delta q_v/\Delta q_s = 0.1$ encloses what is termed the 'bulb of pressure' which is taken as an approximate but useful guide to the zone of influence beneath and around a loaded area. For a circular loaded area, the bulb of pressure is seen to extend to a depth of about twice its diameter. It is important to note that for an elastic homogeneous, isotropic half-space, the changes

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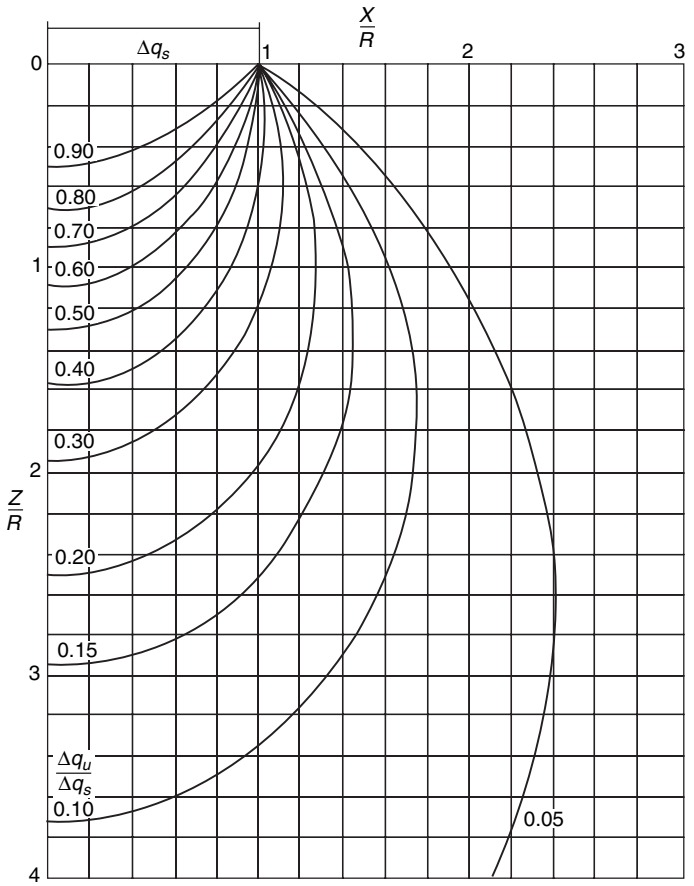


Figure 19.4 Vertical stress changes beneath a uniformly loaded circular area of radius *R*
Reproduced from Lamb and Whitman (1969) © John Wiley & Sons, Inc.

in vertical stress are always independent of Poisson's ratio υ. However for most problems, the other stresses are dependent on it. Poulos and Davis (1974) have assembled a very complete set of elastic solutions for a wide range of problems.

An obvious and important question that is not addressed in most textbooks is: how valid is it to use the Boussinesq theory to predict stress changes in soils beneath foundations? Most soils are patently not elastic, not isotropic and not homogeneous, so at face value the theoretical basis for calculating settlements looks pretty flimsy! As a result of recent analytical and numerical work using more 'soil-like' stress–strain properties, we are in a much better position to answer this question than we were a few years ago. In particular, the effects of nonlinearity, non-homogeneity and anisotropy have been studied.

19.3.1 Nonlinear stress–strain behaviour

A number of theoretical studies have been carried out to assess the influence of nonlinear behaviour on the stress changes beneath uniformly loaded areas. These studies have included elastic-perfectly plastic materials and continuously nonlinear stress–strain behaviour up to failure (see Burland *et al.*, 1977). All these studies have shown that the changes in vertical stress are remarkably similar to those given by the Boussinesq theory. However the other stresses, in particular the horizontal and shear stresses, depart significantly from Boussinesq. **Figure 19.5** shows the stress changes beneath the centre of a uniform circular surface load for an undrained nonlinear plastic material. It can be seen that the vertical stress changes are almost coincident with the Boussinesq values, but that the radial and deviator

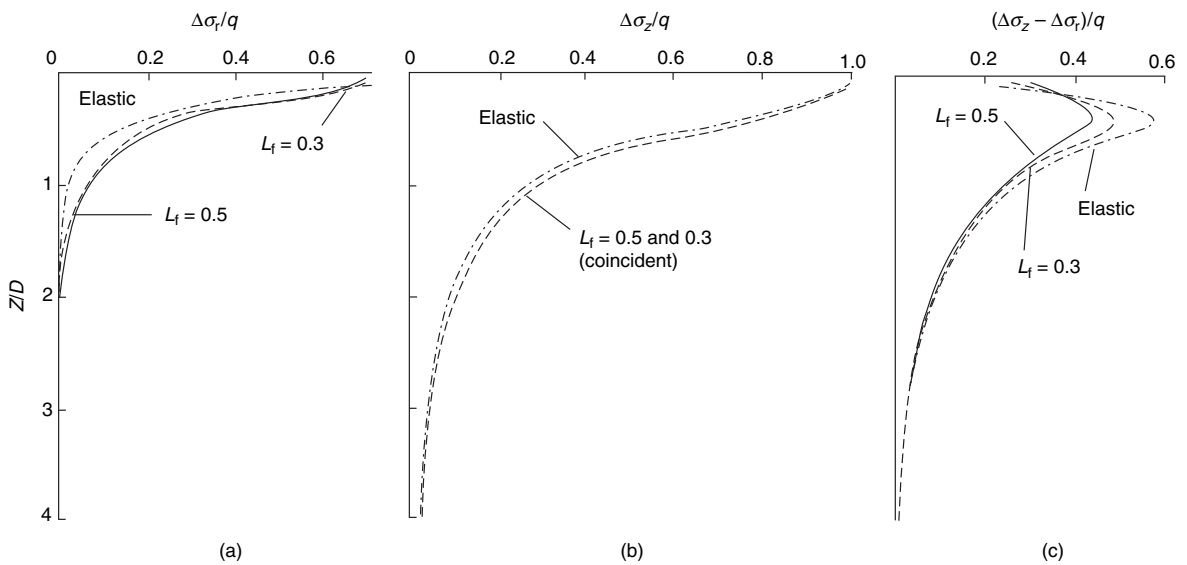


Figure 19.5 Distribution of stress increments beneath the centre of a flexible circular footing for various load factors *L*: (a) radial stress changes; (b) vertical stress changes; (c) deviator stress changes
Reproduced from Jardine *et al.* (1986)

stress changes differ significantly from the Boussinesq values and depend on the stress level.

19.3.2 Non-homogeneity

There is of course an infinite variety of types of non-homogeneity but it is instructive to examine some of the more common ones. One of the most common situations is where stiffness increases with depth, sometimes termed a Gibson soil. Since, for a given soil type, stiffness is approximately proportional to effective confining pressure, this is to be expected. **Figure 19.6** shows the distribution of vertical and horizontal stress changes beneath the centre of a uniform strip load on the surface of an elastic half-space, in which Young's modulus increases linearly with depth from zero at the surface (Gibson and Sills, 1971). The full lines give the Boussineq solution. Remarkably, for Poisson's ratio $\upsilon = 0.5$, the vertical and horizontal stress changes are identical to those given by Boussinesq. For $\upsilon < 0.5$ the vertical stress changes are only slightly larger than those given by Boussinesq, but the horizontal stress changes are much less – being very sensitive to the value of Poisson's ratio.

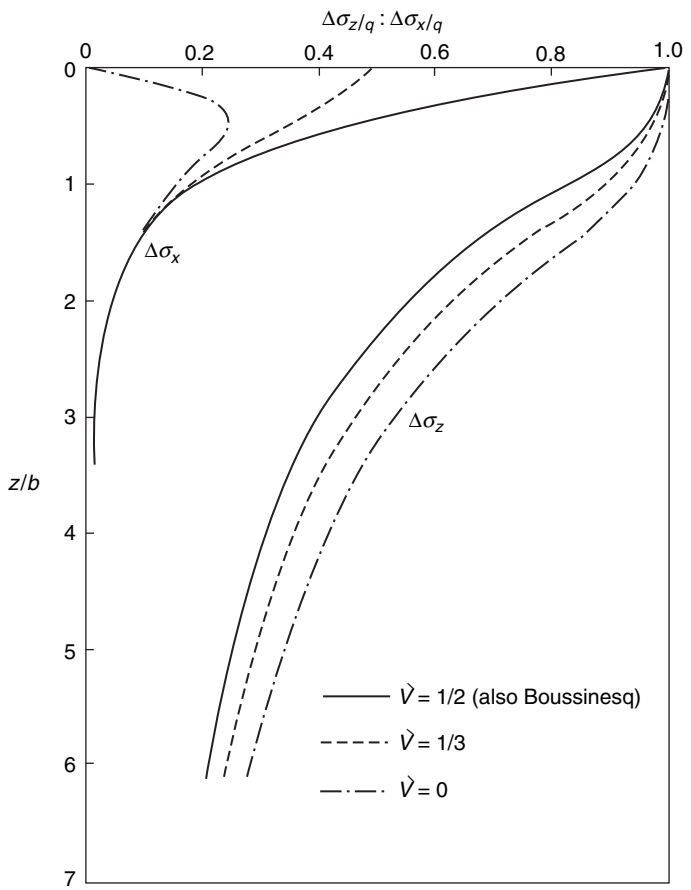


Figure 19.6 Stress changes beneath centre line of a uniform strip load on a Gibson half-space

Reproduced from Gibson and Sills (1971); all rights reserved

Another common form of non-homogeneity is that of a compressible layer of soil overlying rock. **Figure 19.7** shows the distribution of vertical stress change beneath the centre line of a uniform circular load resting on a compressible layer overlying a rigid layer. It can be seen that the vertical stress changes increase only slightly as the depth of the rigid layer decreases. It can be shown that the horizontal stress changes are very significantly affected by the depth of the rigid layer.

Unlike the previous two situations, the presence of a stiff layer overlying a soft layer has a significant effect on the vertical stress changes. **Figure 19.8** shows the vertical stress changes beneath the centre of a uniform circular load resting on a stiff layer overlying a more compressible layer. It can be seen that the presence of a stiff overlying layer significantly reduces the magnitudes of the vertical stress changes in comparison with the Boussinesq stresses. Thus the presence of a stiff overlying layer is very beneficial in reducing the stresses in an underlying softer material.

19.3.3 Anisotropy

In Chapter 14 *Soil as particulate materials* it was demonstrated that deposition under vertical gravity leads to preferred orientation of the particle arrangements, and this leads to inherent anisotropy of the soil skeleton. Hence anisotropy is a common feature of most soils. In Chapter 17 *Strength and deformation behaviour of soils* readers were introduced to the cross-anisotropic porous elastic model of soil in which the horizontal stiffness properties differ from the vertical ones. This model can be fully defined by the five parameters:

E'_v = effective Young's modulus in the vertical direction;

E'_h = effective Young's modulus in the horizontal direction;

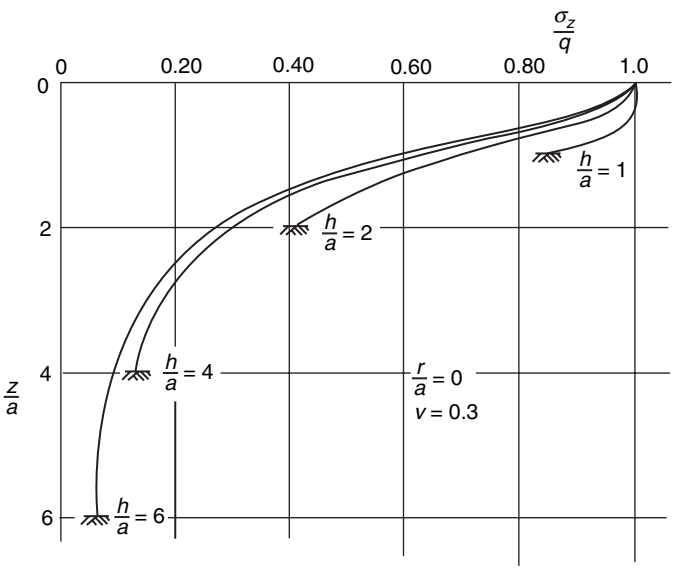


Figure 19.7 Vertical stress changes beneath the centre of a uniform circular load of radius *a* on a compressible layer of various thicknesses *h*

Reproduced from Davis and Poulos (1963); all rights reserved

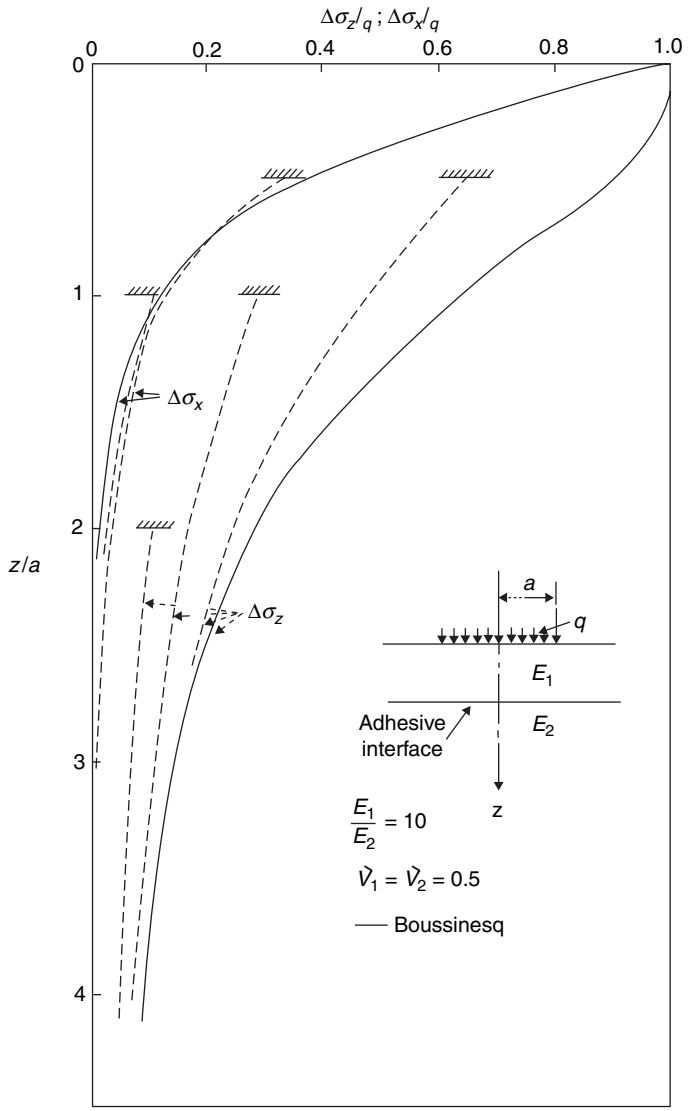


Figure 19.8 Influence of a stiff upper layer of various thicknesses on the vertical and horizontal stress changes beneath the centre of a uniformly loaded circular area
Reproduced from Burland *et al.* (1977)

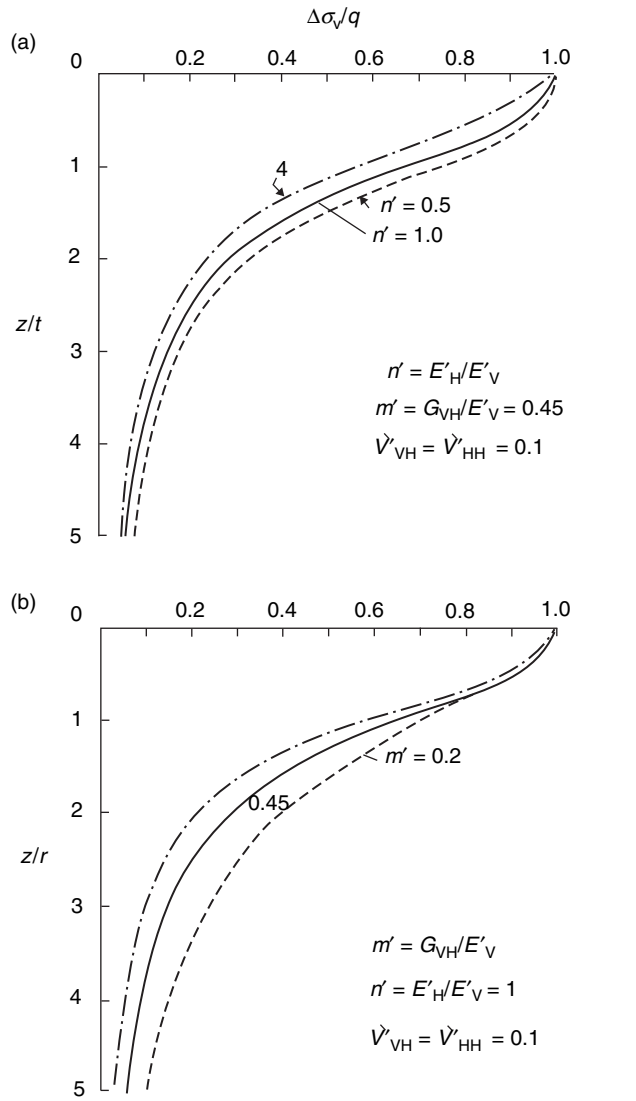


Figure 19.9 Influence of anisotropy on the vertical stress changes beneath the centre of a uniformly loaded circular area: (a) influence of change of E'_H; (b) influence of change of shear modulus G_{VH}

υ'_{vh} = effective Poisson's ratio for the effect of vertical stress change on horizontal strain;

υ'_{hh} = effective Poisson's ratio for the effect of horizontal stress change on orthogonal horizontal strain;

G'_{vh} = shear modulus in the vertical plane.

It is convenient to define the following ratios:

$$E'_h/E'_v = n' \text{ and } G'_{vh}/E'_v = m'.$$

Figure 19.9 shows the vertical stress changes beneath the centre of a uniform circular load on a porous elastic anisotropic soil for (a) varying n', with m, fixed at its equivalent isotropic value [= $0.5(1 + \upsilon'_{vh})$] and (b) varying m', with n, fixed at its isotropic value (=1). It can be seen from **Figure 19.9(a)** that the effect of varying n' has little effect on the vertical stress changes. However varying G'_{vh} (i.e. the value of m') has a much bigger effect.

19.3.4 Conclusions on stress changes

We can draw the following important conclusions from the examples given in this section:

- For many conditions which depart from the ideal of linear, isotropic, homogeneous elasticity, the Boussinesq theory gives very reasonable estimates of the changes in vertical stress beneath areas of known pressure distribution.

- Two exceptions to this are (1) for a stiff layer overlying a softer layer where large errors on the conservative side result from using Boussinesq; and (2) for anisotropic properties, whereas the vertical stress changes are relatively insensitive to the ratio $E'_h/E'_v = n'$ – they are much more sensitive to the ratio $G'_{vh}/E'_v = m'$. It is of interest to note that the static value of G'_{vh} is seldom measured in a routine manner because it is difficult to do so.
- In contrast to the vertical stress changes, the horizontal (and shear) stress changes are very sensitive to all the variables studied. Hence the Boussinesq equations are unlikely to give accurate estimates of these stress changes.

Although only a limited number of examples are given here, these conclusions are widely supported by numerous studies.

19.4 Summary of methods of settlement prediction for clay soils

This section gives a very brief résumé of the most commonly used methods of settlement prediction on soils that can be reasonably sampled and tested to give the deformation properties.

19.4.1 Conventional 1D method

This method was originally proposed by Terzaghi and was originally used for estimating the compression of a layer of clay situated at some depth. It later became used for thick layers of clay extending right to the surface. The procedure is as follows, referring to **Figure 19.10**:

(i) The compressible stratum is divided into a number of layers of thickness Δz (as shown at the bottom of **Figure 19.10**).

(ii) The initial vertical effective stress σ'_{vi} is calculated for the centre of each layer.

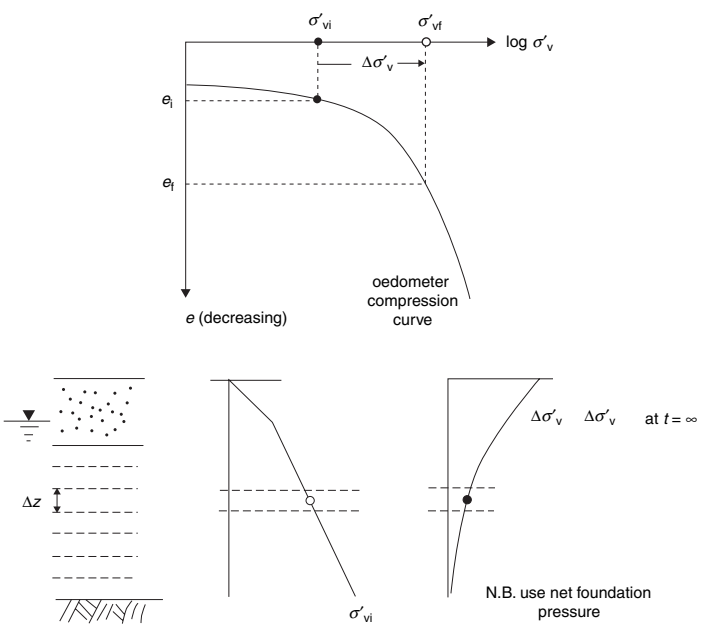


Figure 19.10 Classical one-dimensional method of settlement analysis

(iii) Using Boussinesq stresses, the increase in vertical stress $\Delta\sigma'_v$ is calculated for the centre of each layer. At the end of consolidation, when all excess pore water pressures have been dissipated, the stress increases will be effective stresses. Note that if the foundation is founded in an excavation, it is the *net* increase in vertical effective stress that gives rise to compression.

(iv) The vertical strain ε_v for each layer may be obtained directly from a plot of e versus σ'_v from one-dimensional (oedometer) compression tests (see top of **Figure 19.10**) such that:

$$\varepsilon_v = \frac{e_i - e_f}{1 + e_i} \qquad (= m_v \Delta\sigma'_v), \tag{19.5}$$

where the subscript 'i' refers to the initial value of e and the subscript 'f' refers to the final value of e. If quoted values of m_v are used, it is important to ensure that they have been measured over the appropriate stress range – see Chapter 17 *Strength and deformation behaviour of soils*. It should also be noted that in Equation (17.13) of that chapter, it was shown that m_v is related to the equivalent effective Young's modulus E' and Poisson's ratio υ' by the expression:

$$m_v = \{1 - \frac{2\upsilon'^2}{1 - \upsilon'}\}/E'. \tag{19.6}$$

(v) The compression of each layer = $\varepsilon_v \Delta z$ and, summing over the total thickness of the compressible stratum, the total 1D settlement is given by:

$$\rho_{1XD} = \sum \varepsilon_v \Delta z. \tag{19.7}$$

Note that Terzaghi assumed that ρ_{1D} is equal to the total settlement ρ_t. The accuracy of this assumption will be evaluated later in this chapter.

19.4.2 Skempton and Bjerrum method

In 1955, Skempton, Peck and MacDonald used the 1D method to calculate the total settlement of a number of buildings, but they recognised that the undrained settlement ρ_u could be a significant proportion of the total settlement ρ_t (see equation (19.1)). They used elastic displacement theory, described in section 19.5, to evaluate ρ_u. This led them to conclude that the consolidation settlement ρ_c is given by $\rho_c = \rho_{1XD} - \rho_u$. In the discussion of the above paper, various writers suggested that it was more logical to assume that $\rho_c = \rho_{1XD}$, so that

$$\rho_t = \rho_u + \rho_{1XD}. \tag{19.8}$$

With this background, Skempton and Bjerrum (1957) put forward their seminal method for calculating the consolidation settlement ρ_c. They argued that the consolidation settlement resulted from the dissipation of excess pore pressure Δu. The key assumption was made that consolidation takes place *one-dimensionally* so that the vertical consolidation strain ε_{vc} at a given depth is given by $\varepsilon_{vc} = \Delta u \,.\, m_v$ where Δu can be calculated

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knowing the value of the pore pressure coefficient A (see equation (17.23) in Chapter 17 *Strength and deformation behaviour of soils*), $\Delta\sigma_v$ and $\Delta\sigma_h$. Hence the consolidation settlement ρ_c for the whole layer is obtained by summation:

$$\rho_c = \sum \Delta u . m_v \Delta z. \tag{19.9}$$

Skempton and Bjerrum defined a correction factor μ to be applied to ρ_{1XD} such that:

$$\rho_c = \mu . \rho_{1XD}.$$

Hence the total settlement is given by:

$$\rho_t = \rho_u + \mu\rho_{1XD}. \tag{19.10}$$

Simple elastic Boussinesq stresses were used to evaluate μ for a range of values of A and foundation geometries. The well-known Skempton and Bjerrum chart for values of μ is given in **Figure 19.11**.

It is important to note that the key assumption of the method is that consolidation takes place *one-dimensionally*. For soils which are approximately elastic in their response, this assumption can be significantly in error. Moreover, the values of μ are based on values of $\Delta\sigma_v$ and $\Delta\sigma_h$ obtained from isotropic, homogeneous, semi-infinite elastic theory. We have already seen that values of $\Delta\sigma_h$ obtained in this way are unreliable. Thus, although the method is widely used and taught, caution is needed in applying it.

19.4.3 Stress path method

Although attributed to Lambe (1964), the stress path method was suggested earlier by Skempton (1957) and developed by Davis and Poulos (1963). In essence, the method consists of the following:

(i) Take undisturbed samples from appropriate depths.

(ii) Set up each sample in the triaxial apparatus under the appropriate initial *in situ* effective stresses.

(iii) Without allowing drainage, apply the calculated values of $\Delta\sigma_v$ and $\Delta\sigma_h$. Measure the vertical strain and the excess pore pressure.

(iv) Then allow drainage to occur – if necessary, adjusting the applied stresses to the final calculated values. Measure the vertical strain and volume change.

(v) The measured strains for stages (iii) and (iv) are summed to obtain the undrained settlement and consolidation settlement.

The method appears to be logical but carries with it serious difficulties. The initial vertical and horizontal effective stresses must be known. A high degree of experimental skill

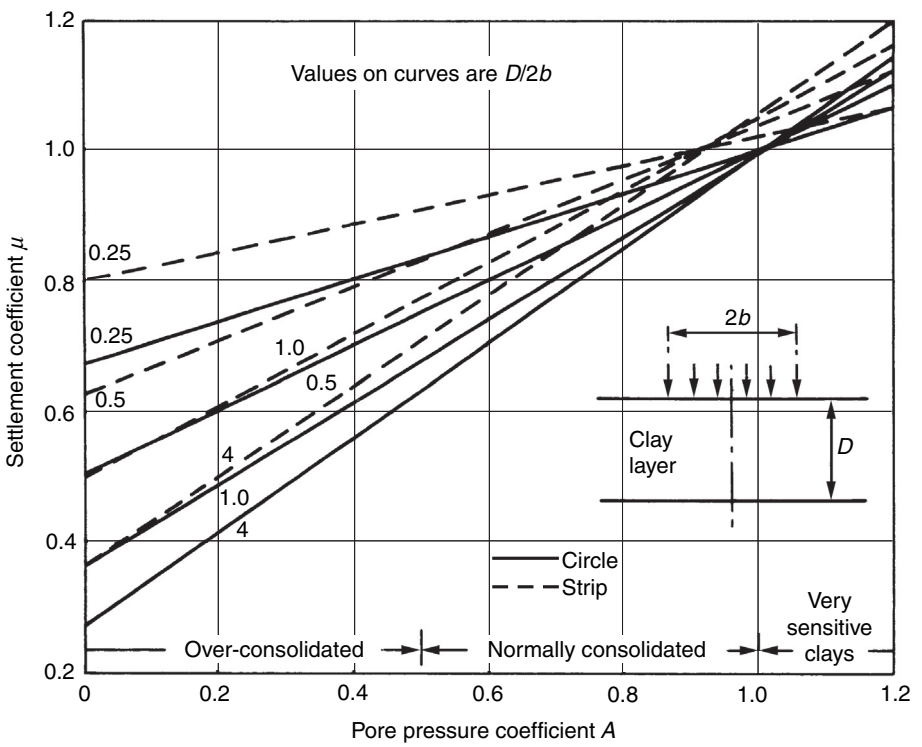


Figure 19.11 Skempton and Bjerrum's settlement coefficient μ for circular and strip foundations
Reproduced from Simons and Menzies (2000); data originally taken from Skempton and Bjerrum (1957)

is required. Isotropic, homogeneous elasticity is used to calculate the stress changes and uncertainties in the accuracy of $\Delta\sigma_h$ therefore exist. The testing is expensive and time-consuming, and a change in the foundation size or loading could render the whole testing program worthless.

A more realistic and flexible approach was suggested by Davis and Poulos (1963). Samples are subjected to appropriate stress levels and stress changes, and the associated equivalent elastic constants E_u, E', and υ' are deduced from the measured strains. The undrained and total settlements can then be obtained by calculating the corresponding equivalent elastic strains and summing them over the appropriate depth:

$$\rho = \sum \varepsilon_v . \Delta z = \sum \frac{1}{E}\{\Delta\sigma_v - \upsilon(\Delta\sigma_x + \Delta\sigma_y)\}.\Delta z. \qquad (19.11)$$

Alternatively, average values of E and υ can be used in conjunction with elastic displacement theory – see section 19.5.

19.4.4 Finite element method

In recent years, powerful numerical methods have become available which can be used for settlement prediction. Most finite element (FE) packages can handle non-homogeneous anisotropic elastic materials. Increasingly more can handle nonlinear stress–strain behaviour of varying complexity. It is perhaps a truism to say that the accuracy of the predictions depends on the quality of the input data. However, engineers are easily seduced by the apparent power and versatility of FE packages and forget, or are not aware of, the many idealisations and assumptions that are built into them.

It is important to examine some of the limitations of routine testing, however well conducted, in providing input data for a numerical analysis:

(i) For triaxial and oedometer tests, only axi-symmetric stress conditions can be applied i.e. σ_a and σ_r.

(ii) The principal stresses and stress changes remain vertical and horizontal.

(iii) Usually only the vertical stiffness E'_v and Poisson's ratio υ'_{vh} are determined.

(iv) Any nonlinear behaviour that is measured is strictly for the highly restricted stress conditions referred to in (ii) above.

(v) The *in situ* stresses and, in particular, stress history of the natural soil, is usually open to considerable speculation.

The above restrictions in the test data should be contrasted with the real situation in the ground:

(i) In real problems, axi-symmetric stress conditions almost never occur.

(ii) Rotations of principal stress directions take place during loading.

(iii) The stiffness both in direct loading and shear is almost certainly anisotropic and related in a complex way to the rotating stresses.

(iv) The previous stress history, and in particular whether the soil is in a loading or unloading cycle, profoundly affects the stress–strain behaviour.

In view of the above, any experimental data used in an FE analysis usually entails profound implicit idealisations.

19.4.5 Comments on methods of settlement prediction

It is evident from the methods presented above that the methods of settlement prediction have progressively become more sophisticated, moving from the classical 1D method through to numerical methods. This has been accompanied by the need for increasingly more soil parameters and more advanced testing. From a purely practical point of view, the engineer is entitled to ask whether such sophistication is necessary and whether greater accuracy is in fact achieved. This question is addressed in section 19.6 by comparing the various approximate methods with known rigorous solutions. First it is necessary to discuss briefly elastic displacement theory.

19.5 Elastic displacement theory

At the beginning of section 19.2, it was pointed out that a settlement calculation involves deriving the vertical strains ε_v at various depths beneath the foundation. For an ideal isotropic elastic material the vertical strain at any depth below a loaded area is given by

$$\varepsilon_v = \frac{1}{E}\left\{\Delta\sigma_v - \upsilon(\Delta\sigma_x + \Delta\sigma_y)\right\}. \qquad (19.12)$$

Note that the two horizontal stress changes and Poisson's ratio are involved.

Elastic displacement theory is based on integrating the vertical strains from $z = 0$ to infinity, such that

$$\rho = \int_0^\infty \varepsilon_v dz\,. \qquad (19.13)$$

Solutions for a wide range of problems are usually expressed in the form

$$\rho = \left\{\frac{qB}{E}(1-\upsilon^2)\right\} I_\rho\,, \qquad (19.14)$$

where:

q = average foundation pressure;

B = a characteristic dimension of the foundation (e.g. breadth);

$I\rho$ = a factor to take account of geometry, depth of founding, depth of rigid layer, etc.

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Examples of some well known solutions are:

Settlement of the centre of a uniform circular load of diameter D:

$$\rho = \frac{qD}{E}(1-\upsilon^2)\,. \tag{19.15}$$

Settlement of the centre of a uniform circular load at great depth:

$$\rho = \frac{qD}{E}(1-\upsilon^2)\frac{1}{2}\,. \tag{19.16}$$

Settlement of a rigid circular load:

$$\rho = \frac{qD}{E}(1-\upsilon^2)\frac{\pi}{4}\,. \tag{19.17}$$

It is of interest to note that the settlement of a rigid circular load given by equation (19.17) is only very slightly less than the average settlement of a uniformly loaded (flexible) circular area (equation (19.15)). Thus, while stiffening a foundation significantly reduces the differential settlements, it has little effect on the average total settlements.

Numerous charts are available for calculating the settlement of various types of loaded area. For example, **Figure 19.12** is the well-known settlement chart for estimating the average undrained settlement of uniformly loaded rectangular areas at depth D with a rigid layer at a depth H beneath foundation level (Christian and Carrier, 1978). These charts are only for Poisson's ratio = 0.5 and are strictly only relevant for estimating undrained settlement.

In section 19.3 we saw that simple elastic theory gives a good estimate of the vertical stress changes for a range of soil-like properties and can therefore be used with considerable confidence. The same cannot generally be said of elastic displacement theory, which must be used with considerable caution. In particular, the calculation of the strains in equation (19.12)

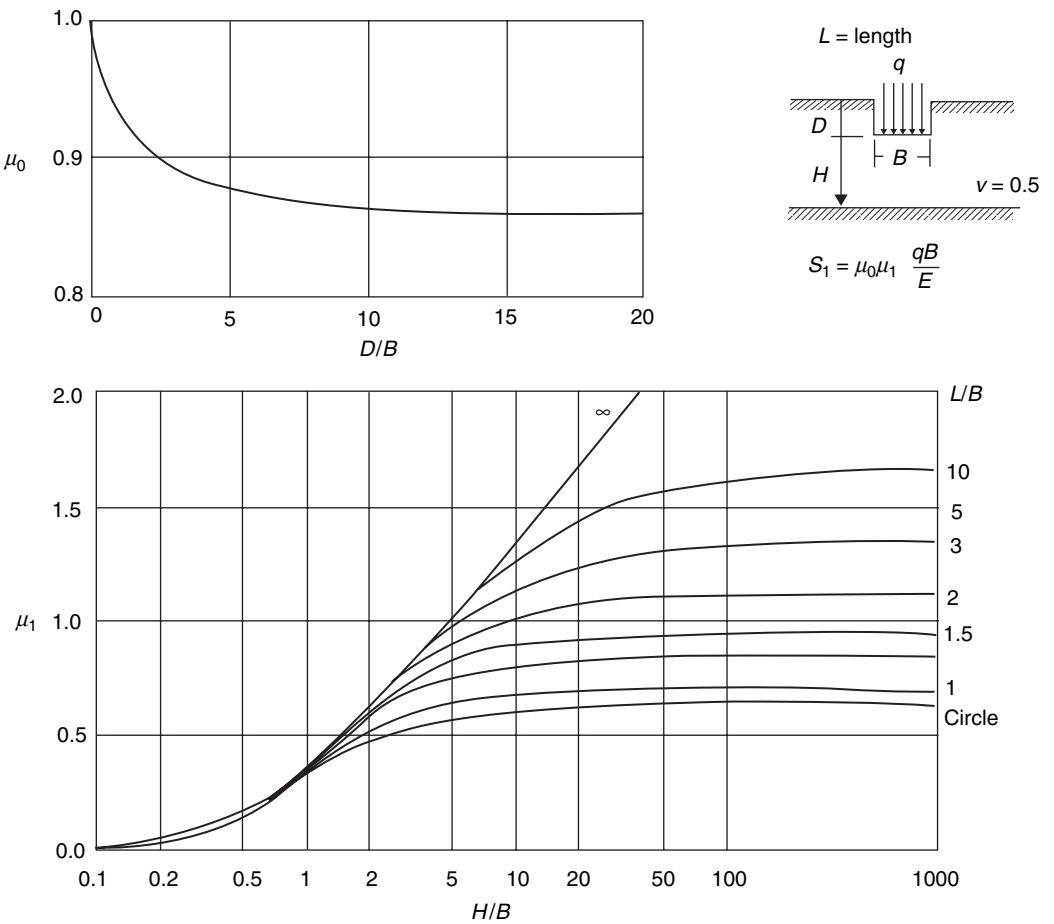


Figure 19.12 Average undrained settlement of uniformly loaded rectangular areas with a foundation depth D and a rigid layer at depth H beneath founding level

Reproduced from Christian and Carrier (1978) © Canadian Science Publishing or its licensors

involves horizontal stress changes which we have shown are very sensitive to many factors. Moreover the integration of the strains in equation (19.13) involves the assumption that the stiffness properties are constant with depth (homogeneity).

When using solutions based on elastic displacement theory, it is no easy matter to make allowance for the influence of such factors as anisotropy, non-homogeneity and nonlinearity of stress–strain behaviour. These influence the stress changes (in particular the horizontal ones) as well as the stiffness. However, when used with discrimination, elastic displacement theory can be very useful, particularly for estimating undrained settlements.

19.6 Theoretical accuracy of settlement predictions

In the past, the difficulty of assessing the accuracy of methods of settlement prediction was that the methods of testing were intimately linked with the analysis. This has meant that it has not been possible to isolate inaccuracies in sampling and testing from the limitations of the analysis.

With the advent of the FE method, it has been possible to obtain rigorous solutions using realistic soil-like constitutive relationships, and compare them with predictions based on the various current approximate methods of settlement prediction. The startling conclusion is reached that there is little to choose between the methods and that in most cases, the simple 1D method gives the best predictions of total settlement ρ_t. Hence all the effort should be devoted to measuring as accurately as possible the straightforward stiffness parameters such as m_v, E'_v and υ'. It is demonstrated that the vertical shear modulus G_{vh} is also a key parameter, but is very seldom determined from static tests.

The above important conclusions are briefly illustrated for a few cases in the following sections. The approach is to compare the rigorous solution with the predictions of the various methods.

19.6.1 Accuracy of the one-dimensional method for homogeneous isotropic elastic material

As explained in Chapter 17 *Strength and deformation behaviour of soils*, a homogeneous porous elastic material is completely defined by the two parameters E' and υ'. It was further shown in equation (17.13) of that chapter, and repeated as equation (19.6) above, that:

$$m_v = \{1 - \frac{2\upsilon'^2}{1-\upsilon'}\}/E' \quad .$$

For m_v constant with depth, equation (19.7) for the 1D settlement is given by:

$$\rho_{1XD} = m_v \int \Delta\sigma'_v dz \qquad (19.18)$$

where $\Delta\sigma'_v$ is obtained from Boussinesq stresses. For a uniform circular load of radius r, the values of $\Delta\sigma'_v$ are given by equation (19.3). It turns out that $\int\Delta\sigma'_v dz$ for this case is particularly simple and is equal to $2qr$. Hence equation (19.18) for the 1D settlement becomes:

$$\rho_{1XD} = 2.q.r.m_v = \frac{q.D}{E'}\left\{1 - \frac{2\upsilon'^2}{1-\upsilon'}\right\}. \qquad (19.19)$$

This is the analytical solution for the 1D settlement of the centre of a uniformly loaded circular load on an isotropic semi-infinite porous elastic material. But we know that the exact total settlement for this problem is given by equation (19.15), so:

$$\rho_{t(exact)} = \frac{qD}{E'}(1-\upsilon'^2) \quad . \qquad (19.20)$$

Hence the ratio of the one-dimensional settlement to the exact settlement, derived from equation (19.16) is:

$$\frac{\rho_{1XD}}{\rho_{t(exact)}} = \frac{1-2\upsilon'}{(1-\upsilon')^2}. \qquad (19.21)$$

Figure 19.13 shows a graph of equation (19.16). It can be seen that when $\upsilon' = 0$, the one-dimensional settlement is exactly equal to the total settlement. As υ' increases, the ratio $\rho_{1XD}/\rho_{t(exact)}$ decreases slowly at first, and then more rapidly until it equals zero when $\upsilon' = 0.5$. Most soils and weak rocks have values of $\upsilon' < 0.25$ over the range of working stress changes. It can therefore be concluded that ρ_{1XD} will usually be within 10% of the total settlement on the low side. If the compressible layer is of limited depth, the 1D settlement will be even closer to the exact total settlement.

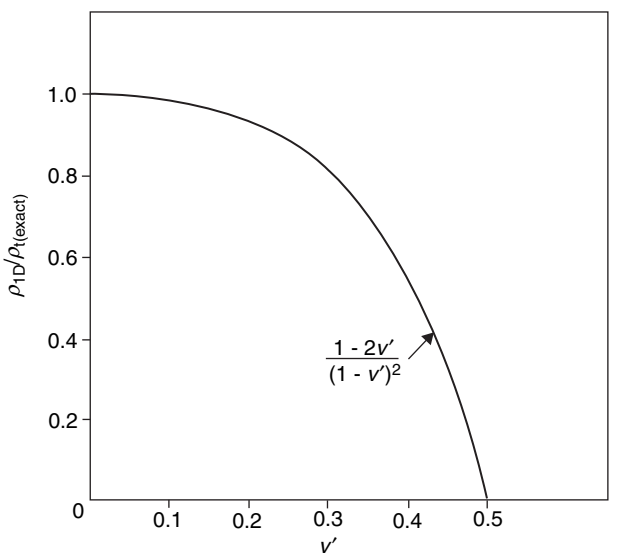


Figure 19.13 The influence of effective Poisson's ratio on the accuracy of the 1D method for a uniform circular load on an isotropic homogeneous elastic half-space

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19.6.2 Accuracy of the one-dimensional method for homogeneous cross-anisotropic elastic material

In section 19.3.3, the five elastic constants that define a cross-anisotropic material were listed. It was pointed out in Chapter 17 *Strength and deformation behaviour of soils* that such a material is generally more representative of a stiff soil or a soft sedimentary rock than an isotropic elastic material. Using the example of a uniform circular load, we now compare the settlements predicted by the four methods described in section 19.4 with the solutions obtained by a rigorous FE analysis (for practical purposes we can assume that these solutions are exact).

It must be remembered that the engineer does not necessarily know that the material is anisotropic. It is assumed that the appropriate routine tests are carried out on vertical samples to measure parameters such as m_v, E', υ', E_u and A (the pore pressure parameter). Moreover, as is normal practice, Boussinesq stress changes are used in carrying out the settlement predictions.

Figure 19.14 shows graphs of $\rho_{(predicted)}/\rho_{t(exact)}$ versus the ratio $E'_h/E'_V = n'$ for the four methods of settlement prediction described in section 19.4. For the purposes of this analysis, the value of $G_{vh}/E'_V = 0.45$ was adopted (the isotropic value) and with $\upsilon'_{vh} = \upsilon'_{hh} = 0.1$. It is found that the solutions are insensitive to small changes in υ'.

The results for the 1D method are shown by the full line in **Figure 19.14**. It can be seen that for $n' < 1.2$, the exact total settlement is slightly underpredicted. As n' increases to 4, the method tends to overpredict slightly, reaching a maximum of about 6%. None of the other three methods give better predictions of the total settlement than the 1D method, despite its simplicity. The greatest discrepancy is given by the Skempton and Bjerrum method which for $n' > 4$ overpredicts the total settlement by 22%. Even the FE method, which makes use of the measured values of E' and υ', overpredicts the total settlement by as much as 20% for $n' > 4$.

19.6.3 Accuracy of the one-dimensional method for cross-anisotropic elastic material – stiffness increasing with depth

As explained in section 19.3.2, increasing stiffness with depth is one of the most common situations in soil mechanics – far more common than uniform stiffness with depth. If cross-anisotropy is also included, we are dealing with a very realistic and reasonably general condition that can be used for evaluating the accuracy of various prediction methods. Using methods identical to those described in section 19.6.2, the four common methods of settlement prediction are compared with exact values obtained using FE analysis. It is found that the one-dimensional method always gives predictions of the total settlement which are within 10% of the exact solution – significantly better than the other methods.

19.6.4 The influence of vertical shear modulus G_{VH}

For all of the above comparisons a value of $G_{VH}/E'_V = 0.45$ was used. This corresponds to the equivalent isotropic value given by $G = E'/2(1 + \nu')$ with $\upsilon' = 0.1$. The vertical shear modulus G_{VH} is an independent variable which is seldom measured other than by dynamic methods. In principle, there seems to be no reason why the value of G_{VH}/E'_V should not vary over quite wide limits. Rigorous FE analyses have been carried out for a uniform circular load varying the value of G_{VH}/E'_V and in **Figure 19.15** the results are compared with the 1D and the Skempton and Bjerrum

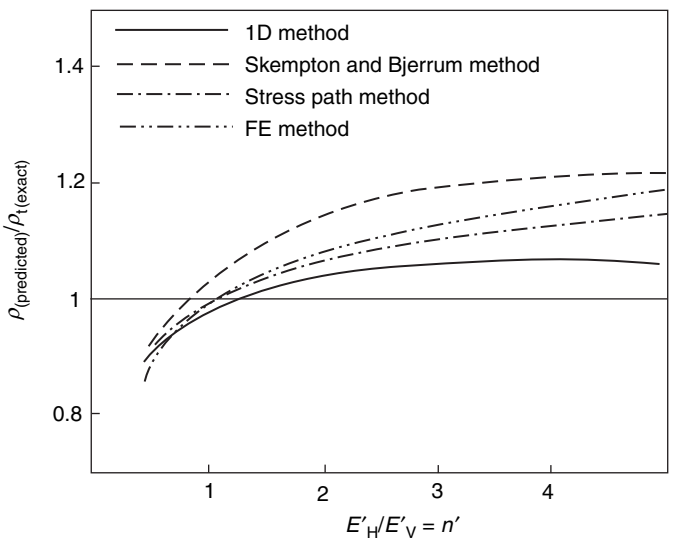


Figure 19.14 The accuracy of four methods of predicting the total settlement of the centre of a uniformly loaded circular area resting on a homogeneous, anisotropic elastic soil. All four methods make use of stiffness parameters obtained from oedometer or triaxial tests

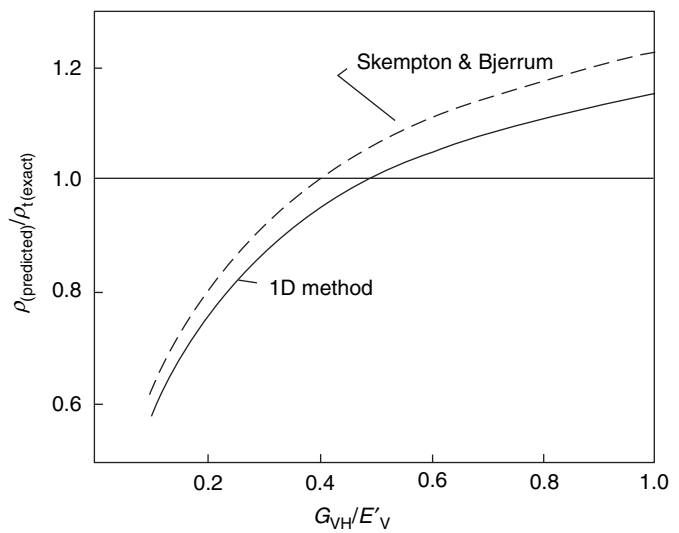


Figure 19.15 Results of rigorous finite element analyses, illustrating the important influence of the vertical shear modulus G_{VH} on settlement

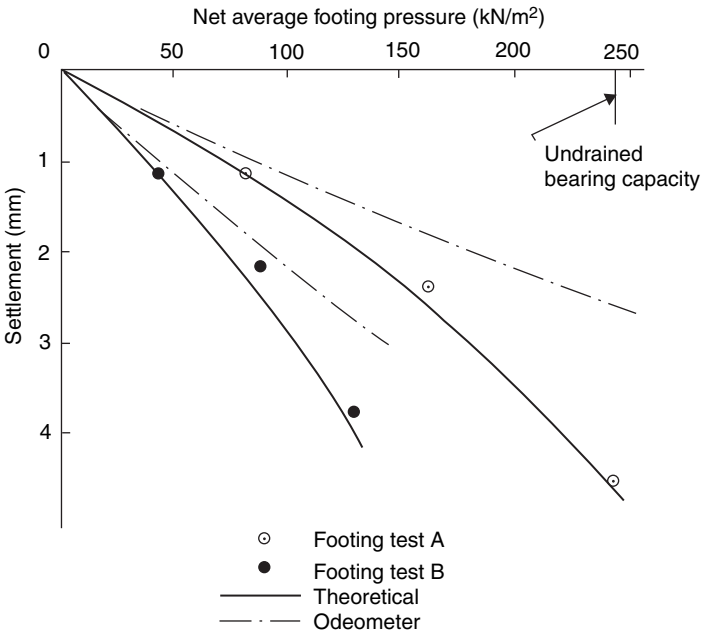


Figure 19.16 Observed consolidation settlements for two model footings on clay compared with predictions based on the 1D method and predictions using the modified Cam Clay model
Reproduced from Burland (1971)

methods, which cover the range given by all four methods. It can be seen that for values of $G_{VH}/E'_V > 0.45$ all of the methods underpredict the settlement whereas for values of $G_{VH}/E'_V < 0.45$, they overpredict. It is important to appreciate that the vertical shear modulus is a key parameter in determining the magnitude of settlement, yet it is seldom measured by static means and traditional methods of settlement prediction take no account of it.

19.6.5 Accuracy of the one-dimensional method for normally consolidated clay

In recent years, considerable developments have been made in developing mathematical models for the stress–strain behaviour of soft normally consolidated clays. A detailed discussion of these developments is outside the scope of this chapter. However, sufficient has been done to demonstrate that, for factors of safety on undrained bearing capacity greater than about three, the measured and theoretical values of the consolidation settlement ρ_c are only slightly larger than the one-dimensional settlement. **Figure 19.16** shows the results of two model footing tests on normally consolidated clays compared with the predictions of the one-dimensional method and the modified Cam Clay model for each model footing test (Burland, 1971). It can be seen that for bearing pressures up to about a third of the undrained bearing capacity, the measured settlements are only slightly larger than the predicted one-dimensional values.

19.7 Undrained settlement

The undrained settlement ρ_u is usually calculated by making use of elastic displacement theory (see section 19.5). Most available charts and formulae are based on homogeneous elastic theory and are therefore of limited value. Butler (1974) developed some useful charts for E_u increasing linearly with depth. A major problem in predicting undrained settlement is the accurate measurement and choice of appropriate values of E_u.

A simple pragmatic empirical approach is to estimate the undrained settlement as a proportion of the total settlement ρ_t. Simons and Som (1970) have reported on the settlement of twelve buildings on overconsolidated clays. They show that ρ_u/ρ_t varies from 0.3 to 0.8 with an average of 0.58. This figure is in agreement with elastic theory and many recent case histories.

For normally consolidated clays, Simons and Som (1970) reviewed nine case histories and show that ρ_u/ρ_t varies from 0.08 to 0.2 with an average of 0.16. Undoubtedly some consolidation would have taken place during construction and ρ_u/ρ_t will usually be less than 0.1.

19.8 Settlement on granular soils

Numerous methods of predicting settlement of foundations on sands and gravels have been published – many more than for clays. The reason lies in the difficulty of obtaining undisturbed samples for the laboratory determination of compressibility under appropriate conditions of stress and stress history. Hence resort has been made to the empirical interpretation of field *in situ* tests, such as standard penetration tests (SPTs) and cone penetration tests (CPTs). This extensive literature has been adequately covered by Sutherland (1974) and Simons and Menzies (2000). The latter authors apply the six most up-to-date methods to a simple illustrative example and show that the predictions differ by a factor of five, even when the representative values of the penetration results are given. Presumably the range would be even wider in practice where the engineer has, in addition, to interpret the penetration data.

The practical significance of the problem was put into perspective by Terzaghi (1956) when he stated that all buildings resting on sand, which were known to him, had settled less than 75 mm, whereas the settlement of buildings on clay foundations quite often exceeded 500 mm!

In view of the unsatisfactory state of the art, and mindful of Terzaghi's assertion, Burland *et al.* (1977) went back to the original databases on which the various empirical methods are based, to see whether a simpler picture emerges which is less dependent on quantitative correlations with erratic penetration tests.

Indeed, a much simpler picture does emerge as shown in **Figure 19.17** where measured settlement per unit pressure (ρ/q) is plotted against foundation breadth B. In each case, the sand is broadly classified as 'loose', 'medium dense' and 'dense', based on average SPT blow counts N. In this correlation no account has been taken of such factors as the water table, depth of loaded area and geometry. Reasonably well defined upper limits for dense, medium dense and loose sands are shown by the full, dotted and chain dotted lines respectively. When assessing the most probable settlement the engineer may

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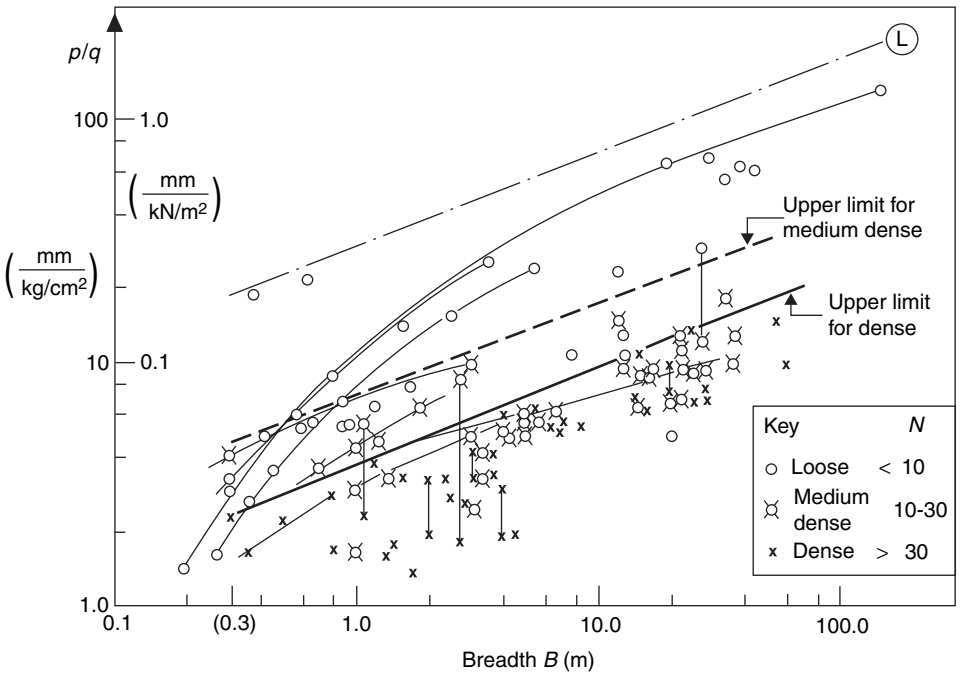


Figure 19.17 Measured settlement of foundations on sand of various relative densities
Reproduced from Burland *et al.* (1977)

elect to work to about 50% of the upper limit values. In most cases the maximum settlement would be unlikely to exceed about 75% of the upper limit. **Figure 19.17** is an example of an empirical approach in which, as far as possible, the measurements of foundation performance are allowed to speak for themselves with a minimum of indirect interpretation; see Chapter 4 *The geotechnical triangle*.

In **Figure 19.17**, the points which are connected by thin full lines are for different sizes of foundation at the same site. If plate loading tests are to be used to extrapolate to larger foundation dimensions, it is clear that the trends are not established at $B = 0.3$ m and that tests with $B = 1$ m are more likely to be successful.

Figure 19.17 is useful for making a preliminary assessment of the probable settlements on granular materials. Subsequent to the publication of this figure, Burland and Burbidge (1985) carried out a detailed statistical analysis of 200 case records of the settlement of foundations on sands and gravels. This resulted in a simple but more refined approach, which leads to an assessment of the likely range of settlements of a given foundation. It was also shown that time-dependent settlements can take place which, after 30 years, can reach 1.5 times the immediate settlements for static loads, and as high as 2.5 times for fluctuating loads. A brief summary of this more refined approach is given by Craig (2004). In concluding this section, it is appropriate to bear in mind the following remarks by Sutherland (1974):

> Before a designer becomes entangled in the details of predicting settlement (in sand) he must satisfy himself whether a real problem actually exists and ascertain what advantages and economies can result from refinements in settlement prediction.

19.9 Summary

- Vertical stress changes beneath loaded areas are insensitive to such factors as nonlinear stress–strain behaviour, anisotropy and increasing stiffness with depth. Therefore the Boussinesq equations can be used to calculate values of $\Delta\sigma_v$ with confidence.
- Two important limitations to the above are where a stiff layer overlies a soft layer and where the vertical shear modulus G_{vh} deviates significantly from the equivalent isotropic value. In the former case, the value of $\Delta\sigma_v$ will be overestimated by using Boussinesq.
- In contrast to the vertical stress changes, the horizontal stress changes are very sensitive to all the variables studied and are therefore likely to depart significantly from the Boussinesq theory.
- Elastic displacement theory is based on very restrictive idealisations and should be used with great caution.
- As methods of settlement prediction have become more complex, increasingly more sophisticated types of testing are required. The limitations and cost of laboratory testing must be recognised.
- A careful comparison between exact solutions and four common prediction methods covering a wide range of properties, shows that the simple classical 1D method is at least as good as any other method for calculating the total settlement ρ_t. For normally consolidated clays it appears that the 1D method gives a good estimate of the consolidation settlement ρ_c.

- For overconsolidated clays $\rho_u \approx 0.5\,\rho_t$ and $\rho_t \approx \rho_{1xd}$.
- For normally consolidated clays $\rho_u \approx 0.1\,\rho_t$ and $\rho_t \approx 1.1\,\rho_{1XD}$.
- A most important conclusion is that most effort must be directed towards the measurement of simple properties such as m_v and E'_v and their variation with depth. Errors in these are much more significant than differences in analytical methods.
- The settlement of foundations on granular soils will usually be small and much of it takes place during construction. Numerous empirical methods have been proposed for predicting the settlement of foundations on granular materials which lead to a wide range of estimates. A straightforward analysis of the measured settlements of over 200 records has led to a remarkably simple approach relating settlement to bearing pressure, breadth and average SPT blow count or cone resistance over the depth of influence.

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All chapters within Sections 1 Context and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

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Chapter 20
Earth pressure theory

William Powrie University of Southampton, UK

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This chapter describes the classical calculation of limiting lateral earth pressures for use in the analysis of retaining walls. The concept of simple active and passive zones of soil at failure is developed for total stress and effective stress failure criteria, for adhesionless and frictionless walls. The effects of dry and flooded tension cracks for the total stress (undrained) condition are considered. The way in which the simple active and passive zones can be modified to take account of the effects of wall adhesion or friction is described, and finally the possible in-service stresses acting on a retaining wall are discussed.

20.1 Introduction

The need to be able to calculate lateral earth pressures arises particularly in the design of vertical or near-vertical retaining walls. Owing to the difficulties of quantifying the detailed stress–strain behaviour of a soil, the approach to the calculation of lateral earth pressures in geotechnical engineering has traditionally been based on the concepts of plasticity. By considering the stress states of soil elements on the verge of failure, it is possible to deduce limits between which the lateral earth pressures must lie.

20.2 Simple active and passive limits

20.2.1 Mohr circle of stress

A state of plane stress within a soil (or indeed any solid material) can be characterised by means of the Mohr circle of stress, plotted on a diagram with axes representing the normal stress σ (in the x-direction) and the shear stress τ (in the y-direction). The Mohr circle shows how the combination of τ and σ on a given line within the plane varies with the orientation of that line. The Mohr circle is in effect a locus defining all possible combinations of τ and σ, acting on lines at different orientations, that make up a given state of stress within a plane. In the current context, the plane is usually the cross-sectional plane of a long retaining wall. The circle is symmetrical about the σ-axis: the two points at which it crosses the σ-axis represent *principal directions*, on which the shear stress τ is zero. The corresponding normal stresses are termed the *major* (greater) and *minor* (lesser) *principal stresses*. The stress state on a line at an angle θ anticlockwise from the line on which the major principal stress σ_1 acts is given by the point on the circle whose radius subtends an angle 2θ at the centre of the circle with the radius through the major principal stress point. This is illustrated in **Figure 20.1**. The angle 2θ between the radii corresponding to the major and the minor principal stresses is 180°, which means that in reality their

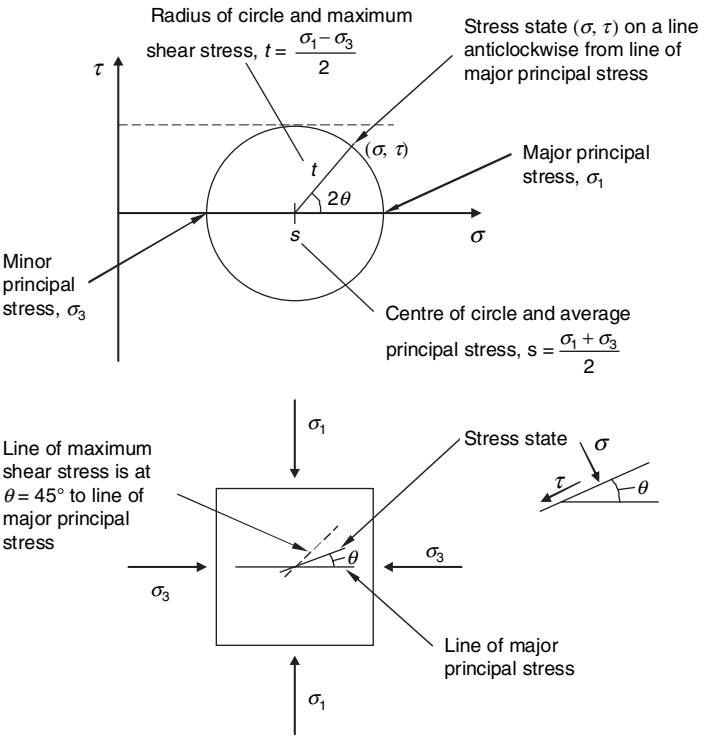


Figure 20.1 Mohr circle representation of the stress state within a principal plane

directions (or the directions of the lines on which they act) are at 90°.

In soils, the Mohr circle may be plotted in terms of either total stresses (σ, τ) or effective stresses (σ', τ), depending on the form of analysis being used.

20.2.2 Total stress failure criterion

If the soil is in a state of limiting equilibrium, there will be some orientation within the plane for which the combination

of stresses (σ, τ) or (σ', τ) lies on the line defining states of stress at failure, or the failure envelope. In soils, two alternative failure criteria or failure envelopes are used; the first is discussed here, the second in section 20.2.3 *Effective stress failure criterion*. In terms of total stresses, the first failure criterion specifies the maximum possible shear stress anywhere within the plane, τ_{max}, in terms of the undrained shear strength of the soil, c_u. This applies only to clay soils sheared rapidly to failure. The undrained shear strength c_u is not a soil property, but depends on the water content of the clay as sheared. The undrained shear strength failure criterion is written as

$$\tau_{max} = c_u \tag{20.1}$$

A Mohr circle of total stress which touches the undrained shear strength failure envelope is shown in **Figure 20.2**. Consideration of the geometry of the Mohr circle shows that the difference between the major and minor principal total stresses, $\sigma_1 - \sigma_3$, is equal to twice the radius of the circle, and the radius is equal to c_u. Thus at failure,

$$\sigma_1 - \sigma_3 = 2c_u \tag{20.2}$$

Assuming for the moment that the principal total stress directions are vertical and horizontal, there are two ways in which the soil could fail. In the first, the major principal total stress is vertical, and the minor principal total stress is horizontal. This could correspond to the withdrawal of lateral support at constant vertical stress until failure is reached. Such conditions might occur behind an embedded retaining wall, and are known as *active*. In active conditions, the horizontal total stress has fallen to its minimum possible value for a given vertical total stress. If the horizontal total stress were reduced any further (with the vertical total stress remaining constant), the Mohr circle of stress would expand beyond the failure envelope – which is not permissible. In active conditions, the (minimum) horizontal total stress is given by

$$\sigma_{h,min} = \sigma_v - 2c_u \tag{20.3}$$

In the second mode of failure, the major principal total stress is horizontal, and the minor principal total stress is vertical. This could correspond to an increase in lateral stress and a constant or possibly reducing vertical stress, as might occur in the soil in front of an embedded retaining wall. These conditions ($\sigma_h > \sigma_v$) are known as *passive*. In passive conditions, the horizontal total stress has increased to its maximum possible value for a given vertical total stress. If the horizontal total stress were increased any further, the Mohr circle of stress would expand beyond the failure envelope, which is again not permissible. In passive conditions, the (maximum) horizontal total stress is given by

$$\sigma_{h,max} = \sigma_v + 2c_u \tag{20.4}$$

As discussed in Chapter 15 *Groundwater profiles and effective stresses*, the vertical total stress is usually taken as $\int \gamma dz$ (= γz in a uniform soil), plus the effect of any surface surcharge q.

When calculating active earth pressures behind a retaining wall using equation (20.3), there are two important provisos. First, negative (tensile) stresses are not permitted, so a no-tension cut-off at $\sigma_h = 0$ comes into play where the uncritical application of equation (20.3) would otherwise give a negative value for $\sigma_{h,min}$ – equivalent to the development of a tension crack penetrating to a depth z_{tc} given by

$$\sigma_{h,min} = (q + \gamma z_{tc} - 2c_u) = 0$$

or

$$z_{tc} = \frac{(2c_u - q)}{\gamma} \tag{20.5}$$

which, in the absence of a surface surcharge, reduces to the more familiar $z_{tc} = 2c_u/\gamma$.

The second proviso is that if there is any possibility that the tension crack could flood, the total lateral stress acting on the soil in the active zone becomes equal to the hydrostatic pressure of water. A flooded tension crack may remain open to a greater depth, z_{ftc}, given by

$$\sigma_{h,min} = (q + \gamma z_{ftc} - 2c_u) = \gamma_w z_{ftc}$$

or

$$z_{ftc} = \frac{(2c_u - q)}{(\gamma - \gamma_w)} \tag{20.6}$$

where γ_w is the unit weight of water. As $\gamma_w \approx 0.5\gamma$, if $q = 0$ the depth to which a flooded tension crack may remain open

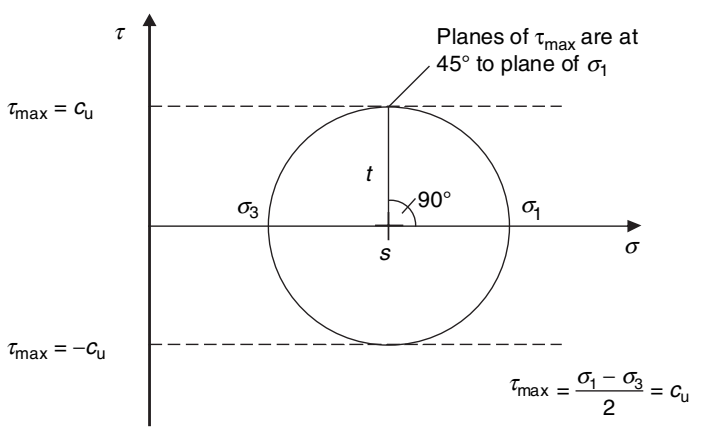


Figure 20.2 Mohr circle of total stress at failure for a clay soil obeying the undrained shear strength failure criterion, $\tau_{max} = c_u$

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(and, more importantly, hydrostatic pressures are exerted on the retaining wall), is $z_{ftc} \approx 4c_u/\gamma$. The tendency of the undrained shear strength c_u to increase with depth means that the depth z_{ftc} could well be more than twice the depth to which a dry tension crack can remain open.

20.2.3 Effective stress failure criterion

The second and more fundamental failure criterion is in terms of effective stresses, and specifies the maximum possible ratio of shear to normal effective stress anywhere within the plane, $(\tau/\sigma')_{max}$, in terms of the tangent of the effective angle of friction shearing resistance or friction of the soil, ϕ'. This applies to all soils at any time, but the pore water pressures must be known in order to calculate the normal effective stress. The critical state effective angle of shearing resistance or friction is a soil property, while the peak effective angle of shearing resistance is not. This is because the latter depends on the potential of the soil to dilate, which in turn depends on a combination of the relative density and the stress state, and must therefore be expected to vary with depth. The effective stress failure criterion is written as

$$\left(\frac{\tau}{\sigma'}\right)_{max} = \tan\phi' \tag{20.7}$$

A Mohr circle of effective stress which touches the effective stress failure envelope is shown in **Figure 20.3**. The radius t of the Mohr circle is equal to half the difference between the major and minor principal effective stresses:

$$t = \frac{(\sigma'_1 - \sigma'_3)}{2}$$

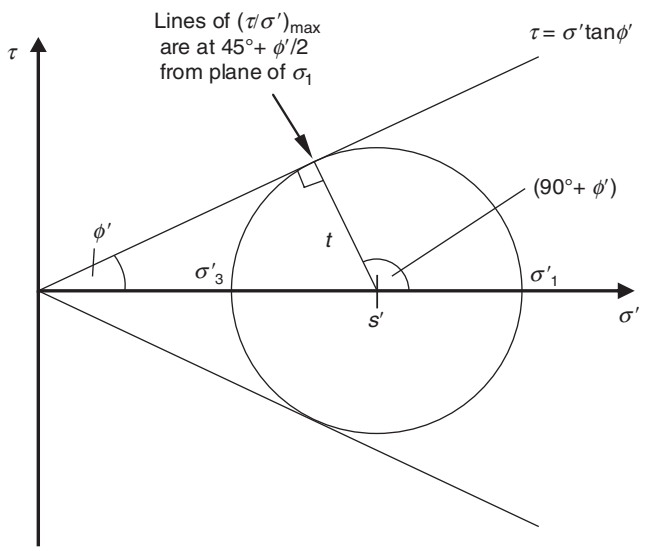


Figure 20.3 Mohr circle of effective stress at failure for a soil obeying the effective stress (frictional) failure criterion $(\tau/\sigma')_{max} = \tan\phi'$

while the distance of its centre from the origin, s', is given by half the sum of the principal effective stresses:

$$s' = \frac{(\sigma'_1 + \sigma'_3)}{2}$$

Further consideration of the Mohr circle geometry shows that $t = s' \sin\phi'$. Substitution of the expressions for t and s' in terms of the major and minor principal effective stresses σ'_1 and σ'_3 into this equation, followed by a little manipulation, gives the ratio of the major and minor principal total stresses σ'_1/σ'_3 at failure, as

$$\frac{\sigma'_1}{\sigma'_3} = \frac{(1+\sin\phi')}{(1-\sin\phi')} \tag{20.8}$$

Assuming again that the principal effective stress directions are vertical and horizontal, there are two ways in which the soil could fail. The first is again active, corresponding potentially to conditions behind a retaining wall with the major principal effective stress vertical and the minor principal effective stress horizontal. In active conditions, the (minimum) horizontal effective stress is given by

$$\sigma'_{h,min} = \frac{(1-\sin\phi')}{(1+\sin\phi')}\sigma'_v = K_a\sigma'_v \tag{20.9}$$

where K_a is the well-known active earth pressure coefficient. From equation (20.9), $K_a = (1 - \sin\phi')/(1 + \sin\phi')$ – this is sometimes written as the trigonometric identity $\tan^2(45° - \phi')$.

The second failure mode is again passive, corresponding potentially to conditions in front of an embedded retaining wall, with the major principal total stress horizontal and the minor principal total stress vertical. In passive conditions, the (maximum) horizontal total stress is given by

$$\sigma'_{h,max} = \frac{(1+\sin\phi')}{(1-\sin\phi')}\sigma'_v = K_p\sigma'_v \tag{20.10}$$

where K_p is the passive earth pressure coefficient. From equation (20.10), $K_p = (1 + \sin\phi')/(1 - \sin\phi')$ – this is sometimes written as $\tan^2(45° + \phi')$.

As discussed in Chapter 15 *Groundwater profiles and effective stresses*, the vertical effective stress is usually taken as $\int\gamma\,dz - u$ ($= \gamma z - u$ in a uniform soil), plus the effect of any surface surcharge q.

Note that in the case of the total stress (undrained shear strength) failure criterion, a difference between the principal stresses is calculated, whereas the use of the effective stress (frictional) failure criterion leads to a ratio of principal effective stresses.

Of course, real walls are not frictionless, and the presence of a shear stress on the back or front of a retaining wall will invalidate the assumption that the principal stress directions are vertical and horizontal, at least adjacent to the retaining wall. This usually works to the designer's advantage in that wall friction or adhesion, provided it is in the right direction,

reduces the active stresses and increases the passive stresses. The calculation of earth pressures taking wall friction into account is discussed in section 20.3.

20.3 Effects of wall friction (adhesion)

It is relatively straightforward to calculate revised earth pressure parameters that take into account a known degree of soil/wall friction (in the case of an effective stress analysis) or undrained soil/wall adhesion (in the case of a total stress analysis). This is done by considering the effect on the average effective or total stress (s' or s) of the rotation in the direction of principal stresses that must occur between (i) a zone of soil near the free surface (within which the principal stresses are horizontal and vertical, as assumed in section 20.2 for the whole of the soil behind or in front of the retaining wall), and (ii) the soil adjacent to the wall (in which the presence of a shear stress on the vertical surface of the wall means that this cannot be a principal direction).

Details of this calculation may be found in Powrie (2004). In practice, it is usual to use the resulting general formulae, charts or tables based upon them, rather than deriving the required result from first principles every time. The key decisions facing the designer then concern the magnitude of the soil/wall adhesion or friction (usually expressed as a proportion of the design soil strength, c_u or ϕ'), and the direction in which the adhesion or friction acts.

A discussion on soil/wall friction is given in Powrie (2004). Eurocode 7 (EC7) (BSI, 1995) gives limiting values of soil/wall friction δ, where $\delta = \phi'_{crit}$ for rough (cast *in situ*) concrete, and $\delta = {}^2/_3\,\phi'_{crit}$ for smooth (precast) concrete or sheet piling supporting sand and gravel. EC7 (BSI, 1995) is silent with regard to values of soil/wall adhesion in an undrained analysis, but Gaba *et al.* (2003) recommend a maximum value of half the design undrained shear strength.

If the soil behind a retaining wall settles relative to the wall and the soil in front heaves, then the directions of soil/wall adhesion or friction act so as to reduce the active earth pressures and increase the passive. Both of these tend to enhance the stability of the wall. However, there are circumstances in which these conventional directions of relative soil/wall movement (and hence adhesion or friction) could be reversed. For example, if the wall carries a vertical load – such as the perimeter wall for a building basement – then the wall may settle relative to the retained soil and the direction of active side adhesion or friction will be reversed. Dewatering inside an excavation could cause relative settlement of the soil in front of the wall, but this is rather less likely.

For undrained conditions, a stress analysis taking into account the effects of a favourable soil/wall adhesion τ_w gives the following expressions for horizontal components of (i) the active stresses:

$$\sigma_{h,min} = \sigma_v - \tau_u\left(1 + \Delta + \cos\Delta\right) \tag{20.11}$$

and (ii), the passive stresses:

$$\sigma_{h,max} = \sigma_v + \tau_u\left(1 + \Delta + \cos\Delta\right) \tag{20.12}$$

where

$$\sin\Delta = \tau_w/\tau_u.$$

If $\tau_w = 0.5\tau_u$, the term in brackets in equations (20.11) and (20.12) becomes numerically equal to 2.39, compared with 2 in the case of the frictionless wall in section 20.2. The same comments regarding the possibility and depth of a dry or flooded tension crack behind the wall apply as in section 20.2. However, there is a minor intellectual complication concerning the depth below the bottom of the tension crack at which soil/wall adhesion starts to be mobilised.

The active and passive earth pressure coefficients, defined now as the horizontal component of the effective stress on the wall at depth z, $\sigma'_{h(z)}$, divided by the notional vertical effective stress at the same level, $(q + \int\gamma\,dz - u)$ where q is any surface surcharge, calculated in an effective stress analysis with an angle of soil/wall friction δ acting favourably are

$$K_a = \frac{\sigma'_{h(z)}}{\left(q + \int\gamma\,dz - u\right)} = \frac{\left[1 - \sin\varphi'\cos\left(\Delta - \delta\right)\right]}{\left[1 + \sin\varphi'\right]} e^{-\left[(\Delta-\delta)\tan\varphi'\right]} \tag{20.13}$$

and

$$K_p = \frac{\sigma'_{h(z)}}{\left(q + \int\gamma\,dz - u\right)} = \frac{\left[1 + \sin\varphi'\cos\left(\Delta + \delta\right)\right]}{\left[1 - \sin\varphi'\right]} e^{\left[(\Delta+\delta)\tan\varphi'\right]} \tag{20.14}$$

respectively, where $\sin\Delta = \sin\delta/\sin\varphi$.

Values of K_a and K_p for various effective friction angles ϕ' and relative soil/wall friction δ/ϕ' are given in **Tables 20.1** and **20.2** respectively. These, together with equations (20.13) and (20.14), are consistent with the equations and charts given in EC7 (BSI, 1995) and Gaba *et al.* (2003).

So far, no judgement has been made as to the value of soil strength used. If conditions at collapse are under investigation, then the soil strength used would be the best estimate of the soil and interface strengths relevant to that limit state. As discussed by Powrie (1996), for embedded retaining walls these may be the critical state soil strength, ϕ'_{crit}, with full wall friction, $\delta = \phi'_{crit}$. In design, an appropriate factor of safety would need to be applied to the soil and interface strengths, together with other measures including an unexpected surcharge on the retained side, an unexpected overdig in front of the wall, and the most onerous foreseeable pore water pressure conditions which might well (in the case of an undrained analysis) include a flooded tension crack between the wall and the retained soil.

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The same techniques can also be used to calculate earth pressure differences and coefficients in cases where the backfill slopes, and/or the back of the wall is battered (see, for example, Powrie, 2004).

20.4 In-service conditions

Modern design codes including EC7 (BSI, 1995) generally require the designer of a retaining wall to guard against failure in the ground by carrying out an ultimate limit state (ULS) calculation with the actual soil strength reduced by a factor of safety or a strength mobilisation factor. (Certain other provisions also apply, such as an allowance for an unexpected surcharge load on the retained side, an unexpected overdig in front of the wall, and the most onerous foreseeable groundwater conditions.) The factored or 'design' strength, being lower than the estimated actual strength of the soil, will result

ϕ' (degrees)	K_a with $\delta = 0$	K_a with $\delta = \phi'/2$	K_a with $\delta = 2\phi'/3$	K_a with $\tan\delta = 0.75 \times \tan\phi'$	K_a with $\tan\delta = \phi'$
12	0.6558	0.6133	0.6038	0.5999	0.5931
13	0.6327	0.5892	0.5794	0.5734	0.5683
14	0.6104	0.5660	0.5560	0.5519	0.5446
15	0.5888	0.5437	0.5336	0.5293	0.5219
16	0.5678	0.5223	0.5120	0.5076	0.5002
17	0.5475	0.5017	0.4914	0.4869	0.4793
18	0.5279	0.4894	0.4715	0.4669	0.4593
19	0.5088	0.4629	0.4525	0.4478	0.4402
20	0.4903	0.4446	0.4341	0.4294	0.4218
21	0.4724	0.4269	0.4165	0.4117	0.4041
22	0.4550	0.4100	0.3996	0.3947	0.3872
23	0.4381	0.3956	0.3833	0.3784	0.3709
24	0.4217	0.3778	0.3676	0.3627	0.3552
25	0.4059	0.3626	0.3380	0.3330	0.3257
26	0.3905	0.3480	0.3380	0.3330	0.3257
27	0.3755	0.3339	0.3240	0.3189	0.3118
28	0.3610	0.3202	0.3105	0.3054	0.2984
29	0.3470	0.3071	0.2976	0.2924	0.2855
30	0.3333	0.2944	0.2851	0.2799	0.2731
31	0.3201	0.2822	0.2730	0.2678	0.2612
32	0.3073	0.2704	0.2614	0.2562	0.2497
33	0.2948	0.2590	0.2502	0.2450	0.2387
34	0.2827	0.2479	0.2394	0.2342	0.2280
35	0.2710	0.2373	0.2289	0.2238	0.2177
36	0.2596	0.2270	0.2189	0.2137	0.2078
37	0.2486	0.2171	0.2092	0.2040	0.1983
38	0.2379	0.2075	0.1998	0.1947	0.1891
39	0.2275	0.1983	0.1908	0.1856	0.1803
40	0.2174	0.1893	0.1820	0.1770	0.1718

Table 20.1 Active earth pressure coefficients K_a consistent with EC7 (BSI, 1995), calculated using equation (20.13) for various values of effective friction angle ϕ' and relative soil/wall friction δ/ϕ'
Data taken from Powrie (2004)

ϕ' (degrees)	K_p with $\delta = 0$	K_p with $\delta = \phi'/2$	K_p with $\delta = 2\phi'/3$	K_p with $\tan\delta = 0.75 \times \tan\phi'$	K_p with $\tan\delta = \phi'$
12	1.525	1.6861	1.724	1.739	1.763
13	1.580	1.7657	1.809	1.826	1.855
14	1.638	1.8500	1.900	1.920	1.953
15	1.698	1.9393	1.996	2.020	2.057
16	1.761	2.0341	2.099	2.126	2.168
17	1.826	2.1347	2.209	2.240	2.287
18	1.894	2.2417	2.326	2.361	2.415
19	1.965	2.3556	2.451	2.492	2.552
20	2.040	2.4770	2.584	2.631	2.699
21	2.117	2.6066	2.728	2.782	2.857
22	2.198	2.7449	2.881	2.943	3.028
23	2.283	2.8930	3.047	3.117	3.212
24	2.371	3.0515	3.225	3.305	3.411
25	2.464	3.2215	3.416	3.509	3.627
26	2.561	3.4042	3.623	3.729	3.861
27	2.663	3.6006	3.847	3.969	4.116
28	2.770	3.8123	4.090	4.229	1.393
29	2.882	4.0407	4.353	4.512	4.695
30	3.000	4.2877	4.639	4.822	5.026
31	3.124	4.5550	4.951	5.162	5.389
32	3.255	4.845	5.291	5.534	5.788
33	3.392	5.160	5.664	5.944	6.227
34	3.537	5.504	6.072	6.395	6.712
35	3.690	5.879	6.522	6.895	7.250
36	3.852	6.289	7.017	7.449	7.847
37	4.023	6.738	7.564	8.066	8.512
38	4.204	7.232	8.170	8.754	9.255
39	4.395	7.777	8.844	9.525	10.088
40	4.599	8.378	9.595	10.390	11.026

Table 20.2 Passive earth pressure coefficients K_p consistent with EC7 (BSI, 1995), calculated using equation (20.13) for various values of effective friction angle ϕ' and relative soil/wall friction δ/ϕ'
Data taken from Powrie (2004)

in increased active earth pressure coefficients, reduced passive earth pressure coefficients, and an increased depth of embedment in the case of an embedded retaining wall.

It will be possible to draw a free body diagram for the wall that is in equilibrium under the action of the factored active and passive lateral stresses. In fact, this is the basis on which the required depth of embedment (for an embedded wall) is calculated. However, the idealised earth pressures obtained from the limit equilibrium analysis with factored soil strengths are unlikely to correspond to those really acting on the wall, and they are likely to overestimate wall bending moments (but not prop loads). The reasons for this are varied and complex; for an embedded wall they may include:

- relative wall/soil flexibility, which will influence local strains and hence mobilised strengths along the depth of the wall (see Rowe, 1952);
- different relationships between wall movement and shear strain on either side of an embedded wall (see Bolton and Powrie, 1988);
- different rates of mobilisation of soil strength with strain or wall movement on either side of an embedded wall, arising from the different recent stress histories relative to the stress paths followed during excavation (see Powrie *et al*., 1998).

These effects are most likely to affect the linearity and magnitude of the lateral stresses with embedded walls propped at the crest, and multi-propped structures such as building basements. For these types of construction, a numerical soil-structure interaction analysis is likely to lead to some economies in design and specification (Gaba *et al.*, 2003).

Concern has also been expressed regarding the possibility of high lateral stresses on embedded *in situ* retaining walls in overconsolidated soils, as a result of the stress history of these deposits (a discussion of which is given in Powrie, 2004). However, field studies of these walls have generally produced no evidence for this (e.g. Richards *et al.*, 2007), probably as a result of the factors listed above, together with the effects of wall installation in reducing the lateral stresses prior to excavation in front of the wall to below the *in situ* values (Richards *et al.*, 2006). In many cases, the use of active earth pressure coefficients behind the wall based on the full soil strength and steady-state pore water pressures, will probably give a reasonable estimate of the bending moments likely to occur in service (Batten and Powrie, 2000; Powrie and Batten, 2000; Gaba *et al.*, 2003). However, this approach will tend to underestimate prop loads (Gaba *et al.*, 2003).

20.5 Summary

- Classical earth pressure theory is based on calculating limiting lateral earth pressures, corresponding to conditions of failure in the soil, and in its simplest form is applied to frictionless or adhesionless retaining walls.
- Active conditions correspond to failure with the major principal stress vertical, and represent the minimum possible lateral stress for a given vertical stress, and are usually associated with conditions behind a retaining wall. Passive conditions correspond to failure with the major principal stress horizontal, and represent the maximum possible lateral stress for a given vertical stress – usually associated with conditions in front of a retaining wall.
- Active and passive pressures can be calculated using either the total stress failure criterion (for clay soils, undrained) or the effective stress failure criterion. In the latter case, the pore water pressures must be considered explicitly as well.
- In an undrained calculation, consideration must be given to the possibility of a flooded tension crack developing between a retaining wall and the soil behind it.
- The simple expressions for active and passive lateral stresses developed for a frictionless or adhesionless wall are usually modified for use in practice to take account of the effects of wall friction or adhesion. However, consideration should then be given to the likely direction of relative soil/wall movement to ensure that wall adhesion or friction can be relied on.
- The in-service stresses acting on a retaining wall are likely to be different from those at failure in the ground, owing to the effects of wall flexibility and differential rates of mobilisation of soil strength (soil stiffness) behind and in front of the wall.

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All chapters within Sections 1 Context and 2 Fundamental principles together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

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Chapter 21
Bearing capacity theory

William Powrie University of Southampton, UK

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This chapter discusses the theory behind the well-known formulae for the bearing capacity of soils. Theoretical solutions for an infinitely long strip foundation on a weightless soil, with the soil above the founding plane acting simply as a surcharge, are presented for both total stress and effective stress failure criteria. The use of empirical adjustment factors to take account of foundation shape, depth and the self-weight of the soil below the founding plane is described. Finally, the effect on bearing capacity of an inclined and/or eccentric load is discussed.

21.1 Introduction

The bearing capacity of a foundation is one of the classical problems in geotechnical engineering. The solution for a long foundation on a soil obeying the undrained shear strength failure criterion $\tau_{max} = c_u$ was published by Prandtl (1920), in the context of the indentation of a metal by a flat die. The corresponding solution for a weightless soil of effective friction angle ϕ' obeying the effective stress (frictional) failure criterion $(\tau/\sigma')_{max} = \tan\phi'$ was given by Reissner (1924).

In the first case, the solution relates the difference between the vertical total stress on the foundation at failure, σ_f, and the vertical total stress in the ground on either side at the level of the bottom of the foundation (the *founding plane*), σ_o, to the undrained shear strength, c_u:

$$\frac{(\sigma_f - \sigma_o)}{c_u} = N_c = (2 + \pi) = 5.14 \qquad (21.1)$$

The ratio $(\sigma_f - \sigma_o)/c_u$ is known as a *bearing capacity factor* and is given the symbol N_c in an undrained (total stress) analysis. The side surcharge, σ_o, will usually arise because of the weight of the soil above the base of the foundation: $\sigma_o = \int \gamma dz = \gamma D$ for a foundation of depth D in a soil of uniform unit weight γ. The weight of the soil does not otherwise affect the calculation, because below the founding plane it adds equally to both σ_f and σ_o and therefore cancels itself out.

In the second case, the solution is in terms of the ratio of the vertical effective stress on the foundation at failure, σ'_f, and the vertical effective stress in the ground on either side at the level of the foundation base σ'_o:

$$\frac{\sigma'_f}{\sigma'_o} = N_q = K_p e^{(\pi \tan \phi')} \qquad (21.2)$$

where ϕ' is the effective angle of friction and $K_p = (1 + \sin\phi')/(1 - \sin\phi')$ is the passive earth pressure coefficient (see Chapter 20). In this case, the bearing capacity factor N_q is the ratio of the vertical effective stresses on the foundation and in the soil to either side, at the level of the foundation base. As before, the side surcharge σ'_o will usually arise because of the weight of the soil above the base of the foundation, $\sigma'_o = \int \gamma dz - u = (\gamma D - u)$ for a foundation of depth D in a soil of uniform unit weight γ, where u is the pore water pressure at the level of the base.

The undrained shear strength and effective stress failure criteria are discussed in Chapter 20 *Earth pressure theory*, and the analyses leading to equations (21.1) and (21.2) are given in full in, for example, Powrie (2004).

21.2 Bearing capacity equation for vertical load – empirical adjustments for shape and depth

The fact that the effective stress solution is developed for a weightless soil is, in practice, an important potential limitation in the use of equation (21.2). For every 10 kPa increase in the vertical effective stress σ'_o on either side of the foundation, the effective stress underneath the foundation, σ'_f, increases by N_q times this, – which for $\phi' = 30°$ could be 184 kPa (with $\phi' = 30°$, $N_q = 18.4$ according to equation (21.2)). Failure of the foundation will involve a mechanism extending to a depth approximately equal to the foundation width B below the founding plane. Thus the use of equation (21.2) on its own would significantly underestimate the true bearing capacity, because the increase in σ'_f with depth is neglected. The depth of soil below the founding plane that takes part in the eventual failure mechanism may be taken into account empirically by adding a further term to equation (21.2), known as the N_γ term, given by

$$N_\gamma \times (0.5\gamma B - \Delta u) \qquad (21.3)$$

where N_γ is a factor analogous to N_q, B is the foundation width, γ is the unit weight of the soil, Δu is the increase in pore water pressure between the founding plane and a depth of $0.5B$ below it, and $(0.5\gamma B - \Delta u)$ is then the increase in vertical effective stress over the same depth, in the soil on either side of the foundation. Thus equation (21.3) gives the bearing capacity of a surface footing for which the depth of embedment D (and

hence σ'_o) is zero. This is added to the basic equation (21.2) to estimate the bearing capacity of a long footing embedded to a depth D.

Two further problems with equations (21.1–21.3) are that in reality, (i) most foundations are not infinitely long and (ii) the strength of the soil on either side of the foundation above the founding plane, which is assumed in the analyses to act simply as a surcharge, will contribute to the bearing capacity of the foundation, especially as the depth of the founding plane D is increased. These are overcome by the use of empirical factors, usually based on the results of laboratory tests on model footings, to modify the basic theoretical equations.

In effective stress terms, the bearing capacity equation is written in the form

$$\sigma'_f = \left(N_q \times s_q \times d_q \times \sigma'_o\right) + \left[N_\gamma \times s_\gamma \times d_\gamma \times r_\gamma \times \left(0.5\gamma B - \Delta u\right)\right] \quad (21.4)$$

where, in addition to the symbols already defined,

- s_q and s_γ are shape factors, which account for the fact that the footing is not infinitely long;
- d_q and d_γ are depth factors, which account for the fact that in reality, the soil above the founding plane has some strength and therefore does more than act simply as a surcharge;
- r_γ is a reduction factor, to prevent the N_γ term increasing indefinitely with footing width B.

N_q, the basic bearing capacity factor, is generally taken as $K_p e^{\pi \tan\phi'}$ for a vertical load applied through the centre of the footing. Values of s_q, d_q, N_γ, s_γ, d_γ and r_γ suggested by Meyerhof (1963), Brinch Hansen (1970) and Bowles (1996) are given in **Table 21.1**.

For foundations on clay soils loaded quickly, the undrained bearing capacity equation may be written as

$$\left(\sigma_f - \sigma_o\right) = \left[N_c \times s_c \times d_c\right] \times c_u. \quad (21.5)$$

Parameter	Meyerhof (1963)	Brinch Hansen (1970)
Shape factor s_q	$1 + 0.1K_p(B/L)$	$1 + (B/L)\tan\phi'$
Depth factor d_q	$1 + 0.1\sqrt{(K_p)}(D/B)$	$1 + 2\tan\phi'(1 - \tan\phi')k$
$N\gamma$	$(N_q - 1)\tan(1.4\phi')$	$1.5(N_q - 1)\tan\phi'$
Shape factor s_γ	$= s_q$	$1 - 0.4(B/L)$
Depth factor d_γ	$= d_q$	1

Notes: Meyerhof's expressions apply for $\phi' > 10°$
$k = D/B$ if $D/B \le 1$; $k = \tan^{-1}(D/B)$ (in radian) if $D/B > 1$
$K_p = (1 + \sin\phi')/(1 - \sin\phi')$
$r_\gamma = 1 - 0.25\log_{10}(B/2)$ for $B \ge 2$ m (Bowles, 1996)
Foundation length L, breadth B and depth D

Table 21.1 Bearing capacity enhancement factors, effective stress failure criterion

Parameter	Skempton (1951)	Meyerhof (1963)
Shape factor s_c	$1 + 0.2(B/L)$	$1 + 0.2(B/L)$
Depth factor d_c	$1 + 0.23\sqrt{(D/B)}$, up to a maximum of 1.46 ($D/B = 4$)	$1 + 0.2(D/B)$

Note: Foundation length L, breadth B and depth D

Table 21.2 Bearing capacity enhancement factors, undrained shear strength failure criterion

For a vertical load acting at the centre of the foundation, the basic ϕ bearing capacity factor N_c is $(2 + \phi) = 5.14$, and s_c and d_c are enhancement factors accounting for the shape and depth of the footing respectively. In this case, there is no N_γ effect because the difference between the total stresses σ_f and σ_o is unaffected by the self-weight of the soil.

Values of s_c and d_c suggested by Skempton (1951) and Meyerhof (1963) are given in **Table 21.2**.

Brinch Hansen (1970) suggests an undrained bearing capacity equation of a slightly different form in which the enhancement factors are additive rather than multiplicative. He also gives adjustment factors that take account of inclined loads and founding planes.

Annex B of BSI (1995) gives equations of a similar form to (21.4) and (21.5), together with factors i_q and i_c intended to account for the effect of inclined loads but with the depth factors d_q and d_c omitted. However, a completely general bearing capacity equation in which parameter values are inserted to suit particular circumstances, can lead to confusion. In any case, inclined loads, non-horizontal footing bases and sloping ground on one or both sides of the footing can be taken into account analytically in the calculation of N_c or N_γ, as shown (for example) in Powrie (2004).

21.3 Inclined loading

The bearing capacity of a shallow foundation subjected to an inclined load acting through its centre can be investigated by modifying the stress analysis used to obtain the basic bearing capacity equations (21.1) and (21.2). An inclined load acting through the centre of the foundation may be split into its horizontal and vertical components, H and V.

For a foundation on a clay soil obeying the undrained shear strength (maximum shear stress) failure criterion, the loads are characterised by a normal stress $\sigma_f = V_f/B$ and a shear stress $\tau_f = H_f/B$, where B is the foundation width and V_f and H_f are the vertical and horizontal loads (in kN/m) at failure. The modified bearing capacity is then given by

$$\frac{\left(\sigma_f - \sigma_o\right)}{c_u} = N_c = \left(1 + \pi - \Delta + \cos\Delta\right) \quad (21.6)$$

where σ_o is the surcharge on either side of the foundation at the level of the founding plane, and $\Delta = \sin^{-1}(\tau_f/c_u)$.

For a foundation on soil obeying the effective stress (maximum stress ratio or frictional) failure criterion, the loads are

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characterised by a normal effective stress σ'_f (= V_f/B, assuming that the pore water pressure on the base of the foundation is zero) and a shear stress τ_f (= H_f/B), where B is the foundation width and V_f and H_f are the vertical and horizontal loads (in kN/m) at failure. The modified bearing capacity is then given by

$$\frac{\sigma'_f}{\sigma'_o} = N_q = \frac{\left[1+\sin\phi'\cos(\Delta+\delta)\right]}{\left[1-\sin\phi'\right]} \times e^{(\pi-\Delta-\delta)\tan\phi'} \quad (21.7)$$

where σ'_o is the effective stress surcharge on either side of the foundation at the level of the founding plane, $\sin\Delta = \sin\delta/\sin\phi'$, and $\delta = \tan^{-1}(\tau_f/\sigma'_o)$.

21.4 Offset loading

A vertical load V acting at a distance e from the geometrical centre of the foundation is statically equivalent to a vertical load V acting through the centre, together with a moment of Ve about the centre. Foundations subjected to combined vertical and moment loading are considered in section 21.5.

Alternatively, a simple approach (suggested by Meyerhof, 1963) to calculate the bearing capacity of a long shallow foundation subjected to an offset or eccentric vertical load is to reduce the width of the foundation to $B' = (B - 2e)$ so that the load then acts through the geometric centre. If this reduced width is used with the limiting vertical stresses given by the appropriate equations presented in sections 21.1 and 21.2, a conservative (i.e. safe) estimate of the reduced load carrying capacity of the foundation will be obtained.

BSI (1995) requires that 'special precautions shall be taken where the eccentricity of loading exceeds 1/3 of the width of a rectangular footing or 0.6 of the radius of a circular footing', because this brings the point of application outside the equivalent of the 'middle third' of the foundation, leading to a tendency to tension or uplift at the edge away from the load.

21.5 Combined vertical, horizontal and moment (*V–H–M*) loading interaction diagram for a surface foundation

In general, a foundation may be subjected to a combination of vertical (V), horizontal (H) and moment (M) loads, either as a result of the application of an inclined point load offset from the centre of the footing (**Figure 21.1(a)**), or directly (**Figure 21.1(b)**). These are statically equivalent, with $V = R\cos\alpha$, $H = R\sin\alpha$ and $M = R\cos\alpha\, e$.

An alternative approach to calculating the bearing capacity of a surface footing subjected to simultaneous vertical, horizontal and moment loading is given by Butterfield and Gottardi (1994). On the basis of a large number of small-scale (model) tests, they showed that combinations of V, H and M at failure lay on a unique three-dimensional surface or failure envelope when plotted on a three-dimensional graph with axes V, H and M/B, where B is the breadth of the footing. The failure surface may be described as cigar-shaped, and is parabolic when viewed in either the V–H or the V–M/B plane (**Figure 21.2**).

Butterfield and Gottardi (1994) normalised their experimental values of V, H and M/B with respect to the vertical load V_{max} that would cause failure of the footing when acting on its own. For a foundation placed directly on the soil surface, the bearing capacity in symmetrical vertical loading σ'_f may be calculated using equation (21.4) with the surcharge on either side due to the weight of soil above the founding plane $\sigma'_o = 0$. The depth

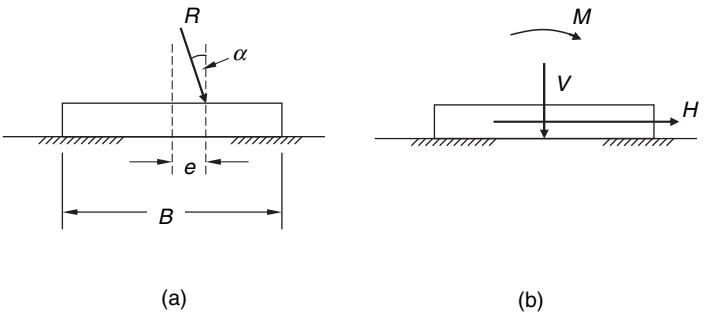


Figure 21.1 Statical equivalence of (a) an eccentric, inclined point load and (b) combined vertical, horizontal and moment loading through the centroid of a shallow foundation (Butterfield and Gottardi, 1994)

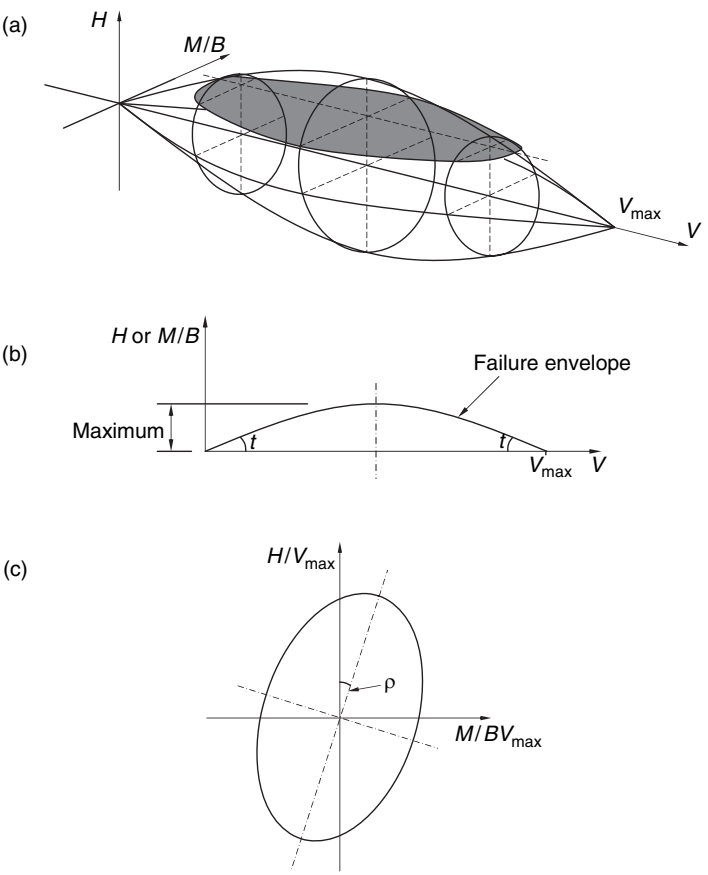


Figure 21.2 Failure envelopes for a shallow footing: (a) three-dimensional view in (*V–M/B–H*) space; (b) *H* or *M/B* against *V*; (c) normalised cross-section perpendicular to the *V* axis at $V = V_{max}/2$ (Parts (a) and (b) from Butterfield and Gottardi, 1994)

factor d_γ and the associated reduction factor r_γ are also omitted because the depth of burial of the foundation D is zero, giving

$$\sigma'_f\,(\text{kPa}) = N_\gamma \times s_\gamma \times (0.5_\gamma B - \Delta u). \tag{21.8}$$

If the foundation is of width B and length L, the vertical load at failure, acting through the centroid, is

$$V_{max}\,(\text{kN}) = \sigma'_f BL = N_\gamma s_\gamma (0.5\gamma B - \Delta u)BL. \tag{21.9}$$

The parabolas representing the projections of the three-dimensional failure surface in the V–H and V–M/B planes have equations

$$\frac{H}{t_h} = \frac{V(V_{max} - V)}{V_{max}} \tag{21.10a}$$

and

$$\left(\frac{M}{B}\right) \Big/ t_m = \frac{V(V_{max} - V)}{V_{max}} \tag{21.10b}$$

where t_h and t_m are the slopes of the parabolas at the origin in the V–H and V–M/B planes respectively (**Figure 21.2(b)**). For Butterfield and Gottardi's experiments, $H_{max} \approx V_{max}/8$ and $(M/B)_{max} \approx V_{max}/11$, corresponding to $t_h = 0.5$ and $t_m = 0.36$. H_{max} is defined at $M/B = 0$, $(M/B)_{max}$ is defined at $H = 0$, and both occur at $V = V_{max}/2$. Sections through the failure surface in the M/B–H plane (i.e. when viewed along the V axis) are ellipses, rotated through an angle ρ from the H axis towards the positive M/B axis (**Figure 21.2(c)**). The reason for this rotation is that the H and M/B loads can act either together or in opposition: if H and M/B act in the directions shown in **Figure 21.1**, the horizontal load at failure will be smaller than if either H or M/B is reversed.

The three-dimensional failure surface may be represented by an equation of the form

$$\left(\frac{H \Big/ V_{max}}{t_h}\right)^2 + \left(\frac{M \Big/ BV_{max}}{t_m}\right)^2 - \left(\frac{2C\dfrac{M}{BV_{max}}\dfrac{H}{V_{max}}}{t_h t_m}\right) = \left[\frac{V}{V_{max}}\left(1 - \frac{V}{V_{max}}\right)\right]^2 \tag{21.11}$$

where the constant C is a function of t_h, t_m and the inclination ρ to the H axis of the major axis of the cross-sectional ellipse. Data from three series of plane strain tests on foundations of different widths on different types and density of sand have a best fit failure surface represented by equation (21.11) with $t_h = 0.52$, $t_m = 0.35$, $\rho = 14°$ and C = 0.22.

The main practical implications of the above are:

- Surface footings are particularly vulnerable to horizontal and moment loading, with $H \approx V_{max}/8$ or $M/B \approx V_{max}/11$ being sufficient to cause failure. For this reason, foundations that are required to carry significant horizontal or moment loads are generally piled (see Chapter 22 *Behaviour of single piles under vertical loads*), or at least buried to some extent.
- The remoteness from the failure surface of a point on the (V–M/B–H) diagram representing a general load will depend on the load path followed to failure. Hence a factor of safety defined as a factor on the collapse load is likely to be ambiguous and potentially unsafe.
- If a surface footing is to be as safe as possible when subjected to a general load increment, the design vertical load should be about $V_{max}/2$.

It is difficult, if not impossible, to achieve a high factor of safety on load for a shallow foundation subjected to a load path that involves an increase in H or M coupled with a reduction in V. This is illustrated in an example given by Powrie (2004).

21.6 Summary

- Classical theory can be used to calculate the bearing capacity of an infinitely long strip foundation on a weightless soil, with the soil above the founding plane acting simply as a surcharge, for both total stress and effective stress failure criteria.
- Empirical adjustment factors are then applied to take account of foundation shape, depth and the self-weight of the soil below the founding plane.
- An inclined and/or eccentric load will affect the bearing capacity of a foundation: either a theoretical or an empirical approach can be used to take this into account, as appropriate.

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All chapters within Sections 1 Context and 2 Fundamental principles together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

Chapter 22

Behaviour of single piles under vertical loads

John B. Burland Imperial College London, UK

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This chapter concentrates on the prediction of the axial capacity of piles in clay and granular materials. It is stressed that piling is a geotechnical process, controlled by an operator, the outcome of which depends on numerous factors. It is therefore not reasonable to expect to be able to predict pile behaviour with great precision.

The fundamental mechanisms of load–settlement behaviour are described. The traditional approach to pile design in clay is presented using undrained strength as the controlling variable. The limitations of this approach are stressed and particular attention is drawn to the wide scatter of results on which design guidance is based. Because of these limitations, much of the chapter is devoted to the application of the effective stress method for pile shaft design. It is shown to give reasonable predictions of the shaft resistance of piles in soft clays with negative friction. The effective stress approach is also used in correlating the results of pile tests on overconsolidated clays. It is shown that these correlations are much better defined than by using undrained strength. The traditional approaches to the design of driven piles in granular soils are described.

22.1 Introduction

The installation of a pile is rather like other geotechnical processes, such as grouting and tunnelling. In essence, a geotechnical process is controlled by an operator or contractor and the outcome of the process depends on numerous factors – including the ground profile and groundwater conditions (see Chapter 13 *The ground profile and its genesis*), the type of pile (see Chapter 54 *Single piles*), the type of piling plant used, the skill and experience of the operator, and the supervision and quality control. Thus successful delivery of the project depends not only on the design and specification of the piles, but on the process of installation.

It is often said that there are as many theories of pile behaviour as there are piling engineers! What is abundantly clear as one studies the vast literature on the subject is that much of it is highly empirical. As pointed out in Chapter 4 *The geotechnical triangle*, empiricism is a key activity but it must be used rigorously with a clear understanding of the basis of the empiricism.

Even though piling engineering is operator-dependent and so inevitably empirical, nevertheless the behaviour of piles has been shown to depend on basic soil mechanics principles. Understanding these principles provides the key to understanding pile behaviour and the many factors that control it. The aim of this chapter is therefore to describe the basic soil mechanics principles that control the carrying capacity of piles.

This chapter does not cover the specialist subject of piles in rock. For an introduction to this topic the reader is referred to Fleming *et al.* (2009).

22.2 Basic load–settlement behaviour

Figure 22.1(a) shows a straight-shafted pile which has been installed in the ground in some manner and is then loaded by applying an axial load P to the top of the pile. This load is transmitted into the ground by the shaft resistance Q_s and the base resistance Q_b. For simplicity we will assume that the pile is rigid and does not compress under load.

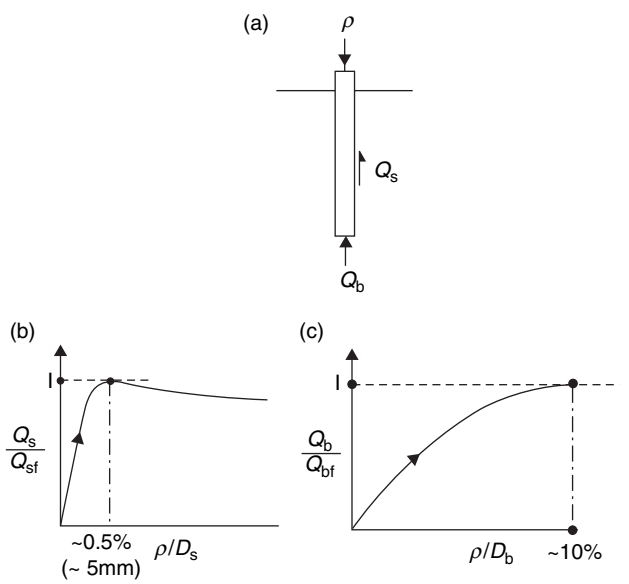


Figure 22.1 Contrasting development of shaft and base resistance with increasing settlement for straight-shafted pile in clay

As the applied load P is steadily increased, settlement ρ of the pile takes place. We will consider separately the development of the shaft resistance and the base resistance as the settlement increases.

22.2.1 Development of the shaft resistance

Figure 22.1(b) shows a graph of the development of the shaft resistance with increasing settlement. The axes are expressed non-dimensionally as Q_s/Q_{sf} versus ρ/D_s where Q_{sf} is the maximum shaft resistance and D_s is the diameter of the shaft. Numerous instrumented pile tests in a variety of ground types have shown that the shaft resistance is fully developed at small settlements. This settlement is usually about 0.5% of the shaft diameter or, alternatively, somewhere between 5 mm and 10 mm for routine pile diameters. Sometimes the shaft resistance reaches a maximum value and then slowly decreases with further settlement; on other occasions it may continue to increase slowly with increasing settlement. It is extremely unusual for the shaft resistance to reduce rapidly and by a large amount after the maximum value has been reached. In this sense, the shaft resistance usually exhibits ductile load–settlement behaviour.

22.2.2 Development of the base resistance

Figure 22.1(c) shows a graph of the development of the base (or end) resistance of a pile with increasing settlement. As for the shaft, the axes are expressed non-dimensionally as Q_b/Q_{bf} versus ρ/D_b, where Q_{bf} is the maximum base resistance and D_b is the diameter of the base. In contrast to the shaft resistance, the base resistance (both for bored and driven piles) requires much larger movements to develop fully – of the order of 10–20% of the base diameter.

22.2.3 Development of the combined resistance

The load–settlement behaviour of a pile is determined by the combined behaviour of the shaft and the base. **Figure 22.2(a)** shows the resultant load–settlement behaviour of a straight-shafted pile in clay, where the base only contributes a small proportion of the ultimate carrying capacity of the pile. The contributions of the shaft and base are shown as broken lines. Such a pile is often referred to as a *friction pile*. This behaviour can be contrasted with the load–settlement behaviour of a pile with an enlarged base, as shown in **Figure 22.2(b)**. In this case, the base makes a much larger contribution to the ultimate carrying capacity of the pile. For such piles, the shaft resistance is often fully developed under working load. Hence a pile with an enlarged base must settle more than a friction pile under working load if it is to operate efficiently.

In summary, it is important to have a clear appreciation of the two very different mechanisms of load transfer – the shaft resistance develops at relatively small settlements, whereas the base resistance requires much larger settlements to fully develop. It is also evident from **Figures 22.2 (a)** and **(b)** that the proportion of the applied load, carried by the shaft and the base, changes as the applied load increases. Thus, for a pile with an enlarged base, whereas under working load the shaft might carry (say) 80% of the applied load, at failure the shaft might only be carrying 20% of the applied load. Having a clear understanding of these two mechanisms of load transfer is very helpful when interpreting the results of pile loading tests.

22.2.4 Measured load–settlement behaviour of an under-reamed bored pile

Whitaker and Cooke (1966) carried out a seminal series of load tests on bored piles in London Clay at Wembley, in which the load carried by the base was measured by means of a sophisticated load cell. Knowing the applied and base loads at any stage of the test, it was possible to deduce the load carried by the shaft.

Figure 22.3 shows the results of one of Whitaker and Cooke's tests. In this case, the diameters of the shaft and base were 0.8 m and 1.7 m respectively and the depth of the pile was 12 m. For the first part of the test the loads on the pile were applied in increments, with the load at each stage being

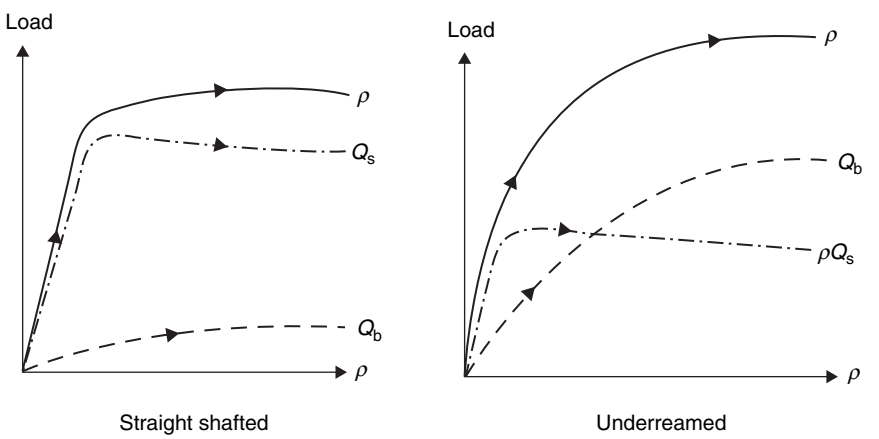


Figure 22.2 Contrasting load–settlement behaviour of (a) straight-shafted and (b) under-reamed piles in clay

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maintained constant until settlement had practically ceased. In some cases the load was held constant for 30 or more days. The second part of the test consisted of carrying out a constant rate of penetration test on the pile so as to determine its ultimate carrying capacity. The full line in **Figure 22.3** shows the load–settlement behaviour of the pile as a whole. Note that, for clarity, the measurements made during the unload-reload cycle, that took place in changing from incremental loading to constant rate of penetration loading, have been omitted from the figure.

The broken line labelled 'Shaft' in **Figure 22.3** shows the measured load–settlement behaviour of the shaft. It is clear that the carrying capacity of the shaft is fully mobilised at very small settlements. The jagged 'saw tooth' shape of the load–settlement curve is of interest. When an additional increment of load is applied to the pile, the load carried by the shaft initially increases and then reduces as settlement takes place. The temporary increase of load carried by the shaft is a 'rate effect' due to the initial high rate of settlement with time (Burland and Twine, 1988). As the rate of settlement steadily reduces, so the load carried by the shaft reduces back to its previous value – with the reduction in shaft load being shed to the base. It can be seen that the equilibrium fully mobilised carrying capacity of the shaft remains remarkably constant with increasing settlement, confirming the ductile nature of the shaft behaviour as described in section 22.2.1. It is evident from **Figure 22.3** that the measured load–settlement behaviour of the base is dramatically different from that of the shaft, being much 'softer' and with full mobilisation taking place at significant settlements.

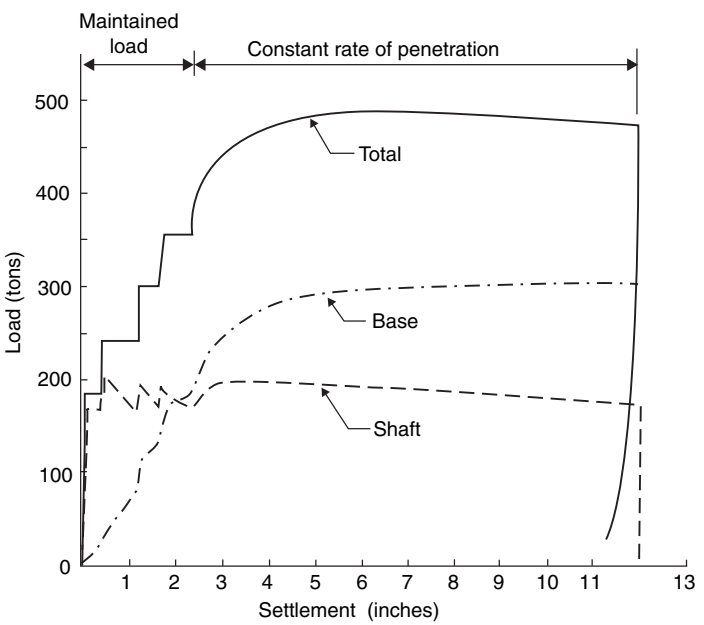


Figure 22.3 Measured load–settlement relationships for instrumented test on an under-reamed pile in London Clay
Reproduced from Whitaker and Cooke (1966)

It is important to note that for working loads in excess of about 170 tons, the pile will be operating with full mobilisation of the shaft-carrying capacity. There must be tens of thousands of large diameter bored piles operating satisfactorily in this way worldwide. As will be described in section 22.4, shaft resistance is essentially a frictional phenomenon, and as such is permanent and reliable – provided the effective stresses acting on the pile do not change significantly.

22.3 Traditional approach to estimating the axial capacity of piles in clay

It has become a well-established tradition to relate the carrying capacity of a pile in clay to undrained strength S_u. This is because undrained strength data are usually included with pile tests' results and it has proved very convenient to attempt to establish correlations between measured pile capacity and undrained strength. Unfortunately this empirical convenience is often overlooked and many piling engineers have come to regard the undrained strength as a fundamental controlling factor. In section 22.4 it is shown that this is not the case for shaft resistance. It is necessary to understand that undrained strength is not a straightforward parameter and this is discussed below.

22.3.1 Undrained strength of clay

The undrained strength of a soil depends on many factors, the following being the principal ones:

(i) *Stress path* For many soils, particularly normally or lightly overconsolidated clays, the undrained strength in plane strain or triaxial extension can differ significantly from the undrained strength measured in triaxial compression, S_{utc}. For soft clays, the strength in plane strain or triaxial extension can be less than the value in triaxial compression and may be as low as half this value.

(ii) *Orientation* The undrained strength usually depends on the orientation of the principal stresses, i.e. the undrained strength is anisotropic.

(iii) *Size* The undrained strength is often critically dependent on the volume of soil being tested. In stiff fissured clays, the undrained strength in the mass may be much less than the average strength of small samples due to the dominant influence of the presence of fissures.

(iv) *Rate* The undrained strength depends on the rate of testing. Usually, the slower the rate, the lower the strength.

(v) *Sample disturbance* This can be a major factor and can operate either way – sometimes reducing strength and sometimes increasing it. A particularly important problem occurs with stiff clays containing saturated silt or sand partings. The stress relief during sampling causes the samples to take up moisture from the partings and soften.

When the above factors are appreciated, it is obvious that different types of test and different sizes of samples give differing

values of S_u. In particular, the values of S_u derived from *in situ* tests such as vane, cone and pressuremeter, will frequently differ from the values measured in triaxial compression. For these reasons, when pile behaviour (in particular shaft resistance) is correlated with undrained strength, it is necessary to be absolutely clear as to how the strength was determined.

22.3.2 Ultimate capacity of whole pile

Figure 22.4 depicts a pile of weight W in a state of failure under a load P_f such that the total shaft resistance is Q_{sf} and the total base resistance is Q_{bf}. The average undrained strength over the length L of the pile is $\overline{S}_{u(shaft)}$ and the undrained strength appropriate to the base is $S_{u(base)}$. For the whole pile at failure:

$$P_f + W = Q_{bf} + Q_{sf} \ . \tag{22.1}$$

The base resistance can be derived from basic bearing capacity theory given in Chapter 21 *Bearing capacity theory* (equation (21.5)):

$$Q_{bf} = A_b[\{N_c \times s_c \times d_c\} \times S_{u(base)} + p_o] \tag{22.2}$$

where A_b is the area of the base and p_o is the overburden pressure (= $\gamma_{bulk}.L$). For a surface strip foundation, the basic bearing capacity factor N_c = 5.14, and for a deep circular or square foundation, the shape factor s_c = 1.2 and the depth factor d_c = 1.46. Hence equation (22.2) can be re-written:

$$Q_{bf} = A_b[9 \times S_{u(base)} + \gamma_{bulk}.L] \ . \tag{22.3}$$

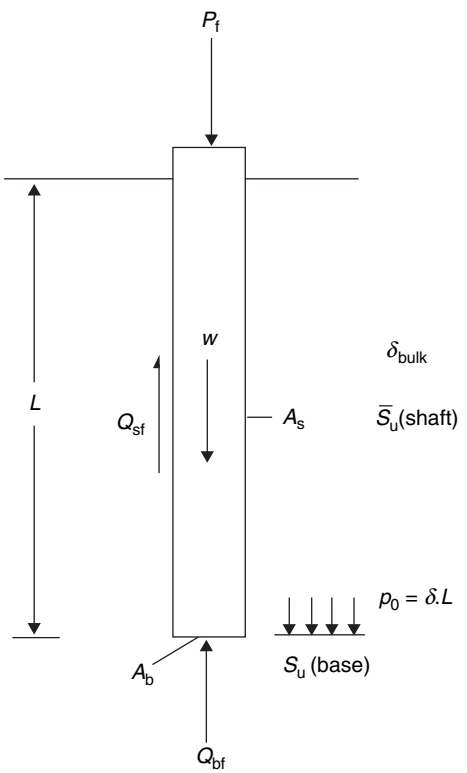


Figure 22.4 Ultimate capacity of a whole pile: a pile of weight W in a state of failure under a load P_f such that the total shaft resistance is Q_{sf} and the total base resistance is Q_{bf}

Undrained strength is used for assessing the base resistance of piles in clay because the undrained bearing capacity is usually much less than the drained bearing capacity, and the end resistance is at its lowest during the application of most of the working load.

For the shaft:

$$Q_{sf} = A_s \times \overline{\tau}_{sf} \tag{22.4}$$

where A_s is the surface area of the shaft and $\overline{\tau}_{sf}$ is the average shaft friction over the length of the shaft. Methods of estimating the value of $\overline{\tau}_{sf}$ are discussed later.

The ultimate capacity of the whole pile is obtained by substituting equations (22.3) and (22.4) into equation (22.1) as follows:

$$P_f + W = A_b[9 \times S_{u(base)} + \gamma_{bulk}.L] + A_s \times \overline{\tau}_{sf} \ . \tag{22.5}$$

The assumption is frequently made that the weight of the pile is equal to the weight of the soil it displaces, so that $W = A_b \times \gamma_{bulk} \times L$ and equation (22.5) simplifies to:

$$P_f = A_b \times 9 \times S_{u(base)} + A_s \times \overline{\tau}_{sf} \ . \tag{22.6}$$

For piles bearing on a strong stratum or with enlarged bases, much of the load at failure will be carried by the base. Hence care is needed in selecting the appropriate value of $S_{u(base)}$ to insert into equation (22.6). When the clay is fissured, the design strength should be close to the lower limit of the scatter of unconsolidated undrained triaxial compressive strengths (Burland, 1990).

22.3.3 Estimation of shaft resistance from undrained strength

Fleming *et al.* (2009) point out that most piles in clay develop a high proportion of their overall capacity along the shaft. Hence the reliable determination of the shaft friction is important. For reasons that were explained at the beginning of this section, it has become traditional to relate the average shaft friction $\overline{\tau}_{sf}$ to the average undrained strength $\overline{S}_{u(shaft)}$ over the length of the shaft such that:

$$\overline{\tau}_{sf} = \alpha \times \overline{S}_{u(shaft)} \ . \tag{22.7}$$

The factor α is known as the 'shaft adhesion factor' and is a purely empirical quantity which has been evaluated from the results of pile tests. It is not a constant but depends on a number of variables.

One of the most important considerations when adopting values of α in design is to be very clear about how the values of $\overline{S}_{u(shaft)}$ were determined in the original empirical correlation. This point can be illustrated by referring to the classic work of

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Skempton (1959) who analysed a number of load tests on cast *in situ* bored piles in London Clay and obtained an average value of $\alpha = 0.45$. He began his study by plotting the results of undrained triaxial tests on 1.5 inch (38 mm) diameter samples against depth, as shown in **Figure 22.5**. Each point represents the average of two to four samples at a given depth. Even so, it can be seen that there is a considerable scatter and this is common for London Clay, due largely to its fissured nature. Skempton drew a mean line through the strength results as shown by the full line. He used this basic line to obtain values of $\overline{S}_{u(shaft)}$ and $S_{u(base)}$ for each of the piles in the study. For each test pile, the value of Q_{bf} was estimated from bearing capacity theory and was subtracted from the measured ultimate load to obtain the value of Q_{sf} and hence the value of $\overline{\tau}_{sf}$ $(= Q_{sf}/A_s)$. **Figure 22.6** shows Skempton's plot of $\overline{\tau}_{sf}$ (y-axis) versus $\overline{S}_{u(shaft)}$ (x-axis). It can be seen that the points correspond to values of $\alpha (= \overline{\tau}_{sf}/\overline{S}_{u(shaft)})$ ranging between 0.3 and 0.6 with an average value of 0.45. Hence the recommended value of $\alpha = 0.45$ is based on two averaging processes of widely scattered data, firstly of the measured strength–depth results, and secondly of the correlation between $\overline{\tau}_{sf}$ and $\overline{S}_{u(shaft)}$.

In recent years, the practice has developed of measuring the undrained strength in triaxial compression using 98 mm rather than 38 mm diameter samples. It has been found that the average strength of 98 mm diameter samples of stiff fissured clay is significantly lower than for 38 mm diameter samples, because the larger samples contain more fissures (Marsland, 1974; Burland, 1990). Hence it is no longer appropriate to adopt a value of $\alpha = 0.45$; a higher value should be used to account for the lower average measured strengths.

Tomlinson (1995) has published design curves of α versus $\overline{S}_{u(shaft)}$ for driven piles in clay, based on the analysis of published and unpublished records of pile tests. In general, the suggested design values of α reduce from about 1 (at low undrained strengths) to as low as 0.25 (for strengths greater than about 150 kPa). The data points on which the suggested design curves are based are illustrated in **Figure 22.7** – which can be seen to show a very wide scatter (Tomlinson, 1971). Similarly, Weltman and Healy (1978) propose curves of α versus undrained strength for bored and driven piles in glacial till, based on data which shows a wide scatter. Hence great care is needed in the application of such design curves which should be regarded as indicative only.

In view of the uncertainties associated with the correlations between the shaft adhesion factor α and the average undrained strength, it is clearly advisable to carry out pile loading tests for specific projects. It is of course vital that the test piles are installed in the same way as the proposed working piles.

22.4 Shaft friction of piles in clay, in terms of effective stress

Section 22.3.3 describes the traditional approach of evaluating shaft friction as a proportion α of the average undrained strength $\overline{S}_{u(shaft)}$ over the length of the shaft. It was pointed out that the

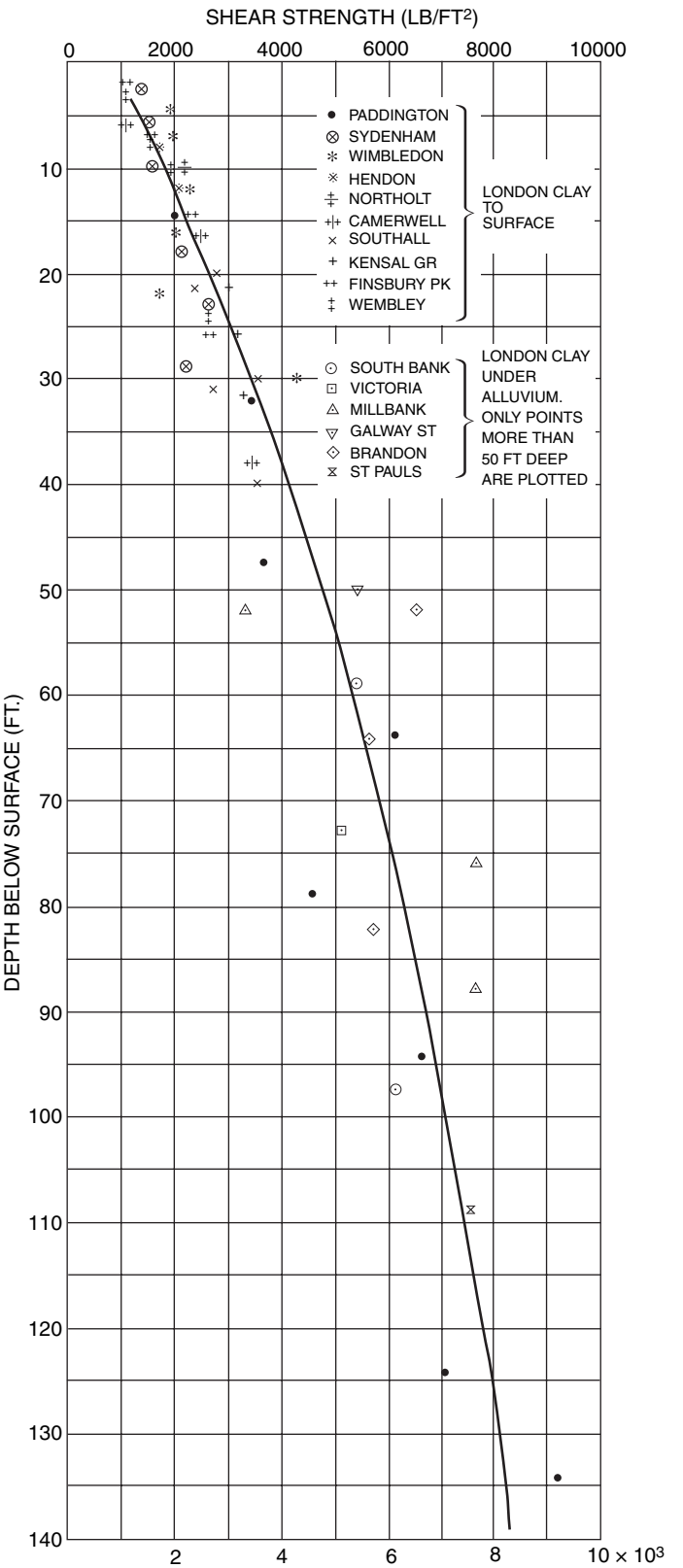


Figure 22.5 Values of average undrained strength S_u plotted against depth from various sites in London
Reproduced from Skempton (1959)

undrained strength does not have a unique value for a soil and is dependent on the test method and the quality of sampling. In stiff clays, the scatter of the undrained strength results can be very large due to the fissured nature of the clay. Examination of the pile test data on which suggested design values of α are based, shows that the scatter is also very large. In addition to these uncertainties, a major problem with the traditional approach to evaluating shaft friction is that it is wholly empirical; there is no fundamental underpinning theoretical framework as an aid to understanding the key factors controlling shaft friction. The effective stress approach also has its limitations but it is soundly based on fundamental soil mechanics principles.

22.4.1 Shaft friction in terms of the effective stress parameter β

The basis of the effective stress approach was first suggested by Johanneson and Bjerrum (1965) in relation to assessment of negative friction. Chandler (1968) suggested it could be

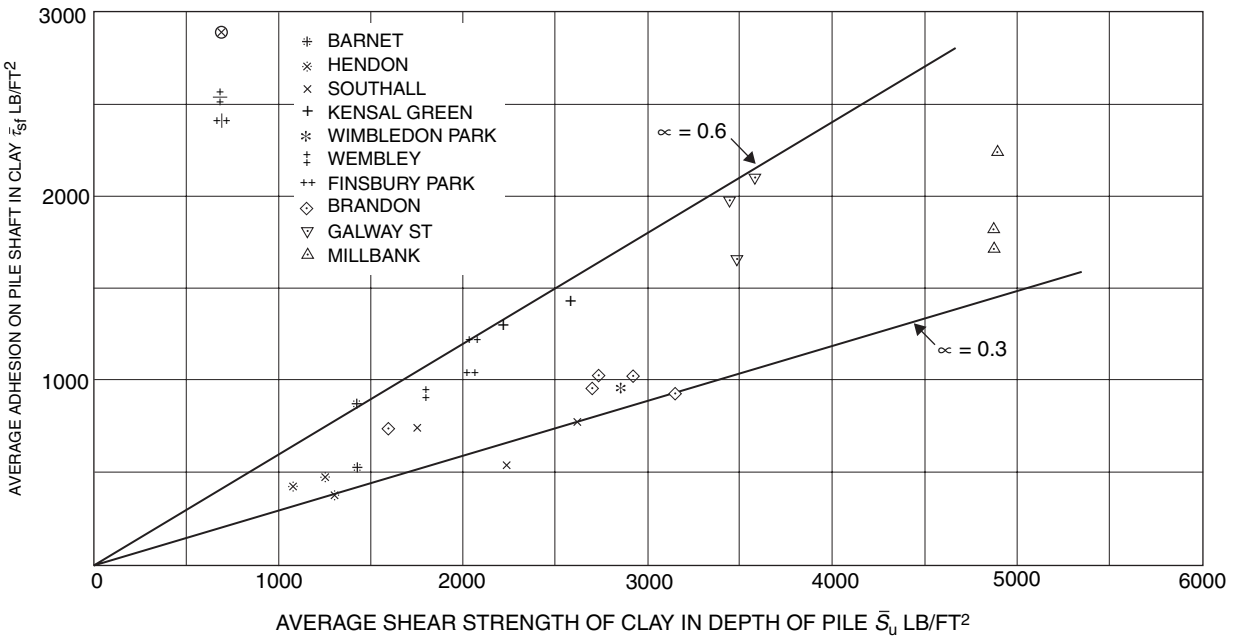


Figure 22.6 Range of measured α values for bored piles in London Clay
Reproduced from Skempton (1959)

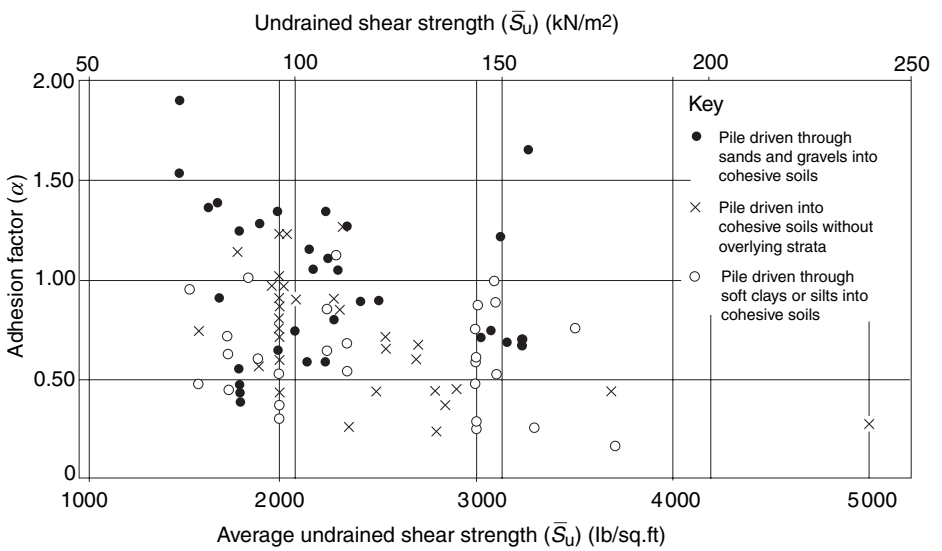


Figure 22.7 Relationship between adhesion factor α and average undrained strength over length of piles embedded in stiff to very stiff clays
Reproduced from Tomlinson (1971)

applied to the interpretation of the results of tests on bored piles in London Clay, and Burland (1973) applied it more generally. Other papers followed, notably by Meyerhof (1976) and Flaate and Selness (1977).

Research on instrumented piles (e.g. Lehane *et al.*, 1994) shows that limiting shaft friction is controlled by the simple Coulomb effective stress interface sliding law:

$$\tau_{sf} = \sigma'_{rf} \tan \delta'_f \tag{22.8}$$

where σ'_{rf} is the radial effective stress acting on the shaft at failure and δ'_f is the effective angle of interface friction at failure. Thus, the problem of evaluating shaft friction at any depth on a pile requires the determination of these two quantities.

The value of the radial effective stress at failure σ'_{rf} depends on the following factors, in approximate order of significance: (a) the vertical effective overburden stress σ'_v; (b) the overconsolidation history of the deposit which can be simplified to the evaluation of the overconsolidation ratio (OCR); (c) the effects of installing the pile; and (d) the change in σ'_r during loading. With regard to (c), it is normally assumed that any excess pore pressures set up during installation are completely dissipated. Regarding (d), it is reasonable to assume that loading takes place under fully drained conditions because the zone of major distortion adjacent to the pile is relatively thin and drainage will take place rapidly.

Because of the dominant influence of the effective self-weight of the surrounding ground and its ease and reliability of determination, the vertical effective overburden stress σ'_v can be incorporated into equation (22.8) as follows:

$$\frac{\tau_{sf}}{\sigma'_v} = \frac{\sigma'_{rf}}{\sigma'_v} \tan \delta'_f = K_{sf} \tan \delta'_f = \beta \tag{22.9}$$

where K_{sf} is the earth pressure coefficient on the shaft at failure. Burland (1973) pointed out that the parameter β can be obtained empirically by direct measurement of τ'_{sf} in pile tests (usually giving mean values $\bar{\beta}$). When determined in this way, β is analogous to the adhesion factor α in equation (22.7) but is much more rigorously defined, since σ'_v can be determined accurately and unambiguously, which we have seen is not the case for the undrained strength S_u. However, not only is β more rigorously defined empirically, but it can be evaluated by measuring δ'_f and determining K_{sf} from soil mechanics principles. Thus the parameter β is a much more clearly defined and fundamental parameter than α is.

22.4.2 Predicted shaft friction of an 'ideal' pile in normally consolidated clay

In this section we investigate the shaft resistance of an 'ideal' pile with a view to establishing a basic frame of reference for comparison with the measured behaviour of actual piles. An 'ideal' pile is one which has been installed in the ground without changing the *in situ* stresses. Moreover, it is assumed that shearing takes place through the remoulded ground next to the pile shaft so that $\delta'_f = \varphi'_{cv}$, the critical state angle of shearing resistance. Hence, for an 'ideal' pile equation (22.9) becomes:

$$\frac{\tau_{sf}}{\sigma'_v} = K_o \tan \varphi'_{cv} = \beta_{ideal} \tag{22.10}$$

where K_o is the earth pressure coefficient at rest – see Chapter 15 *Groundwater profiles and effective stresses*. For a normally consolidated clay, the value of K_o is accurately given by the Jaky expression $(1 - \sin \phi'_{cv})$. It follows that for a normally consolidated clay, equation (22.10) for an 'ideal' pile becomes:

$$\beta_{ideal} = (1 - \sin \varphi'_{cv}) \tan \varphi'_{cv} \ . \tag{22.11}$$

As shown in **Figure 22.8**, whereas K_o and $\tan \phi'_{cv}$ vary widely as ϕ'_{cv} changes, their product (β_{ideal}) changes very little, varying between 0.21 and 0.29 for the range of practical ϕ'_{cv} values. Equation (22.11) provides a basic framework for ideal pile behaviour in normally consolidated clays. Its validity can be tested against real pile behaviour and as a consequence it is possible to judge the extent to which the assumptions that $K_{sf} = K_o$ and $\delta'_{sf} = \phi'_{cv}$ need to be adjusted to account for the measured behaviour.

22.4.3 Comparison between measured and ideal values of β for normally consolidated clays

Burland (1973) compared this 'ideal' behaviour with careful experiments on some driven piles in two very different

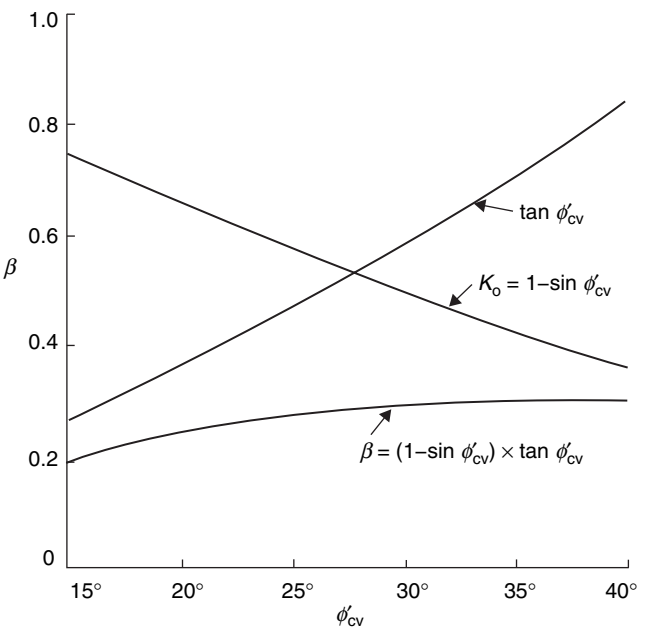


Figure 22.8 Relationship between the effective stress shaft friction parameter β and ϕ'_{cv}
Reproduced from Burland (1973); Emap

normally consolidated clays. **Figure 22.9** shows a plot of deduced average shaft friction $\overline{\tau}_{sf}$ versus average effective overburden pressure $\overline{\sigma}'_{vo}$. All except one of the results plotted in this figure are from tests described by Hutchinson and Jensen (1968) on a number of concrete, steel and timber piles driven into deep estuarine clay in the Port of Khorramshahr, Iran. The average liquid and plastic limits (*LL* and *PL*) for the clay are 48% and 23% respectively, and it has a sensitivity of between 2.5 and 3.0. The deduced values of α ranged from 0.43 to 0.79. It can be seen from **Figure 22.9** that the deduced average values of $\overline{\beta}$ lie between 0.25 and 0.4, with an average value of 0.32. Eide *et al.* (1961) published the results of some long-term tests on a timber pile driven into Drammen clay. The average liquid and plastic limits for this material are 35% and 15% respectively, and the clay has a sensitivity of between 4 and 8. The deduced value of α was 1.6. The black point in the figure corresponds to a deduced value of $\overline{\beta}$ equal to 0.32.

In spite of the clay at the two sites having very different properties and the deduced values of α for the two sites having extreme upper and lower limits for soft clay ranging from 1.6 to 0.42, the average values of $\overline{\beta}$ are the same. This average measured value of 0.32 is slightly larger than the average value for an 'ideal' pile of about 0.27 (corresponding to $\phi'_{cv} = 25°$ in equation (22.10)). Burland (1973) went on to examine a large number of results for driven piles in normally consolidated soils and found that the results lay between $\overline{\beta} = 0.25$ and 0.4. He suggested that a reasonable average design value would be 0.3, a little higher than the value of 0.27 given by an 'ideal' pile.

For a real pile driven into a normally consolidated sediment, it is reasonable to expect that the radial effective stresses would be somewhat larger than the 'at rest' pressures given by K_o, leading to somewhat higher measured values of $\overline{\beta}$. Hence, for the case of most normally consolidated clays, the simple ideal effective stress approach is seen to give very reasonable results. Results from fully instrumented tests on driven piles in a normally consolidated clayey silt give values of β of less than 0.2 (Burland, 1993). Such exceptionally low values are strongly suggestive of arching around the pile due to the highly contractive behaviour of the material.

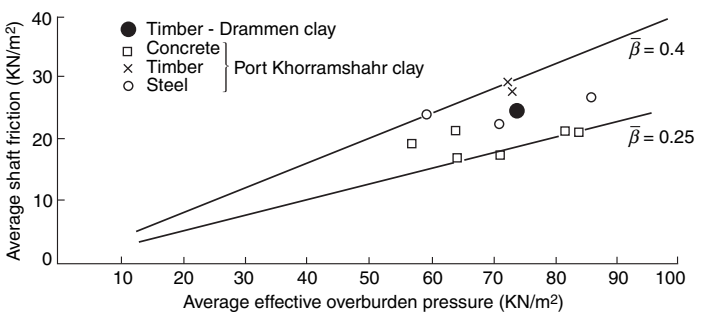


Figure 22.9 Comparison between results of pile tests on Port Khorramshahr clay (*LL* = 48; *PL* = 23; sensitivity = 2.5–3.0; α = 0.43–0.79) and Drammen clay (*LL* = 39; *PL* = 20; sensitivity = 4–8; α = 1.6)
Reproduced from Burland (*1973*); Emap

22.4.4 Negative friction on piles in soft clay

Negative friction, or 'downdrag' can occur on the shaft of a pile installed through a compressible layer of clay which undergoes consolidation. Such consolidation may result from the placing of fill on the surface, groundwater lowering, or even the dissipation of excess pore water pressures set up during pile driving. The consolidating layer exerts a downward drag on the pile and therefore the direction of shaft friction in the layer is reversed. Negative skin friction usually develops gradually as consolidation of the clay proceeds and the vertical and horizontal effective stresses increase with time due to the dissipation of pore water pressure.

Burland and Starke (1994) analysed ten case records in which negative friction had been measured. The results were analysed in terms of the effective shaft friction factor β. The causes of negative friction included surface surcharge, underdrainage and disturbance during pile driving. Both steel- and concrete-driven piles were considered, some of which were end bearing and others 'floating'. The period of measurement varied from a few months up to 17 years.

In spite of the wide range of conditions, average values of β for soft compressible sediments were found to lie within relatively narrow limits of 0.15 to 0.35. A careful study of the soil types revealed that for low plasticity marine clays the values of β were generally in the range 0.15 to 0.25, whereas for higher plasticity clays and silty clays β tended to lie between 0.2 and 0.35. It therefore appears that the values of β operating for negative friction are similar but somewhat lower than those associated with driven-friction piles in soft clays.

22.4.5 Predicted shaft friction of an 'ideal' pile in overconsolidated clay

There can be no doubt that the effective stress approach to shaft friction for piles in soft clays has major advantages over the traditional α approach with all its uncertainties. This is because the coefficient of earth pressure at rest K_o in equation (22.10) is well defined for normally consolidated clays. For overconsolidated clays, the determination of the *in situ* horizontal effective stresses requires very sophisticated *in situ* or laboratory testing, and the effective stress approach is therefore not nearly so straightforward. Nevertheless, establishing the behaviour of an 'ideal' pile in overconsolidated clay provides very useful insights into the factors controlling its behaviour.

As pointed out in Chapter 15 *Groundwater profiles and effective stresses*, for overconsolidated clays which have previously been subjected to a maximum vertical effective stress $\sigma'_{v,\,max}$, the horizontal effective stress caused on first loading tends to remain 'locked in' on unloading, resulting in an

increase in K_o. A widely used empirical equation for capturing this was developed by Mayne and Kulhawy (1982):

$$\frac{\sigma'_h}{\sigma'_v} = K_o = (1 - \sin\varphi'_{cv}) \times OCR^{\sin\varphi'_{cv}} \quad (22.12)$$

where OCR is the overconsolidation ratio (= $\sigma'_{v,max}/\sigma'_{v,current}$). Note that when OCR = 1, equation (22.12) reduces to the Jaky expression $K_o = (1 - \sin\phi'_{cv})$. Substituting equation (22.12) into equation (22.10):

$$\frac{\tau_{sf}}{\sigma'_v} = \beta_{ideal} = (1 - \sin\varphi'_{cv}) \times OCR^{\sin\varphi'_{cv}} \times \tan\varphi'_{cv} \ . \quad (22.13)$$

Hence it would seem appropriate to investigate the relationship between β and OCR. However, since the determination of OCR requires high quality sampling and oedometer testing, there is little available pile test data with which to test equation (22.13). What is needed is some simple indirect measure of OCR. It is well known (Jamiolkowski *et al.,* 1985) that the ratio (S_u/σ'_v) is related to OCR by an expression of the form:

$$S_{utc}/\sigma'_v = (S_{utc}/\sigma'_v)_{nc} \times OCR^m \ . \quad (22.14)$$

where S_{utc} is the undrained strength measured in an unconsolidated undrained triaxial compression test and $(S_{utc}/\sigma'_v)_{nc}$ refers to normally consolidated conditions where m is an empirical coefficient. Solving for OCR in equation (22.14) and substituting in equation (22.13) gives:

$$\beta_{ideal} = (1 - \sin\varphi'_{cv}) \times \tan\varphi'_{cv} \times \left[\frac{S_{utc}/\sigma'_v}{(S_{utc}/\sigma'_v)_{nc}}\right]^{\frac{\sin\varphi'_{cv}}{m}} \quad (22.15)$$

Although equation (22.15) looks rather complicated, it suggests that there should be a relationship between β_{ideal} and S_{utc}/σ'_v. To explore this relationship, equation (22.15) may be considerably simplified as follows.

The term $(1 - \sin\phi'_{cv}) \times \tan\varphi'_{cv}$ equals β_{ideal} for a normally consolidated clay, and we have seen that β_{ideal} changes very little with changes in ϕ'_{cv}. It is reasonable to set it equal to 0.27, corresponding to $\phi'_{cv} = 25°$. It is well known (Burland, 1990) that for normally consolidated natural clays the ratio $(S_{utc}/\sigma'_v)_{nc}$ is remarkably well defined, generally varying between about 0.28 and 0.32; a mean value of 0.3 can reasonably be adopted. A major uncertainty exists over the likely range of m and hence equation (22.15) can be simplified to:

$$\beta_{ideal} = 0.27\left[\frac{S_{utc}/\sigma'_v}{0.3}\right]^E \quad (22.16)$$

where E is an empirically determined exponent. This work suggests that there is likely to be a relationship between the effective shaft friction factor β and the undrained compressive strength normalised by the effective overburden pressure S_{utc}/σ'_v.

22.4.6 Measured values of β for driven piles in overconsolidated clays

Flaate and Selnes (1977) published the results of a number of pile tests on timber and concrete piles driven into normally and lightly overconsolidated clays. Semple and Rigden (1984) assembled an extensive database for piles driven into overconsolidated clays which formed the basis of guidance by the American Petroleum Institute for many years. After following the reasoning described in the previous section, Burland (1993) plotted these two sets of data on a graph of β versus S_{utc}/σ'_v and demonstrated that there is a very satisfactory correlation. Chow (1996) added significantly to these two databases and Patrizi and Burland (2001) showed that the effective shaft friction factor could be accurately represented by the expression:

$$\beta = 0.1 + 0.4(S_{utc}/\sigma'_v) \ . \quad (22.17)$$

The solid line in **Figure 22.10** shows the comparison between the results from the three databases and equation (22.17). The insert shows the data points for values of $S_{utc}/\sigma'_v < 1$ plotted to a larger scale. (It is of interest to note that the broken line plotted in this figure is given by equation (22.16) for an 'ideal' pile with the empirical exponent $E = 0.72$). The open points in **Figure 22.10** are for low plasticity clayey silts and it can be seen that there is a tendency for the values of β to be somewhat lower for these materials – a result referred to previously in sections 22.4.3 and 22.4.4.

It is necessary to mention that the values of S_{utc} are determined for intact samples of clay. If the clay is fissured, the values of undrained strength will not be representative of the intact material and are likely to be significantly lower than the intact strengths. When plotting results from tests, this leads to points lying significantly above the line given by equation (22.17) and when using this equation for predicting design values of β, they are likely to be significantly underestimated.

Patrizi and Burland (2001) demonstrated that the correlation between equation (22.17) and the results of the database is very good. An even better correlation can be obtained using the Imperial College Pile (ICP) design methods developed by Jardine *et al.* (2005), which are based on instrumented pile tests driven into both sands and clays. These approaches have been developed for the offshore industry and require advanced laboratory and *in situ* tests.

22.4.7 The effective stress behaviour of bored piles in stiff fissured clays

Burland (1973) developed the following expression for the average shaft friction $\overline{\tau}_{sf}$ for an 'ideal' pile in overconsolidated clay, which takes account of the variation of K_o with depth:

$$\overline{\tau}_{sf} = \frac{1}{L}\sum_0^L \sigma'_v \times K_o \times \tan\delta'_f \times \Delta L \quad (22.18)$$

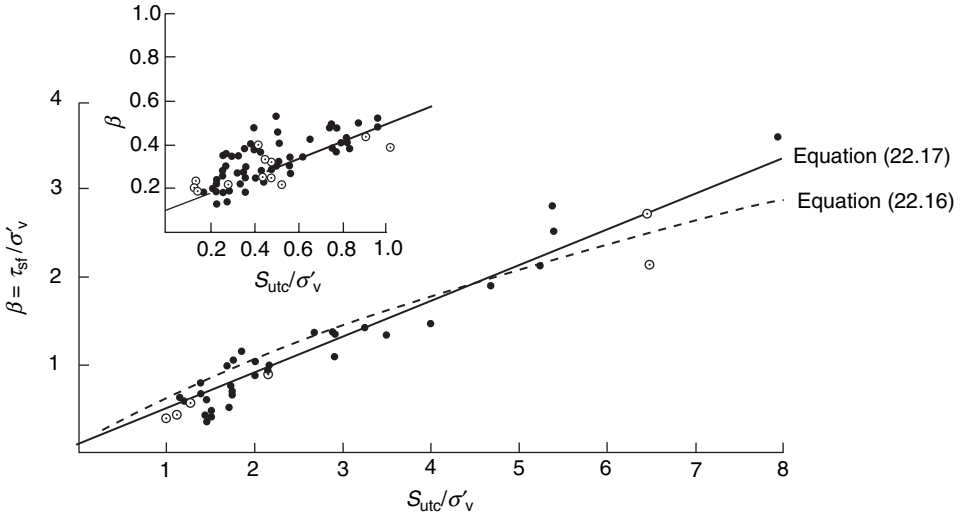


Figure 22.10 Driven piles in clay - relationship between the effective stress shaft friction parameter β and S_{utc}/σ'_v where S_{utc} is the *intact* unconsolidated undrained strength in triaxial compression. (This correlation is not appropriate for fissured clays for which S_{utc} is not representative of intact strength)

For London Clay, the variation of K_o with depth was determined by Skempton (1961) and Bishop *et al.* (1965) and an average distribution adopted. The assumption for an 'ideal' pile that $\delta'_f = \phi'_{cv} = 21.5^o$ was also made. **Figure 22.11** shows a plot of average shaft friction τ_{sf} versus average depth in clay. The full line was obtained from equation (22.18) and represents the predicted values of average shaft friction of 'ideal' piles in London Clay.

The open points plotted in **Figure 22.11** represent the average measured shaft friction deduced from maintained load tests on instrumented piles carried out at Wembley by Whitaker and Cooke (1966), previously referred to in section 22.2.4. It can be seen that, apart from one outlier, the results show relatively little scatter and give measured values of average shaft friction which are less than the 'ideal' values. Burland (1973) attributed this to the effect that the boring of the pile had on reducing the equilibrium value of the horizontal effective stress to below the 'at rest' K_o condition assumed for the 'ideal' pile.

Burland and Twine (1988) re-examined the Wembley data and concluded that the main reason for the measured values of $\overline{\tau}_{sf}$ being less than the 'ideal' values was that the angle of interface friction δ'_f was close to the residual angle of shearing resistance ϕ'_r, having values as low as 12°. Various causes for this were suggested – the two most probable being (i) the upward and downward passage of the auger causing a smooth surface of highly orientated clay particles; and (ii) thermal shortening of the shaft of only a few millimetres would be sufficient to induce a residual shear surface over much of the length of the pile. Burland and Twine also concluded that constant rate of penetration (CRP) testing must be interpreted with considerable caution, as current rates of testing can lead to a significant over-estimation of the shaft resistance under static load.

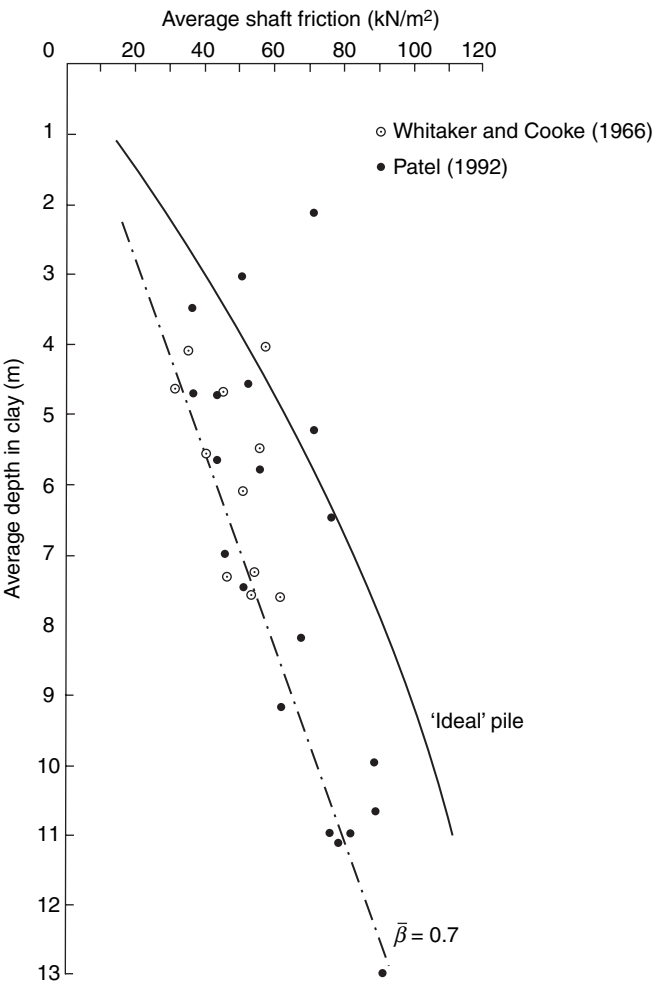


Figure 22.11 Relationship between measured average shaft friction and average depth in clay for bored piles in London Clay

Patel (1992) analysed a number of load tests on bored piles from 28 different sites in London. Maintained load tests were carried out at 16 of these sites and the shaft resistances were evaluated by subtracting the calculated base resistances from the ultimate resistance of the pile. The full points in **Figure 22.11** show the deduced values of $\bar{\tau}_{sf}$. Once again, apart from two outliers at shallow depth, the scatter is remarkably small. The chain dotted line representing a value of $\bar{\beta} = 0.7$ is close to the lower limit of the scatter and would be appropriate for design purposes for bored piles in London Clay (i.e. $\bar{\tau}_{sf} = 0.7 \times \bar{\sigma}'_v$). Patel (1992) also confirmed the conclusion of Burland and Twine (1988) that the use of CRP tests can overestimate the true values of shaft friction by up to 20% due to rate effects.

The use of **Figure 22.11** for assessing shaft friction in terms of effective stress for bored piles in London Clay offers a very simple and dependable method which is based on highly reliable load tests. It can provide a valuable check when using the traditional α method described in section 22.3.3, or when interpreting the results of pile tests. The approach is particularly appropriate for assessment of shaft resistance where long-term changes in effective stress will occur, as is the case beneath a deep excavation or where changes in ground water level are taking place. The traditional α method of assessing shaft friction is completely inappropriate for such conditions.

More generally, when analysing the results of pile tests in clay, it is worthwhile plotting the deduced values of $\bar{\tau}_{sf}$ against average depth in clay and evaluating the associated values of $\bar{\beta}\,(= \bar{\tau}_{sf}/\bar{\sigma}'_v)$. In this way, the basic data are allowed to speak for themselves without the confusion caused by correlating the results with average undrained strength and all the uncertainties that are associated with it. Bown and O'Brien (2008) have shown that when reliable methods are available for measuring σ'_{ho} (namely the *in situ* horizontal effective stress), equation (22.8) can be applied directly. They proposed that the average shaft friction of a bored pile in stiff plastic clay can be expressed as:

$$\bar{\tau}_{sf} = c.\bar{\sigma}'_{ho} \tan \delta'_f \qquad (22.19)$$

where c is a reduction factor for installation effects with values varying typically from 0.9 to 0.8, and decreasing with depth. For London Clay, a value of effective angle of interface friction $\delta'_f = 16^\circ$ was found to be appropriate.

22.5 Piles in granular materials

Piles in granular soils usually act primarily in end bearing and, unlike many piles in clay, shaft friction usually contributes only a small proportion of the load-carrying capacity. It is extremely difficult to obtain reasonably undisturbed samples of granular soils from boreholes. Moreover, the process of driving the pile will inevitably change the density of soil which will significantly affect the carrying capacity of both the base and the shaft. These facts, in addition to the inherent variability of granular deposits, make the prediction of pile behaviour by analytical methods both difficult and unreliable. It is therefore the usual practice to assess the ultimate bearing capacities of piles in these soils from the results of *in situ* tests.

Carrying capacity can be assessed on the basis of:

- bearing capacity theory aided by standard penetration tests (SPTs);
- cone penetrations tests (CPTs);
- pile driving formulae.

22.5.1 Bearing capacity theory

Equation (21.4) in Chapter 21 *Bearing capacity theory* gives the basic bearing capacity equation for a granular soil. For the tip of a pile the self-weight term involving N_γ becomes negligibly small. Moreover the shape and depth factors, s_q and d_q respectively, may be subsumed into the bearing capacity factor for surcharge N_q. Hence the ultimate base resistance of a pile in embedded a granular soil becomes:

$$q_{bf} = N_q \times \sigma'_v \,. \qquad (22.20)$$

A wide range of values for N_q are quoted in the literature, but perhaps the most generally accepted are those given by Berezantzev *et al.* (1961). **Figure 22.12** shows their proposed relationships between N_q (which include s_q and d_q) and the effective angle of shearing resistance ϕ' adapted by Tomlinson (1995). If the depths of penetration of the pile into the granular material are less than about five times the pile width, values of N_q corresponding to a surface footing should be used – see Chapter 21 *Bearing capacity theory*.

Evaluation of N_q in equation (22.20) requires knowledge of the effective angle of shearing resistance of the soil ϕ'. This is often done by making use of the well-known relationship

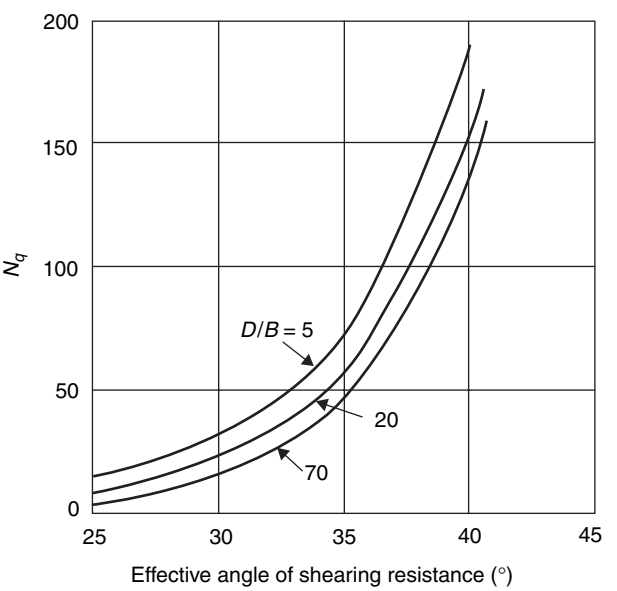


Figure 22.12 Relationship between N_q and ϕ'
Reproduced from Berezantsev *et al.* (1961); all rights reserved

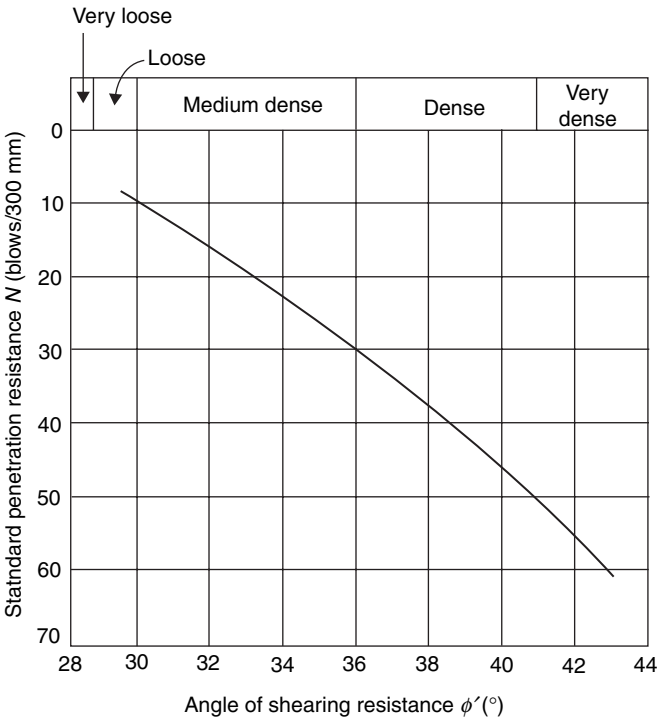


Figure 22.13 Relationship between SPT *N*-values and ϕ'
Reproduced from Peck *et al.* (1967); John Wiley & Sons, Inc.

between SPT *N*-value and ϕ' proposed by Peck *et al.* (1967), illustrated in **Figure 22.13**.

The expression for the ultimate shaft friction τ_{sf} is identical to equation (22.8) and can be re-written:

$$\tau_{sf} = \sigma'_v \times K_{sf} \times \tan \delta'_f \qquad (22.21)$$

where K_{sf} is the earth pressure coefficient against the pile shaft at failure and δ'_f is the angle of interface friction along the shaft.

There are various rules for evaluating δ'_f in equation (22.21). For a concrete or rough steel interface it appears that $\delta'_f \approx \phi'_{cv}$. For smoother interfaces, Jardine *et al.* (1992) conclude that δ'_f is a function of mean particle size (D_{50}) and varies from about 34° to 22° as D_{50} increases from 0.05 mm to 2.0 mm.

The evaluation of K_{sf} in equation (22.21) presents a real problem as it depends largely on the initial density of the soil, the pile installation method, and the volume of the displaced soil. Broms (1965) suggested the following values of K_{sf} in granular soils:

Type of pile	Loose	Dense
Steel	0.5	1.0
Concrete	1.0	2.0
Timber	1.5	3.0

Fleming *et al.* (2009) argue that the value of K_{sf} varies in a similar fashion to N_q and may be estimated from the expression

$$K_{sf} = N_q/50 \ . \qquad (22.22)$$

22.5.2 Methods based on static cone penetration tests

Methods based on CPTs are widely used for the design of offshore piles and their use on land is increasing. The electric cone provides a profile with depth of end resistance q_c and sleeve friction f_s which can be used for the design of driven piles. A brief description of the equipment is given by Simons *et al.* (2002).

Where the pile is deeply embedded in the granular layer, the tip of the cone may be regarded as a model of the base of the pile. Hence the base resistance of the pile q_{bf} is taken as equal to the cone resistance q_c. However the cone normally gives a fluctuating resistance with depth and requires careful interpretation. The currently accepted method of taking account of these fluctuations in deriving q_{bf} for a deeply embedded pile is given by Thorburn and Buchanan (1979) as follows:

$$q_{bf} = 0.25q_{co} + 0.25q_{c_1} + 0.5q_{c_2} \qquad (22.23)$$

where:

q_{co} = the average static cone resistance over a distance of two pile diameters below the pile base;

q_{c1} = the minimum cone resistance over the same distance below the pile base;

q_{c2} = the average of the minimum cone resistances over a distance above the pile base, neglecting any values greater than q_{c1}.

In situations where the cone resistance decreases to depths below the pile base greater than or equal to 3.5 pile diameters, the following expression gives a reasonable prediction of the ultimate base resistance:

$$q_{bf} = 0.5q_{cb} + 0.5q_{ca} \qquad (22.24)$$

where

q_{cb} is the average cone resistance over a distance below the pile base equivalent to 3.5 pile diameters and is determined from the following expression:

$$q_{cb} = \frac{q_{c1} + q_{c2} + q_{c3} + \ldots q_{cn}}{2n} + n.q_{cn} \ . \qquad (22.25)$$

q_{ca} is the average cone resistance over a distance above the pile base equivalent to 8 pile diameters, neglecting any values greater than q_{cn};

q_{c1}, q_{c2}, q_{c3}, … q_{cn}, are the cone resistances at regular intervals to a depth of 3.5 pile diameters below the pile base, q_{cn} being the lowest resistance at this depth.

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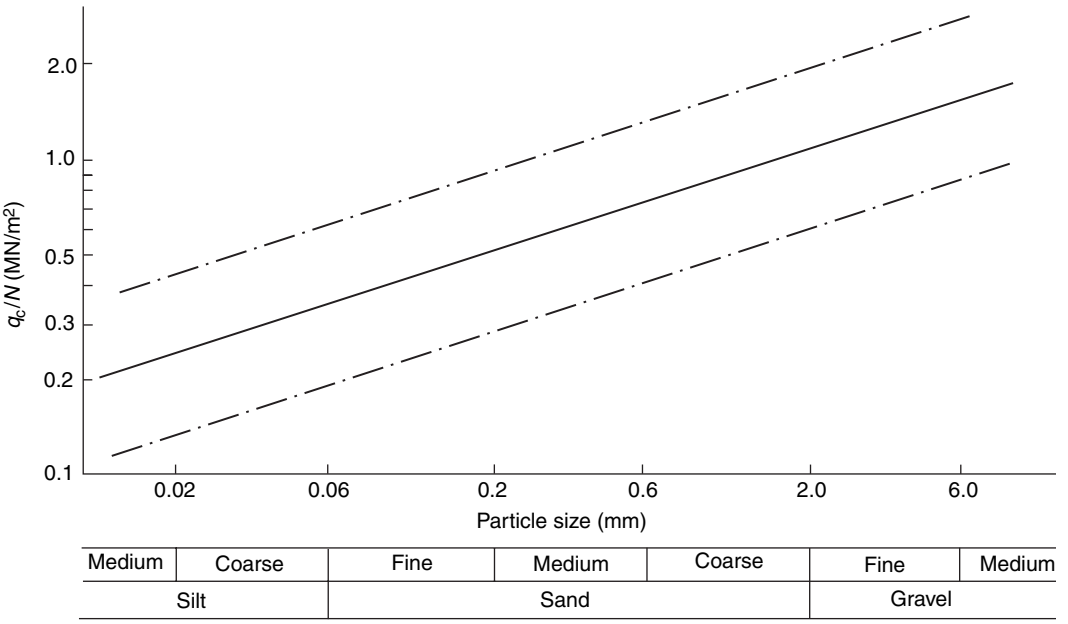


Figure 22.14 Relationship between q_c/N and grain size
Reproduced from Burland and Burbidge (1985)

The compressed rules given by Beringen *et al.* (1979) are broadly in agreement with the above.

De Beer *et al.* (1979) present some case histories of piles penetrating only a small distance into dense sands. They show that for small penetrations the end resistance of a pile can be significantly less than that given by the cone resistance. Various methods have been proposed to make allowance for this important scale effect. The method proposed by de Beer (1963) is widely used but is believed to be very conservative.

The results of CPTs are regularly used for predicting shaft friction τ_{sf}. This is usually taken as some fraction of the cone tip resistance q_c. Thorburn (1979) recommends that for sands $\overline{\tau}_{sf} = q_{c(ave)}/200$ where $q_{c(ave)}$ is the average point resistance over the embedded length of the pile. For silts, he recommends that $\overline{\tau}_{sf} = q_{c(ave)}/150$. For driven H-piles in granular materials, Meyerhof (1976) recommends $\overline{\tau}_{sf} = q_{c(ave)}/400$.

22.5.3 Methods based on standard penetration tests

In section 22.5.1, reference was made to the use of SPTs for evaluating the angle of shearing resistance ϕ' for use in the bearing capacity equation (22.20). Some approaches to pile design are based on direct empirical correlations between SPT blow count N and allowable end bearing and shaft friction – see, for example, GEO (1996) for bored piles in completely decomposed granite in Hong Kong. It is most important that the basis of the correlations is understood when using them, and that the piling method and ground conditions are appropriate.

When CPT tests are not available, a reasonable approach is to convert the SPT N values to equivalent cone resistance q_c using the graph in **Figure 22.14** (Burland and Burbidge, 1985). The methods outlined in section 22.5.2 can then be applied.

22.5.4 Pile driving formulae

Because of the uncertainties in predicting the carrying capacity of piles driven into granular materials, it has long been recognised that the driveability of a pile should provide a useful measure of its static carrying capacity. Moreover, in variable ground conditions, the driving record of each pile should be a good indicator of its quality. It is therefore not surprising that many attempts have been made to determine the relationship between the dynamic resistance of a pile during driving and the static load carrying capacity of the pile.

Whitaker (1975) has carried out a detailed review of a number of the better known pile driving formulae. The purpose of this section is to provide a brief introduction to the subject. Driving formulae assume an energy balance between the dynamic input energy (= $\eta.W.h$) of the hammer and the work (= $R(s+c/2)$) required to displace the pile permanently by a small amount s, where:

η = the efficiency of the hammer;

W = the weight of the hammer;

H = the drop height;

R = the pile resistance;

s = the permanent set of the pile;

c = the elastic, or recoverable movement of the pile.

Hence:

$$R = \frac{\eta.W.h}{(s + c/2)} . \qquad (22.26)$$

According to the widely used Hiley formula of 1925 the efficiency is given by:

$$\eta = \frac{k(W + e^2 W_p)}{W + W_p} \tag{22.27}$$

where

k = the output efficiency of the hammer;

W_p = the weight of the pile;

e = the coefficient of restitution between the hammer and the pile cushion.

Typical values of k and e are given by Whitaker (1975), and Fleming *et al.* (2009). In applying any pile driving formula, the fundamental assumption is made that the driving resistance R is equal to the ultimate static bearing capacity of the pile. This assumption can be significantly in error, particularly in clay soils, fine granular soils and cemented soils such as chalk. Whitaker (1975) and Tomlinson (1995) give valuable guidance on the practical application of pile driving formulae and more sophisticated forms of dynamic analysis.

22.5.5 Concluding remarks on piles in granular materials

The present state-of-the-art on the prediction of pile capacity in granular soils is such that pile loading tests should be carried out, if at all possible. Driving resistance records on all piles should be kept (blows per 0.2 m or 0.5 m penetration). These can be useful, particularly when correlated with loading tests at the site. Beware of pile heave due to adjacent pile driving – this is particularly important with saturated fine grained granular soils, i.e. silts. It may be necessary to re-drive some of the piles.

22.6 Overall conclusions

- The installation of a pile is a geotechnical process, controlled by an operator, the outcome of which depends on numerous factors. In these circumstances it is not reasonable to expect to be able to predict pile behaviour with great precision.
- Methods of prediction of pile behaviour are inevitably highly empirical. It is therefore important to seek to understand the basis of any empirical factor or correlation that is used, and the likely range of prediction.
- The load–settlement behaviour of a pile results from the summation of two very different mechanisms. The shaft resistance is essentially frictional and is fully mobilised at small settlements – typically about 0.5% of the shaft diameter or 5–10 mm. The behaviour of the shaft is usually ductile in the sense that little reduction in carrying capacity takes place, with continued settlement after full mobilisation. The full mobilisation of the base resistance requires much larger settlements than the shaft – typically 10–20% of the base diameter. Having a clear understanding of these two mechanisms of load transfer is very helpful when interpreting the results of pile loading tests.
- Basic undrained bearing capacity theory can be used to calculate the base resistance of a pile in clay. Usually the average undrained strengths from unconsolidated undrained triaxial compression tests (S_{uc}) are used for this purpose. In stiff fissured clays, for which the undrained strengths usually show considerable scatter, the *in situ* operational strength is close to the lower limit of the scatter and should be used for design purposes.
- Traditionally, the measured shaft friction τ_{sfh}' of piles in clay has been correlated with undrained strength (i.e. $\tau_{sf} = \alpha.S_u$) for the simple reason that undrained strength data are usually included with pile test results. This approach has serious limitations. It has been shown that the undrained strength of a particular soil is not a unique quantity and can show considerable scatter. The value of α has been shown to vary over wide limits. Frequently published design curves relating α to undrained strength do not include the enormous scatter on which they are based. It is essential that the engineer is fully aware of these limitations as they are frequently glossed over.
- It is because of these serious limitations that much of this chapter has been devoted to the determination of shaft friction in terms of effective stress. It is now clear beyond all reasonable doubt that shaft friction is controlled by the simple effective stress Coulomb sliding law given by equation (22.8). This equation can be normalised by the vertical effective stress and re-written as:

$$\frac{\tau_{sf}}{\sigma_v'} = \frac{\sigma_{rf}'}{\sigma_v'} \tan\delta_f' = K_{sf} \tan\delta_f' = \beta \,. \tag{22.28}$$

- The effective stress shaft friction parameter β can be obtained empirically from pile tests and is much more rigorously defined than α, since σ_v' can usually be accurately determined. The value of β can also be obtained from fundamental soil mechanics principles to give 'ideal' values, which can be used for judging the influence of various installation and operational processes.
- It has been shown that for driven piles in normally consolidated clays, measured values of $\overline{\beta}$ are well defined, lying between 0.25 to 0.4 – a little larger than the 'ideal' values.
- The effective stress approach is really the only viable approach to evaluating negative friction or 'down drag'. It has been shown that local values of β lie between 0.15 and 0.25 for low plasticity clays, whereas for high plasticity clays the values lie between 0.2 and 0.35.
- An approach is outlined for applying the effective stress method to overconsolidated clays when values of K_o are not known. It is shown that a theoretical relationship exists between β and S_{utc}/σ_v', where S_{utc} is the *intact* undrained strength in unconsolidated triaxial compression. Three large databases for driven piles in clay have been used to show that there exists a simple approximately linear relationship between β and S_{utc}/σ_v'.
- The effective stress method has been applied to bored piles in London Clay for which the distribution of K_o with depth is approximately known. The 'ideal' curve has been compared with a number of incremental load tests from various parts of London. The measured values are significantly less than the ideal values – the reason is believed to be that the angle of interface friction for traditional bored piles in London Clay operates close to residual

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strength rather than critical state strength, which was assumed in the 'ideal' case.

- For bored piles in London Clay, the simple effective stress relationship $\bar{\tau}_{sf} = 0.7 \times \bar{\sigma}'_v$ is close to the lower limit of the scatter in **Figure 22.11** and is appropriate for design purposes. More generally, it is argued that there is a strong case for presenting the results of tests on bored piles in stiff clays in terms of average shaft resistance versus depth, with the appropriate values of $\bar{\beta}$ drawn in.
- Piles in granular material act primarily in end bearing. Because of the difficulty of obtaining undisturbed samples of granular soils, considerable reliance is placed on the use of *in situ* tests for evaluating the end and shaft resistance. The standard empirical methods of applying the results of CPTs and SPTs to pile design are described.
- A brief introduction is given to the application of pile driving formulae and their limitations.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

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Chapter 23
Slope stability

Edward N. Bromhead Kingston University, London, UK

This chapter explores the basics of slope stability and how relative stability is calculated using a limit equilibrium approach. It discusses the principles of slope stabilisation as they impact on the analysis required, and makes recommendations regarding the management of slope instability risks within the context primarily of the analysis.

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23.1 Factors affecting the stability and instability of natural and engineered slopes

23.1.1 Introduction

It is customary to consider the problems of natural slopes as being distinct from those of engineered slopes, and in the latter to discriminate between cut (excavated) slopes and filled slopes, despite many of the technical issues being the same. Filled slopes give the engineer the option of controlling the material properties of the soils involved as well as controlling the shape of the resulting slope, whereas in both cut and natural slopes the geological succession and resulting properties are whatever nature presents. In cut slopes, one at least has the possibility of forming them to a desired shape, but in natural slopes, again, the engineer has often to deal with what is there.

Engineered slopes are created in connection with urban development, transportation routes, dams, mining and quarrying, municipal waste disposal and numerous other construction activities requiring the formation of excavations, and in building mounds. Where the available space does not permit stable slopes to be created, retaining structures are required (Section 6 of this manual).

Natural slopes present stability problems associated with contemporary coastal and river erosion; the erosion processes cause oversteepening. In addition, the range of processes that form hills and valleys commonly leave hill slopes in a state where they can be readily destabilised. For example, extremes of rainfall or modest earthworks may cause hitherto dormant soil masses that were at one time unstable to move again.

Many engineers would further distinguish between rock slopes and soil slopes, where in the former the behaviour is characterised largely by failure primarily along pre-existing discontinuities such as joints and faults, and much less so through the intact material and *vice versa*. However, many soil slopes fail along bedding, or weak layers constructed into a fill, so that such a distinction is somewhat arbitrary. The rest of this chapter deals primarily with soils.

23.1.2 Factors giving rise to slope failure

Factors giving rise to slope failure comprise external and internal geometric factors, intrinsic properties of the materials involved, the presence of discontinuities and other structural weaknesses (which may in part be due to geological causes and reflect the processes by which the soils were formed or emplaced), the presence of fluids (air and water) in the soil, and processes of weathering or ageing – some of which relate to long-term fluid pressure changes. Much of the loading in a soil comes from self-weight, but some slopes may be subjected to external loads from foundations and anchorages, and stresses induced dynamically, of which the most obvious is seismic shaking.

External geometric factors include its height and slope angle, and its shape, both in section and plan. Internal geometric factors include the disposition of different material types inside the slope. It is extremely common to find that failure in an engineered slope occurs because a weak layer was left in place in an embankment within its foundation, or in a cut slope within the body of the soil mass. Natural deposits, particularly of sedimentary origin, often contain strata which are weaker than the surrounding soils, and failures occur along such weaker layers.

23.1.3 The soil strength model

In the majority of slopes, the most significant loading comes from the self-weight of the soil mass, and therefore its density (unit weight γ, or *weight density* according to Eurocode 7) is a significant factor. It is convenient to model the resistance to shear s using a simple c–ϕ effective stress-based relationship:

$$s = c' + (\sigma - u)\tan\phi' \qquad (23.1)$$

as for a small range of stresses, and with appropriately selected parameters, this permits the analysis to consider the self-weight and pore fluid pressure effects in the soil (these are usually due to water, so the phrase *pore water pressure* is taken as equivalent in the following), both from steady and unsteady seepage, and caused or modified by undrained stress changes in the soil mass. Experience shows that laboratory measured peak parameters are often poor choices for c' and ϕ', and reduced φ'-values (largely based on empiricism or experience) may be required.

Soils vary so much in their composition that it is rarely possible to identify values for c' and ϕ' merely from a soil description, although some rules of thumb are useful. For example, in granular soils such as sands and gravels, ϕ' is almost invariably greater than 30° and increases with density and particle interlock, so that granular soils with angular particles and a range of particle sizes are normally much stronger than those with rounded, non-interlocking, similar-sized particles. Micaceous sands may not exhibit the interlock phenomenon, and may be weaker. Mineral cements such as iron compounds and calcite may further increase ϕ', but their effect is greatest on c', which would otherwise (in granular soils) be negligible. Cements are unreliable if present to a small degree – decalcification, for example, can occur with weak acids from vegetation or acid rain.

Clay soils commonly exhibit ϕ' in a range from just below 20° to about 25°. Factors that increase ϕ' are, again, the content of cementitious minerals and the type of clay minerals present (which may be apparent through their plasticity). Most clays lose significant amounts of strength when sheared, leading to the formation in the soil mass of slickensided shear surfaces. The amount of strength degradation or brittleness varies widely, and is usually highest where the soils contain smectites and lowest where they contain kaolinites. The strength along slickensided shear surfaces may also be represented by a c–ϕ, effective stress-based relationship, but using different c–ϕ values for the sheared or *residual strength* condition. This is denoted by a subscript 'r'. Commonly, illite clays exhibit ϕ'_r in the range 9–15°, but smectites may be much lower, down to around 4°. Effective residual cohesion is usually negligible.

Silty soils fall in between the two limits depending on whether they are devoid of, or rich in, clay.

The stability of a saturated clay mass is dominated not by the mean strength, but by the strength of the weakest path through the mass. Such weaker zones may be created at the time of deposition or placement of the soil and are often the result of having less effective cohesion. Therefore, notwithstanding the existence of effective cohesion in many clays (and it is almost invariably shown up in effective stress shear strength tests), stability analyses routinely employ little or no effective cohesion.

All assessments of stability using effective stress-based shear strength rely on the correctness of the pore water pressures used in the analyses. Soils with low densities, particularly organic soils such as peat and municipal solid waste, are extremely sensitive to fluctuations in the pore water pressure, as in equation (23.1) the direct stress σ (which arises out of self-weight) may not be much greater than the pore water pressure u.

23.1.4 Undrained strength

Under some loading conditions – notably the application of loads without change in the moisture content of the soil, and for clay fills that are placed in an unsaturated state – it may be convenient to employ an undrained strength soil model. Usually, this is nominally a c–ϕ model, but ϕ_u is commonly small or zero, and is ignored. Strictly, the undrained strength is s_u, but the term c_u is used and the two are interchangeable.

When using the undrained strength, pore pressure effects (some of which are highly complex) are lumped in with the other strength parameters, and the analysis is done as though the soil is devoid of pore pressures. Needless to say, that simplifies the analysis.

Undrained strengths in some saturated soils are subject to degradation upon remoulding or reworking. This is termed sensitivity. Unlike drained brittleness, which is a phenomenon of the soil particles, sensitivity is commonly a pore water pressure effect, although that in turn comes from disruption to the soil fabric. Unsaturated soils obtain much of their undrained strength from capillarity, and this may be lost upon soaking or saturation.

Slopes with clay soils or clay strata tend to fail in the clays, and subsequent to failure, slickensided shear surfaces are left in the ground. These surfaces dominate the subsequent behaviour and stability of the slope as a whole, although local slope facets may remain where the peak or unsheared strength continues to control local stability.

23.2 Modes and types of failure commonly encountered

There are a number of classification and descriptive schemes for slope failures (Cruden and Varnes, 1996; Dikau *et al.*, 1996). The critical distinction between slope deformation and slope failure is that in the latter a mass of soil or rock is detached from the parent mass and moves independently, whereas slope deformation is movement of a part of the parent mass with no, or incomplete, detachment. The detached mass may stay in contact as it moves, or may lose contact. If the moving mass breaks into a few pieces, and it loses contact (the pieces may bounce or roll, or simply free fall until impact), then it is usual to describe the slope failure as a *fall* (e.g. rock fall). Where a fall breaks up into many pieces during movement, the mass tends to hug the topographic surface and exhibit characteristics we associate with fluid flow, and it is then described as a *flow* (e.g. debris flow, mud flow).

Slides occur when one (or a few) discrete blocks of soil and rock stay in contact with, although are clearly detached from, the parent mass. Slides may also degenerate into flows if they break up into many parts that move independently.

From the perspective of engineered slopes, falls are most commonly experienced from rock slopes, and result from

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weathering or water ingress into the slope – causing blocks to loosen and become detached along pre-existing discontinuities, with little or no failure through the intact rock mass. Falls of soil can, and do, occur from oversteepened temporary excavations such as trenches and pits.

Flows occur where the soil loses strength en masse. In loose granular soils, this can occur by vibration, rapid loading creating pore pressures, movement of water into or through the soil mass and deformation during sliding. Loose fills on sidelong ground are particularly susceptible to this mode of failure, as are soils placed hydraulically (e.g. from dredgings) where the failure may be termed liquefaction.

Slide failures are the most common type of failure, as they can occur on a range of slope angles including extremely low angled slopes, and are often the first stage of an event that turns into a flow or fall. Slide failures are particularly common where there is a pre-existing appropriately-orientated weakness in the slope. This can range from a fault, bedding or joints in a rock mass particularly where they dip out of the slope, to a weak (possibly organic) horizon in the foundation of an earth embankment. Such inappropriately orientated weaknesses are sometimes incorporated in earthworks by accident or neglect, including clay capping layers in municipal solid waste (MSW) facilities, or poorly prepared interfaces in clay fills.

Other modes of failure have been described in the relevant literature, but they occur chiefly in natural slopes, and are not commonly encountered.

23.3 Methods of analysis for slopes, exploring their limitations of applicability

23.3.1 Sliding analyses

Since many failures of slopes initiate with sliding, slope stability is usually assessed by means of a sliding model, employing a shear strength relationship of the type described above. The simplest of possible models employs a planar sliding surface, although this can be elaborated to cope with curved or compound (partly planar, partly curved) shapes, and is most often applied to a plane section through the slope. Occasionally, the three-dimensional shape of the slope and transverse curvature of potential sliding surfaces are taken into account. In rare cases, where the geology of a slope has demanded it, a kinked or dogleg section has been adopted. An analysis considering the collapse or ultimate limit state (giving rise to the epithet of 'limit equilibrium analysis') is usually sufficient for the analysis and design of routine engineered slopes and for the analysis of many natural slopes.

Continuum methods of analysis are used to explore the stress, strain and deformation conditions in slopes under their expected conditions of operation when the financial value of the slope itself (or the consequences of failure or underperformance) merits the additional costs of analysis and determination of parameters. Sometimes, such analyses are used to explore the conditions that led to failure in a *post hoc* or forensic investigation, as these methods must follow the correct stress path (or sequence of events).

Applied external forces such as live loads, anchors, etc., are simply in addition to, and often much less significant than, self-weight and pore pressure forces. More detailed treatment of the available methods is given by Bromhead (1992) and by Duncan and Wright (2005).

23.3.2 Planar sliding

Figure 23.1 shows a cross-section of a soil slope. If we consider the mass of soil of weight W above a planar surface A–A inclined at an angle to the horizontal of α, the component of this weight D acting down the plane A–A is:

$$D = W \sin \alpha \tag{23.2}$$

The component normal to the plane is $W \cos \alpha$, so that the maximum possible resistance R to sliding in the absence of water is

$$R = W \cos a \tan \phi' + c' A \tag{23.3}$$

where A is the area of contact between the block and the parent soil or rock mass. If there is a pore water pressure acting over the area of contact, then the force from the average pressure u multiplied by the area A must be taken into account, thus:

$$R = (W \cos a - uA) \tan \phi' + c' A = \tag{23.4}$$

Since the resistance is a reaction, and must therefore be equal to the applied force, we find that

$$D = W \sin \alpha = \frac{R}{F} = (W \cos \alpha - uA) \frac{\tan \phi'}{F} + \frac{c' A}{F} = \tag{23.5}$$

Note that if R is less than D, the mass of soil would accelerate under the unbalanced force $D-R$.

This equation could be used to determine the actual proportion of the shear strength parameters mobilised (i.e. to compute a factor of safety) or, by employing an inequality (<), to assure that the scenario is adequately stable when employing factored parameters.

23.3.3 Sliding on curved and compound surfaces

As soil slopes commonly fail on slip surfaces that combine curves and straights in a plane section, it is useful to consider the sliding mass to be made up from a series of vertical sided slices, each sliding down its own plane. (Non-vertical sided slices can also be employed, but vertical slices are much more common.) The precise number of slices is taken to ensure that the base of each is approximately planar. Since the above equation could be applied to each slice in turn to calculate a 'local' factor of safety, the factor of safety of the whole mass must lie between the maximum and minimum values so computed. Indeed, it is likely that the factor of safety of the whole is some sort of average of the individual values. An average would also take into account the tendency of some slices, if isolated, to

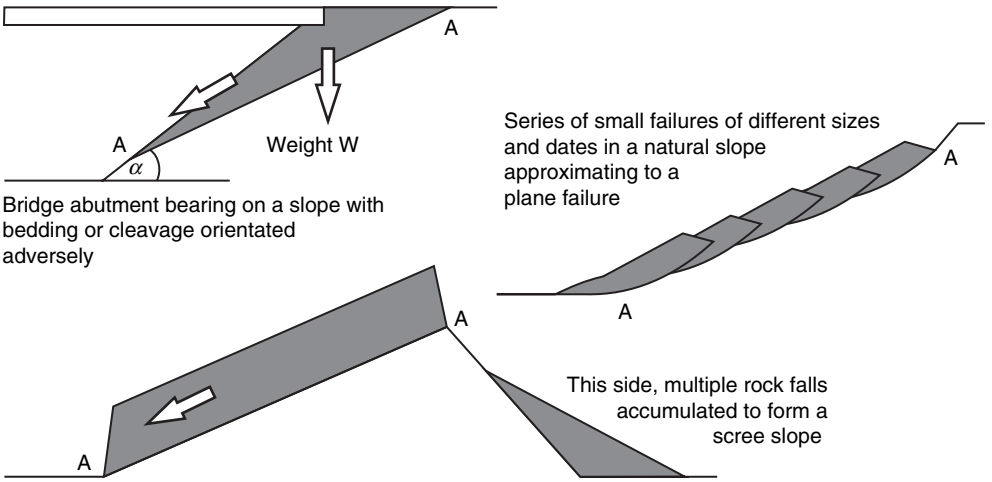


Figure 23.1 Examples of plane failure. The mass slides along plane A–A

slide in the opposite sense to the mass as a whole, especially in a toe zone that breaks out upwards.

A convenient averaging technique is to compute the sums of the driving (D) and resisting (R) terms, before dividing the latter by the former, since this takes into account implicitly the relative sizes and importance of the slices, and gives the overall factor of safety F as:

$$F = \frac{\sum_{i=1}^{n} \{(W\cos\alpha - uA)\tan\phi' + c'A\}_i}{\sum_{i=1}^{n} \{W\sin\alpha\}_i}. \qquad (23.6)$$

Note that this definition of F continues to be identical to the mobilisation factor on the shear strength parameters.

Consideration of the form of this equation shows that it reverts to the planar slide formula under the appropriate circumstances and is useful for shallow slip surfaces. However, if the slip surface is markedly curved, or descends deeply in the slope (especially when pore pressures are variable and high), the lack of computed interaction between the slices (i.e. the absence of inter-slice forces) makes the stress analysis change from merely approximate to highly dubious, and introduces errors which lead to a conservative estimate of F. Starting with an iterative procedure for dealing solely with the horizontal component of inter-slice forces published by Bishop (1955), efforts have been made to incorporate – as realistically as possible – the effects of inter-slice forces. One outcome of this is the publication of a multiplicity of named methods for the analysis of curved and compound sliding surfaces. Fortunately, provided that some attempts are made to model the inter-slice forces, the results do not appear to depend greatly on the precise technique used. Methods include using horizontal inter-slice forces (e.g. Bishop, 1955), inclined but parallel inter-slice forces (Spencer, 1967), and positioning the inter-slice force line of action at (say) 1/3 height on the slice. Morgenstern and Price (1965) introduced a procedure where the distribution of inter-slice forces is decided *a priori* by the analyst, but the precise magnitudes are determined during the equilibrium computation. This approach is followed by Sarma (1979).

The computational procedures almost always involve an iterative solution method (except where the soils are purely cohesive) once inter-slice forces are considered, however approximately. The analyst is usually faced with selecting one of a small range of options in whatever software he has, and does not have a free choice amongst the various methods available at large.

Since even Bishop's procedure is laborious to compute by hand, slope stability calculations became an early application for computers. To summarise, the essential differences between the methods lie in the treatment of inter-slice forces – and this choice is largely immaterial, and even the once-important issue of how long it took to produce results is of little significance, given the speed of modern computing hardware and software.

23.3.4 Slip circles

It is found that in many cases an arc of a circle (**Figure 23.2**) on the two-dimensional slope section provides an adequately good first approximation to the shape of a failure. Indeed, some case examples show that under particular combinations of conditions it can be a *good* approximation. These surfaces are called slip circles. Since all of the geometric factors can be calculated simply from the position of the centre of a slip circle together with its radius, the slip circle lends itself to repetitive analyses using many slip circles to find the one or ones with least factors of safety. Most analysis software uses a regular rectangular grid of centres together with multiple circles emanating from each centre. Refinements may include constraints on where the circles may intersect the slope, and grids of centres

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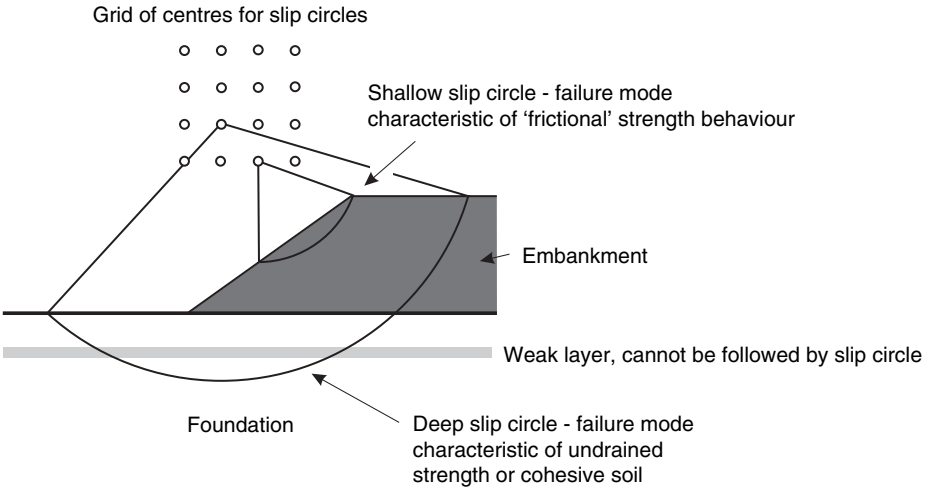


Figure 23.2 The slip circle, commonly employed where the critical clip surface is unknown

of various types and orientations. The particular method of indicating the critical slip circle (i.e. the one with the least factor of safety) and how it relates to other possible slip circles is not standardised, but depends in part on the imagination of the programmer and, to an extent, that of the analyst.

Stability analyses of slopes with soil strengths that are dominated by their angles of shearing resistance (ϕ') tend to indicate that shallow slip surfaces are the most critical. In the undrained case, and where strength is dominated by a cohesion parameter, the critical slip circles are deep, akin to foundation failures. A fundamental defect of the slip circle approach is that no circle can follow a thin stratum of weak soil (**Figures 23.2** and **23.3**), and where slip circles do indicate the weaker zones of a slope (low shear strength parameters, but also where high pore pressures exist) these zones are discoverable by visual inspection anyway. Since it is not possible to generate compound slip surfaces with quite the facility with which slip circles can be generated (from a grid of centres), it is useful to undertake a slip circle based analysis in the first instance, and use the most critical slip circles as a basis for extending into compound and other shapes to include the weak zones.

Many cases of slope failure in engineered slopes occur because at the time of construction a weak layer in an adverse orientation was incorporated into the fill or left in the embankment foundation. Similarly, in cuts and natural slopes, a weak horizon again provides a path for a failure surface. It is therefore essential in these cases not to rely simply on slip circle approaches.

23.3.5 Three-dimensional effects

Three dimensional analyses are rarely done in routine engineering earthworks design, simply because there is no 3D analogue of the slip circle search procedure. In forensic (post-failure) analyses, the 3D approach may convey some benefits. However, an alternative approach is to make 2D analyses and combine the results. This is often sufficient.

A determined analyst will find that 2D calculations often yield a lower factor of safety than the 3D calculation. This is sometimes the result of failing to consider adequately the variation in pore water pressure in the out-of-plane direction: field failures all have a three dimensional element.

23.3.6 Pore water pressure changes during the 'lifetime' of a slope

In the foundation of an earth fill, undrained loading usually gives rise to substantial pore pressures. Provided that failure does not occur, the dissipation of excess foundation pore pressures leads to a gain in foundation strength and therefore to increased stability with time, although the effect of pore pressure dissipation in the foundation decreases with time. In some cases, the migration of pore water laterally in horizontal layers of higher permeability may, early in the process, lead to a transfer of pressures from under the embankment crest where they do little harm, to the toe, where they do damage. The critical point for stability, therefore, in the lifetime of a filled slope on a weak foundation is either at the end of construction, or shortly thereafter if pore pressures do migrate out to the toe. Some benefit is obtained by filling slowly, or by allowing rest periods (often throughout the rainy season when fill cannot be placed) during which time the foundation consolidates and is therefore capable of bearing the following season's lift of fill.

In the case of a cut slope, the total stresses decrease during excavation, and the undrained pore pressure change in response decreases the pore pressure state over the lower part of the cut slope to a state of suction. Loss of this suction occurs with infiltration of water from the ground surface or from depth, and this leads to loss of strength and in some cases to delayed failure. The equilibration of pore pressures may take decades. The process is most marked in stiff clays.

Clay fills are inevitably placed incompletely saturated and manifest strength due to capillary effects in whatever pore water

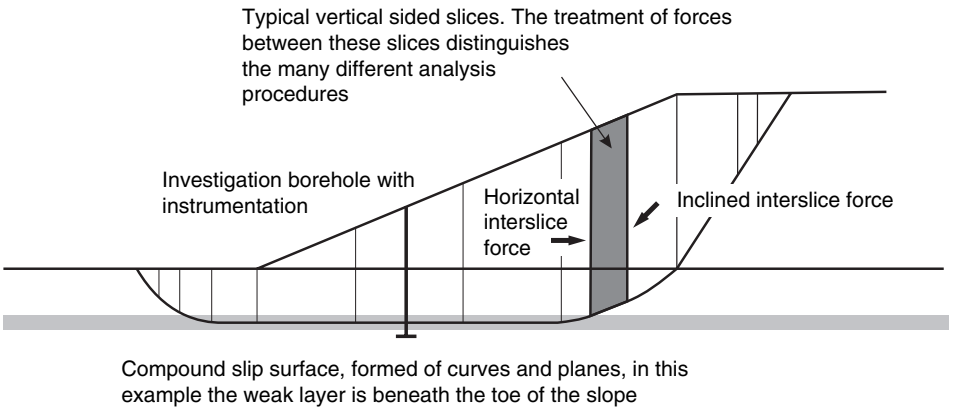


Figure 23.3 Compound failure with sub-horizontal bedding-controlled shear surface

there is. As in the case of a cut slope, loss of suction may occur due to water ingress, and this can, in clay fill slopes, also lead to delayed failure. However, suctions near the surface can be reinstated by dry weather, or by moisture abstraction through the roots of vegetation. The progress towards delayed failure is neither as certain nor as monotonic as the gain in strength of an embankment foundation as it consolidates. Although recent research work has been directed to understanding and quantifying these effects, they are not routinely considered in slope stability analyses, which proceed on the basis of pessimistically-assessed positive pore pressure conditions.

Partly submerged slopes gain support from the external water load, but lose strength as water enters the slope and exerts pore water pressures. These two effects act in opposite senses, and the result may increase or decrease the overall stability relative to the slope prior to submergence. In many cases, it is found that there is a particular water level termed the 'critical pool level' at which point the stability is least. Care on first impounding water-retaining earthworks is therefore called for. Stability is often severely affected if the support from the water is rapidly withdrawn ('rapid drawdown'), especially if there is insufficient time for water in the slope to flow out. This, as in all the cases of partly-submerged slopes, is modelled in slope stability analyses through the use of appropriate distributions of pore water pressures.

23.3.7 Seismic stability

During an earthquake, slopes may be accelerated and decelerated multiple times in a variety of directions, but usually for very short durations in each. A simple approach to dealing with this is to take the inertial forces as some fraction of gravity, and to apply the resulting forces in a standard slope stability analysis as a horizontal body force. In some cases, vertical body forces are applied in addition to weight. There may also be undrained pore pressure changes. Inevitably, slopes with a high static factor of safety and moderate to low inertial forces appear to perform well, and those with low initial stability or where the inertial forces exceed some threshold appear to be in line for failure. The body forces are calculated in the horizontal direction from $k_h W$, where k_h is the proportion of gravitational acceleration assumed to act horizontally.

An alternative procedure is to calculate the value of k_h that causes F to drop to one. This factor is termed k_{crit} or k_c. Then, whenever k_h exceeds k_c, the slope will begin to move. However, if the times involved are in milliseconds, the displacements per seismic pulse are tiny, although they increment pulse by pulse through each design model earthquake. We thus discover that $F < 1$ is not necessarily a disaster, provided that it occurs for a short time, and the soil properties are not left in a damaged state that will permit failure even when the shaking stops (e.g. via increased pore water pressures).

Slopes that continue to move after the earthquake vibrations have ceased are of course the primary concern. This may be the result of high earthquake-generated pore pressures. Loose fine granular soils and saturated cohesive soils with a porous fabric are most prone to loss of strength on shaking. Stiff clays form slip surfaces with diminished residual strength parameters, but this is often associated with dilation and lowered undrained pore pressures, and may not be significant.

23.3.8 Errors in slope stability analyses

Errors made in slope stability analyses are usually the result of:

(i) Incorrect assessment of design values for shear strength, including a failure to consider whether drained, consolidated drained, or consolidated undrained conditions are operating.

(ii) Incorrect assessment of pore water pressures in the ground.

(iii) Inadequate assessment of the effect of weak layers in the slope which may be present in the soil for stratigraphic reasons, or have been created by construction activities.

(iv) Selection of inadequate mobilisation factors in a particular case, especially where there is uncertainty surrounding the soil parameters and pore water pressures, but less obviously, where there is the risk of progressive failure.

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The third item in this list sometimes manifests itself by a weak layer not having been recognised during investigation or construction, and also by the analyst not considering slippage along such weak layers. The use of slip circles alone may result in this situation arising in the analysis.

The technique most commonly employed in finding the critical (least safe) slip surface is a 'brute force' approach of analysing often thousands of slip circles. It is easy to forget that the critical surface may not be the arc of a circle.

Errors in the computation itself are much rarer than in the above technique, probably the result of the widespread deployment of quality-controlled computer software.

23.3.9 Continuum methods

Continuum methods (such as finite element and finite difference stress analyses) are occasionally employed in slope stability problems, but they need parameters that are more difficult to obtain, and an even more skilled analyst, so are expensive to apply in routine situations. They provide estimates of deformations, absent from a limit-equilibrium analysis (including the pattern of deformations) and they do not require analyst intervention to show the most probable mode of deformation leading to failure.

However, the problem of dealing with thin weak bands in the soil is perhaps more intractable than in limit equilibrium analyses.

23.4 Rectification of unstable slopes

23.4.1 Introduction

Unstable, or potentially unstable, slopes may pose threats to human interests and intervention of some sort is often called for. The baseline case is that nothing is done. Value judgements can then be made on the cost of intervention against the anticipated losses.

Slope stabilisation methods are described in Chapter 72 *Slope stabilisation methods* with the generalities of design in Section 7 of this manual as a whole; here, the analysis issues with those methods are discussed. Retaining structures in particular are described in Section 6 of this manual.

One form of intervention that may be highly successful is to ensure that no valuable assets lie in areas of high risk. Many of the best examples of the application of such controls are adjacent to the coast. They include prohibition of development via statutory ordinances, realignment inland of coastal roads, and in one celebrated case, the historic Belle Tout lighthouse at Beachy Head, East Sussex, was shifted bodily inland by about 50 m to protect it from coastal cliff erosion.

Where the commonest form of instability is that of small to moderate flows (e.g. debris flows in mountainous areas) or of occasional rock falls, assets may be protected by various forms of barrier and catch structures, or deflection barriers and channels. Barriers range from earth or rockfill bunds, through reinforced concrete walls, to steel or occasionally timber post walls. Catch structures may be bunded enclosures or fences and netting. Where the failures are frequent, the catch structures need periodic clearing to maintain their effectiveness, combined with some repairs (especially where a catch structure has been called upon to resist loading beyond its design envelope) to compensate for corrosion, rot, vandalism or changed environmental conditions.

The final alternative is to reduce the probability of failure by engineering works on the slope itself.

23.4.2 Earthworks solutions

Slopes may be reprofiled to an overall flatter gradient. Where the likely mode of failure is relatively shallow, this is the optimum expedient. However, there are numerous cases where grading to a flatter angle is not only useless, but may provoke failures. For example, a coastal slope with a large, pre-existing landslide may exhibit a coastal cliff with small failures. Flattening this cliff may solve the small-scale local stability problem, while at the same time destabilising the deeper slide system as it constitutes an unloading of the toe zone of the deep-seated slide. Many slopes with pre-existing adversely orientated weaknesses, or with pore water pressure conditions independent of the regraded slope facet, are subject to this effect.

Deeper-seated failure modes tend to be improved with the construction of toe weighting fills and berms. However, such earthworks may cause local slope stability issues if inappropriately founded, say for example, on weak compressible subsoils or on sidelong ground. Toe fills may not be fully mobilised to support a slope until movements have taken place and compressed the fill laterally. A further effect of some concern to geotechnical engineers is the problem that arises in slope stability analysis if the toe of a slip surface rises through a toe fill with a high angle of shearing resistance ϕ'. This tends to make the computed factor of safety excessively high as a result of the assumptions in the analysis method. Care is needed in interpretation of these results.

Unloading the head of a slip failure is a useful method of stabilising it, but at the risk of affecting the head scar of the failure, which may require more significant interventions to keep it stable.

In general terms, when attempting to stabilise a slip, fills are highly effective above parts of the slip surface where it rises to ground level at the toe, and cuts are highly effective where the slip surface is steeper than ϕ' (where the slip surface rises towards the head of the slope). In between these locations, the relative effectiveness of cuts and fills depends on the pore pressure response in the slope to stress change.

The analysis of the stabilisation solution is a re-run with the modified slope profile and soil properties.

23.4.3 Drainage solutions

Slope drainage systems fall naturally into two main groups: preventing water from penetrating the slope in the first instance, and removing it once it is there.

A good tree canopy is effective in controlling rainwater ingress, but deciduous trees lose this capability annually. In any case, the canopy is established slowly, and can be breached by the loss of

individual trees or wholesale felling. Creation of impermeable coverings inhibits the movement outwards of water already in the slope, and creates problems of peak discharge during heavy storms, as well as potential aesthetic issues. Very shallow drainage in a slope, such as ditch networks and land drains formed from clay pipes, tile drains and modern perforated plastic pipes, has the effect of controlling infiltration. Buried systems have the additional benefit of slowing down the release of captured water. Land drainage systems are easily blocked, and are difficult to maintain. When blocked, they back up water, overpressuring the drainage system and the soil, causing local failures.

Deep drainage systems may be installed to release pore pressures, such as undrained loading-induced pore pressures that are then unlikely to be replenished, or to control seepage-induced pore pressures that are continuously replenished. In the former case, filtration is unlikely to be of major significance, but in the latter case it is. Filters prevent loss of fines into and through the drainage system that would result in internal erosion in the soil, and possible blockages in the drainage system.

Pore pressures induced by undrained loading are most often associated with the construction of embankments over weak and deformable foundations, where the load from the embankment fill compresses the foundation. Release systems for these pore water pressures usually take the form of vertical filter-fabric wicks, or sand-filled drain wells, with hybrid systems sometimes employed. They may connect to permeable strata at depth, or to a collector drainage layer at their top, or both. Accelerated consolidation of the ground can be caused by applying suction, through the use of electrical systems, or where stability permits, by applying surcharge loads. Where the pore water pressures are induced in high fills, the drainage system may be laid at the time of construction. In this case, it is often a drainage layer formed from appropriately selected granular material with a high permeability. The cost of drainage layers in a very large earthwork can be high, and to minimise this, the layers are formed from a grid of interconnecting 'strips'. In earth dams, the drainage layers to control construction-induced pore pressures may form part of the permanent drainage system, in which case filters are obligatory. Filters may take the form of further granular layers or be made from thin sheets of manufactured and non-degradable fabric.

Permanent drainage systems may also be incorporated in a slope by drilling sub-horizontal or other drains. These are kept open in soils with perforated plastic (or other) liners. Filter protection of these employs proprietary systems of variable longevity, most of which prove difficult to maintain. Bored drains can be installed from the surface, or from shafts and tunnels, both of which operate as drains. Shafts collect water at their bases and ideally are interconnected by a tunnel or series of borings to permit the water to drain under gravity; if not, then pumps must be employed. These pumps are subject to breakdown, and have running and replacement costs.

The efflux from drainage systems may contain contaminants, requiring treatment before it can be discharged. These contaminants may include microbial contaminants (e.g. from MSW, or where septic tank foul drainage is employed locally), or mineral content such as the iron-rich oxygen demanding acid mine drainage water from underneath some types of mine waste tips. In some cases, discharge of water from a slope drainage system may not be permitted, or may be permitted only subject to severe constraints, by the appropriate local authority or environmental protection agency. Some forms of drainage water, most notably calcite-rich or iron-rich water can cause drain blockage due to precipitation, in the latter case assisted by bacteriological action.

Drainage trenches of several metres or more in depth are often employed to stabilise failures in cut slopes. Regardless of their depth, they can reduce pore water pressures. However, when they are deep enough to penetrate sheared surfaces or other zones of weakness, they also have a reinforcing or buttressing effect in the slope, in which case they are termed counterfort drains. Drainage trenches infilled with gravel and open to the surface may permit infiltration of water, and if the efflux is obstructed, they can operate in the opposite sense to which they were intended. Although drains may accelerate the equilibration of lowered excavation-induced pore pressures to their final state, that state is usually better for stability than if the drains were not there, and the net effect is beneficial overall.

Significant drainage actions lead to consolidation in the ground and surface deflections and strains. Drainage water collected in a pipework system has the capacity to do damage if it is released in the wrong place. Transporting drainage water across or through unstable slopes is not usually without risk of promoting failure should the drains leak.

Analysis of slope stability with drains is simply a question of modifying the pore pressures used in the analysis.

23.4.4 Anchorages and reinforcements

Active systems of restraint can be installed using pre-tensioned ground anchors. These employ cable anchorages running in boreholes through a slope into stable ground underneath where they are anchored, with tensioning loads applied at the ground surface. Pre-tensioned anchors may need periodic restressing if the ground consolidates, and in the case of some types of ground movement. They are highly susceptible to corrosion, and require elaborate protection systems. Furthermore, the existence of large cable tensions in ground anchors may give rise to long-term safety concerns, for example if they are forgotten and the site is redeveloped in ignorance. It is rarely possible to install pre-tensioned ground anchors in their mathematically optimum orientation and position (their length, access to anchor head positions for installation and restressing, etc., being key constraints). They are often installed to control ground movement in their vicinity even if they do not stabilise the slope as a whole. It is preferable to spread the load from anchors into the ground with individual pads, as this minimises the risk of progressive failure of the whole anchorage system (**Figure 23.4**).

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Figure 23.4 Failed slope with planar slide which occurred largely as a result of failure of the ground anchor system
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Figure 23.5 Riverbank stabilisation in the Ironbridge Gorge, Shropshire: construction of piles reinforced by steel tubes by Birse Civils (contractor) and Jacobs (engineer)



Figure 23.6 Failed sheet pile retaining wall, originally installed in association with grading, drainage and sea defences, to stabilise compound slip with basal shear surface at approximately mid-slope height

Plastic and metal grids or strips can be incorporated into fill slopes to retain the face, where this is oversteep, or to prevent lateral spreading (sliding on sub-horizontal planes). They are also used to restrain the sliding tendency of the face of a fill on sidelong ground. Since the capacity and durability of these systems depends on their manufacturer, it is conventional to employ manufacturer-specific design tools and construction specifications. Reinforcement grids may be installed in multiple layers, but then care is needed to ensure that the layers do not slide over each other. Loads are induced in these reinforcement systems by deformations during construction, which makes them a passive system.

Passive restraints can be induced in other reinforcing elements in the slope and are usually inserted into drill holes. Where they are of comparatively small diameter and often low-angle they are called 'soil nails'. Soil nails are essentially small diameter steel-reinforced concrete 'piles', which act in tension. Piles may also be employed to reinforce the soil mass in a slope. They may take the form of dowels (rows or grids of medium diameter piles reinforced with traditional reinforcement cages of bars), or larger diameter piles reinforced with steel tubes (**Figure 23.5**), or structural steel I- or H-sections. Bored piling techniques are frequently preferred to driven piling techniques, as they prevent dynamic loading of the ground and employ larger sections than can conveniently be driven.

Trenches that go deep enough to penetrate sheared surfaces or other zones of weakness and which are subsequently filled with concrete or compacted rockfill provide shear keys. As noted above, if they are permeable, they can provide some drainage.

Sheet piles may prove effective to stabilise small slides, but they are intrinsically flexible, susceptible to corrosion, and come apart at the interlocks (**Figure 23.6**).

In analyses of slopes with reinforcing elements, the necessary forces and reactions can be considered in the equilibrium calculation. It is usually a serious error to incorporate soil zones with 'concrete' or 'steel' properties into a slope stability analysis dataset and then run slip surfaces through it.

23.5 Factors of safety in slope engineering

23.5.1 Risk

Risk is defined as the potential loss from a hazard. It is the product of the probability of the damaging occurrence (in a given place and timescale) multiplied by the consequences when it happens. *When* is used, not *if*, as any probability of failure other than zero implies that sooner or later it *will* happen. Those consequences are most easily expressed in financial terms, and reflect the number and value of elements at risk and

their vulnerability, or the proportion of their value lost in the damaging event(s). Fortunately, in the UK, fatalities and injuries from slope instability are comparatively rare. Apart from one small event, the entire record shows that financial effects far outweigh personal injury. This makes most risk analyses simply a matter of money. Other parts of the world are less fortunate and must find ways of dealing with the two aspects concurrently.

The entire gamut of engineering slope stabilisation measures deals with risk by reducing the probability of occurrence of damaging events (slope failures). Relocation of assets and control of access to sites reduces the number and value of elements at risk. Barriers and control structures may be seen as reducing the vulnerability of those elements. Hence, the treatment of slope instability (actual or potential) is a matter of risk reduction (Bromhead, 1997, 2005).

23.5.2 Factor of safety approach

In an ideal world, we might approach every slope with an intention to reduce the risk from failure to tolerable levels. In practice, engineering calculations rely on factors of safety, and these (as articulated above) represent a factoring down of shear strength parameters. Thus the traditional methods of slope stability engineering fit neatly with Eurocode 7's philosophy. The recommended factors in Eurocode 7 are believed to result (in most everyday practice) in tolerable levels of risk, provided that appropriate assessments of parameters and mechanisms are made in the first instance. There are almost certain to be cases where larger factors are required, for example where the consequences of failure are dire, or where uncertainties in soil behaviour or properties demand it. Natural slopes of large scale may require smaller factors of safety (or those recommended by Eurocode 7 may simply be unachievable) for an equivalent level of security. For example, residual strength on a very deep sheared surface is unlikely to be subject to further reduction, and this may not require factoring in the same way as the unsheared strength that may be sensitive or subject to degradation with strain. However, a highly sensitive soil may be subject to progressive failure at a mobilisation factor that is satisfactory where the subsoil is insensitive.

23.5.3 Serviceability limits

In the case of the routine design of earthworks of moderate height with conventional fills and placement methods, the Eurocode 7 factors also provide some control over deflections associated with the mobilisation of shear resistance in the ground (but not, of course, to consolidation settlement and related mechanisms). In these types of earthwork, adoption of lower factors of safety gives rise to ground deformations as shear strength is mobilised. The surface manifestations include toe bulging and cracking at the embankment crest. If the earthwork consolidates and gains strength, it may be possible to give the embankment a cosmetic 'makeover' without permanent impairment of its performance. However, such deformations are more commonly experienced as a precursor to collapse, probably because the soils involved are susceptible to progressive, brittle, failure and as a result, should be treated with caution. This is especially so if the embankment fill or subsoil is clayey and can develop slickensided shear surfaces at residual strength.

In-slope geotechnical instrumentation can detect the onset of ground deformations before they are apparent to the naked eye. Careful inspection and interpretation of geotechnical instrumentation readouts is therefore an essential adjunct to construction, especially of particularly high or steep earthworks, those on weak subsoils, and where the consequences of collapse are critical (from the perspective of collateral damage) and affect the construction programme.

23.6 Post-failure investigations

Post-failure investigations are conducted from a variety of motivations, from facilitating remediation through to attributing blame. In clay soils, an essential element of investigation is to ascertain the location of the basal shear surface of the slide. This may be done by visual examination of borehole samples, or, when the slide is still moving, by the use of appropriate instrumentation. Inferences on slip surface position and depth may be drawn from the surface morphology and surface deformations, but logged boreholes and instrumentation provide the best information. Shafts, adits and pits are used to expose shear surfaces *in situ*, but they are potentially hazardous, requiring proper support at all times, and they should only be entered by trained operatives acting within an appropriate safety plan.

Many failures occur under climate-driven or other transient pore water conditions; using post-failure instrumentation, it may still be not be possible to recover the situation.

In operation, failures in infrastructure earthworks and in both temporary and permanent slopes may prove disruptive and inconvenient to leave for later forensic investigation. It is therefore essential that proper investigations take place concurrently with the clear-up operations. During construction works, the costs of remediation of some slope failures may be significantly less than after completion of the works – as plant is on site and access is usually better. However, as failures are not normally contemplated at the design stage, it may be necessary to implement remedial works with additional care; they may well contain activities not catered for in the initial site health and safety plan.

It is sometimes the case that an engineer investigating a failed slope needs to form an opinion on the cause or causes of failure. In this situation, the failure is often the result of several preparatory factors, combined with a final trigger. It is naive to single out the final trigger as the causative factor. Instead, the individual preparatory factors should be investigated and ranked in decreasing order of impact on the slope. It is then usually clear that several factors in combination have led to the failure.

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All chapters within Sections 1 Context and 2 Fundamental principles together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

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Chapter 24

Dynamic and seismic loading of soils

Jeffrey Priest Geomechanics Research Group, University of Southampton, UK

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There are many circumstances where cyclic loads are applied to the soil, either by natural or manmade forces. Although the magnitudes of these dynamic loads are often much smaller than static loads, inertial forces may become important and must be considered in geotechnical design. This chapter introduces the reader to the dynamic loading of soils and the main characteristics that differentiate dynamic and static loading: loading frequency and loading cycles, and the strain dependent behaviour of soil. A variety of measurement techniques for characterising the dynamic response of the soil are available and these are outlined, differentiating those routinely employed during low and high strain cyclic loading, both in field measurements and laboratory testing. The theoretical response of the soil to cyclic loading is briefly examined and the main features highlighted. The factors that affect the dynamic soil behaviour are discussed, through the use of results from extensive laboratory testing in this area. The phenomenon of 'soil liquefaction' is also highlighted along with the factors that influence it.

24.1 Introduction

Traditional soil mechanics and geotechnical engineering design was predominantly concerned with soil strength and stiffness so that the design engineer could define the failure state of the soil and/or control excessive deformation of the soil/structure. In these circumstances, the loading is considered static and the strain imposed within the soil may vary from about 10^{-3} (in service) to a few percent (at failure).

There are however, many circumstances where cyclic (dynamic) loads are applied to the soil either by natural forces such as earthquakes (seismic), wind and water waves, or from manmade sources such as bomb blasts, traffic loads and machine foundations. The magnitudes of these dynamic loads are generally much smaller than most static loads and generate strains within the soil as low as 10^{-6}. However, during cyclic loading the soil is subject to inertial forces, which increase proportionally with the square of the loading frequency and may become important. It is therefore imperative for the design engineer to have an understanding of the dynamic behaviour of soils in circumstances where the soil may be subject to these loading events.

This chapter introduces the reader to the dynamic behaviour of soils and outlines some of its main features. References are given where more detailed information can be obtained if required.

24.1.1 Dynamic loading

Dynamic loading can be defined by two main characteristics, which are loading frequency and the number of load cycles (**Figure 24.1**). Generally, dynamic problems are where the cyclic load is applied at frequencies greater than around 1 Hz (1 cycle per second). Depending on the loading event, the number of cycles can vary from single loads, such as those generated by bombs or blasting, up to many thousands of cycles as in machine foundation vibration. Problems with dynamic load can be classified into three general areas based on the number of load cycles: (i) impulse load, such as bomb blasts where a single impulse of short duration occurs; (ii) vibration or wave propagation, generally applied to earthquake events and construction activities where a number of loading cycles occur (10–1000) at frequencies from 1–100 Hz; (iii) where the number of cycles is inordinately large, such as those associated with machine foundations and traffic loading (cars and trains). In this case the soil behaviour can be considered as potentially *fatigue* related.

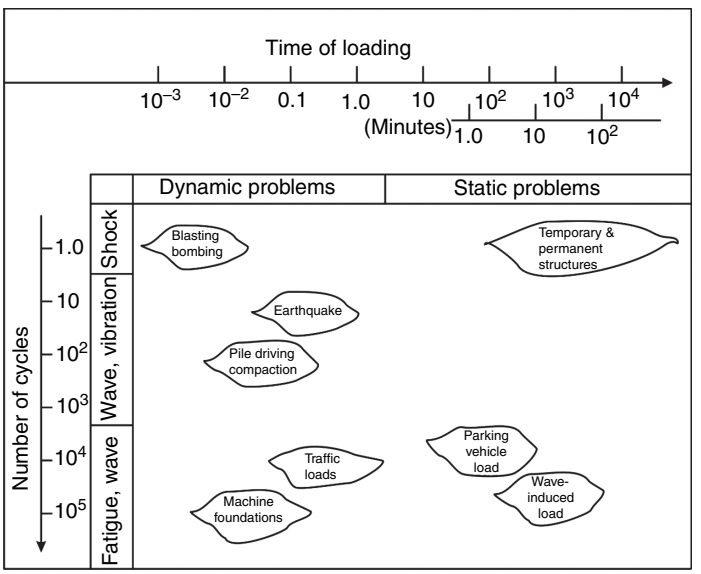


Figure 24.1 Classification of dynamic problems
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24.1.2 Strain dependent behaviour of soils

Over recent years, it has been shown that the deformation behaviour of soils is heavily dependent on the level of shear

strain (γ) to which the soil is subjected. **Figure 24.2** shows the shear strain dependent behaviour of soils. It can be seen that for very small strains (10^{-6} to 10^{-4}) such as those induced by wave propagation and vibration, the soil response is elastic and recoverable (i.e. there is no permanent deformation). At these strains, both the number of loading cycles and the rate of loading have a negligible effect on the soil behaviour. At the intermediate strain range (10^{-4} to 10^{-2}) the soil is elastoplastic and irrecoverable permanent deformations occur, usually associated with cracks and differential settlement of structures supported on the soil. At these strains, the number of loading cycles and the rate of loading both have an effect. These strain levels are typical for a number of geotechnical structures such as retaining walls, tunnels and deep excavations (Mair, 1993). At higher strains the material exhibits failure, where the strains increase with no further increase in shear stress. These are typically associated with slope failure, compaction and liquefaction of non-clay soils.

Also listed in **Figure 24.2** are typical test methods employed for determining dynamic soil properties along with an estimate of the ranges of strain that can be generated. Generally there is little crossover between dynamic tests and those defined as static (less than 1 Hz frequency) with respect to the shear strain level applied. Thus, historical differences in stiffness – measured using dynamic and static tests – may result from the strain level induced in the soil. With field measurements using vibration or wave propagation techniques, large amounts of energy are required to generate strains in excess of 10^{-3} due to the inertial forces involved in mobilising the soil. Therefore, for tests at higher strains, low frequency repeated load tests (where inertial forces are negligible) are more commonly used. In these circumstances, the loading becomes a repetition of the static load (Ishihara, 1996). Large strain vibration tests are also difficult to achieve in the laboratory. Although sufficient energy may be available to induce large strains in a soil sample during vibration, the governing equations become invalid (see Richart *et al*., 1970). With repeat load tests at large strains, the determination of elastic constants is straightforward. It is important to specify the right test procedure to derive accurate parameters for use in the design process.

24.2 Wave propagation in soil

The tests highlighted in **Figure 24.2** can be broadly grouped into low and high strain measurements. At high strain, the stiffness and damping are calculated directly from the stresses and strains measured in the soil as would be done in a static test (see Chapter 49 *Sampling and laboratory testing*). Although recent advances in measuring small deformations in soils have allowed direct measurements of soil properties at small strains (Clayton and Heymann, 2001; Jardine *et al*., 1984; Xu *et al*., 2007), they are not routinely undertaken and so vibration/wave propagation techniques are typically employed for testing at small strains.

24.2.1 Wave velocity

Soil parameters are computed from wave propagation techniques based on the theoretical response of the soil during the propagation of a stress wave through it. There are a variety of waves that can propagate in an elastic material (see Richart *et al*. (1970) or Kramer (1996), for a detailed theoretical background to groundbourne wave propagation). For soils, three types of waves are readily encountered and are of importance. The first two are termed body waves, which are propagated

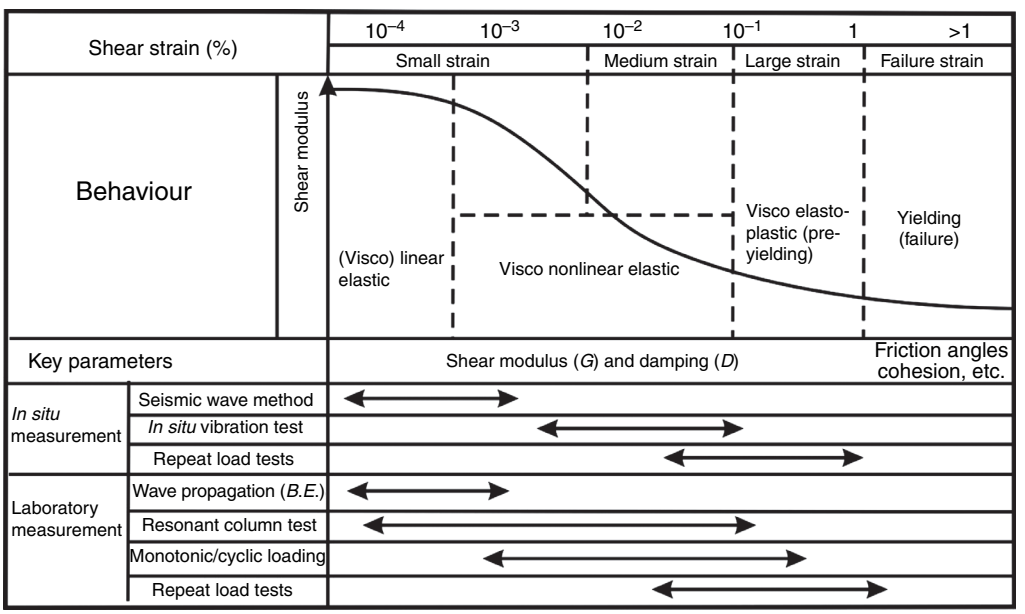


Figure 24.2 Strain dependent properties of soils. B.E. stands for Bender Element, which is a laboratory-based wave propagation technique
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within the soil and comprise the compressional wave (P-wave) and shear wave (S-wave). In relatively soft saturated near surface sediments (which is likely to be the case for a large number of geotechnical problems) the P-wave (V_p) is dominated by the bulk modulus of the pore fluid (the water is 'hard' compared to the soil) and the resultant V_p may be close to that of the pore fluid. If the soil is unsaturated, V_p can range from that of the soil matrix (with no pore fluid) to that for the saturated case. Therefore the use of V_p to determine the properties of soils is problematic, even with detailed knowledge of the hydrological regime, unless the bulk modulus of the soil is much greater than that of the pore fluid. However, pore fluids do not carry shear stresses, so the velocity of S-waves (V_s) is only influenced by the soil and not by the pore fluid. It can also be shown that during most dynamic loading events, it is cyclic changes in shear stresses that influence the behaviour of the soil. From the theory of propagating waves it can be shown that the wave velocity through the soil is related to the stiffness of the soil by

$$G = \rho V_s^2 \tag{24.1}$$

where G is the shear modulus of the soil, and ρ is the soil density.

Therefore changes in V_s within a soil can be used to determine the small strain stiffness, and measurement of S-waves is commonly employed to determine the dynamic behaviour of soils at small strains.

The third wave that is frequently encountered in soils is the Rayleigh wave, which travels along the ground surface. Rayleigh wave measurements have gained in importance in recent years since ground surface measurements can be readily and easily undertaken without the need for any intrusive investigations, such as those required for measuring P-waves or S-waves. There is no direct link between the velocity of Rayleigh waves (V_R) and soil stiffness (Young's modulus, E or G), however, it has been shown that for nearly all values of Poisson's ratio, $V_s \approx 1.09V_R$ (Richart *et al*., 1970). Therefore measurements of Rayleigh waves can give a good determination of shear wave velocities (Hiltunen and Woods, 1988) from which G can be derived.

24.2.2 Damping–wave attenuation

The theory of wave propagation is usually considered in terms of linear elastic materials, such that the waves would travel indefinitely without loss of strength (wave amplitude). However, in real soils the propagating waves are attenuated due to inherent material damping of the soil and geometric damping, where geometric damping relates to the loss of energy as the wavefront increases in length as it radiates out from the source. Material damping is associated with dissipation of energy during the passage of the stress wave as a result of friction, heat generation, etc. Thus, material damping is also a key parameter in determining the dynamic response of a soil.

24.3 Dynamic measurement techniques

As previously highlighted, the key parameters for understanding the dynamic behaviour of soils are shear modulus and damping. Although other parameters, such as Poisson's ratio and density, influence the dynamic behaviour of soils, these are generally secondary to stiffness and damping. A variety of field and laboratory techniques have been developed for measuring the shear modulus and damping, each with their own advantages and limitations. As the dynamic properties of soils are highly nonlinear with respect to strain, the choice of which technique (or range of techniques) to adopt needs to be carefully considered with respect to the existing stress conditions and those that may be imposed.

Detailed descriptions of soil investigation techniques and the data reduction methods employed to determine soil properties are given in Chapter 45 *Geophysical exploration and remote sensing* (field measurements) and Chapter 49 *Sampling and laboratory testing* (laboratory measurements). Section 24.4 lists the techniques relevant to determining dynamic soil properties.

24.3.1 Field measurement techniques

There are a number of well known advantages in performing dynamic field tests. These include: the ability to measure soil properties under *in situ* stress conditions; no sample disturbance to be accounted for during testing; the ability to survey large volumes of soil rather than using discrete locations (laboratory sampling). However, these must be considered in the light of certain disadvantages, such as working stress conditions cannot easily be applied to the soil; hydrological conditions cannot be controlled (pore water drainage); and for most tests, material properties are not measured directly but inferred (either theoretically or empirically).

24.3.1.1 Low strain tests

A number of low strain field measurement techniques can be used to measure dynamic soil properties and all measure the velocity of wave propagation in the soil. Those that determine soil properties from surface measurements include seismic reflection profiling and seismic refraction profiling – typically used to measure V_p, and continuous surface wave (CSW) or spectral analysis of surface waves (SASW) – used to measure V_R from which V_s is calculated. Those that determine soil properties from predrilled boreholes include seismic cross-hole and down-hole tests – which can be used to measure both V_p and V_s and respective damping parameters (with multiple receivers). The seismic cone is similar to the down-hole test but does not require a separate borehole since the cone is mechanically pushed into the ground.

24.3.1.2 High strain tests

For the majority of high strain field tests, the soil stiffness is usually obtained through empirical correlation with other non-related parameters from these tests. Such tests include the standard penetration test (SPT), the cone penetration test (CPT) and the dilatometer test (DMT) (see Chapter 47 *Field geotechnical testing*). The Ménard pressuremeter test (PMT) is the exception;

it can measure the soil stiffness directly through the stress–strain response of the soil.

24.3.2 Laboratory measurement techniques

In a similar manner to field measurement techniques, laboratory tests can be classified as either small or large strain tests. Also of importance in choosing a test method, is an understanding of the stress regime within the soil and the likely stresses that may be imposed, since each laboratory test can represent certain aspects of the required stress path.

24.3.2.1 Low strain tests

Advances in transducer accuracy and measurement techniques have allowed local strain measurements to be measured on samples under static loading. However, the most commonly used methods to determine small strain stiffness parameters are those relying on wave propagation. The three most popular methods are the *resonant column*, *ultrasonic pulse transmission* and *bender element* tests. These tests are generally considered to be non-destructive in that the soil remains unchanged during the dynamic test.

The resonant column is the most widely used test equipment and is routinely used to measure strain from 10^{-6} to 10^{-3}. Different modes of operation have allowed different soil parameters to be determined, such as G and E, as well as their respective damping (see Clayton *et al*., 2005). The resonant column works by vibrating the soil at its resonant frequency from which the wave propagation velocity can be derived. In contrast, the ultrasonic pulse transmission and bender element tests directly record the travel time of the generated wave between the source and the receiver. Typically the ultrasonic pulse transmission test will generate P-waves while the bender element test generates S-waves. For these two tests the strain imposed upon the soil is usually constant and of the order of 10^{-6}. Both these tests can be incorporated into conventional soil testing apparatus such as the conventional triaxial apparatus (Viggiani and Atkinson, 1995), the simple shear device, oedometers, etc.

24.3.2.2 High strain tests

For laboratory samples subject to high dynamic shear strains, the stiffness of soils is derived from the stresses and strains measured during a test. Soils tested in drained conditions are likely to exhibit changes in volumetric strain, while those tested in undrained conditions (where volumetric strains are prevented) exhibit changes in pore pressure (thereby changing the effective stress). Thus for high strain tests, measurements of pore pressure and/or volume change are required to determine soil stiffness and damping accurately.

A number of different devices are used to measure stiffness at high strains including the *cyclic triaxial*, the *cyclic direct simple shear* and the *cyclic torsional* tests. All tests measure the same basic parameters; however, there are differences between the tests which relate to how well they represent the shear stresses within the ground. Although the cyclic triaxial is the most commonly used apparatus for measuring the dynamic soil properties at high strain, it is the least accurate in reproducing the stress conditions that exist during dynamic loading. This is because the principal stresses are constrained to act in the vertical and horizontal directions. The cyclic direct simple shear test overcomes this shortfall by deforming the soil in a similar way to that during propagation of shear stress in the ground. However, shear stresses are only applied on the top and bottom surfaces of the sample. The cyclic torsional test overcomes these shortcomings and the stress conditions reproduce those under dynamic loading more closely.

24.4 Dynamic soil properties

24.4.1 Theoretical behaviour of soils

Complex geotechnical design problems focusing on soil–structure interactions can now be solved using a variety of numerical modelling techniques. In addition, advances in desktop computing power have allowed these problems to be routinely solved by most practising engineers if a model of the main characteristics of soil behaviour under cyclic loading is provided (see Chapter 6 *Computer analysis principles in geotechnical engineering*).

As discussed previously, at very small strains (10^{-6}) the behaviour of the soil is generally considered to be elastic in nature such that shear modulus is a key parameter. At intermediate strains, from 10^{-5} to 10^{-2}, the soil behaviour becomes elastoplastic with the shear modulus reducing with increasing strain. In addition, energy is dissipated during each cycle of stress due to slippage of particles at grain contacts. The energy loss (damping) is rate dependent and hysteretic in nature. Thus the dynamic behaviour of soils can be fairly accurately described using nonlinear viscoelastic theory (it should be noted that the material damping within the soil is not viscous in nature, but it is mathematically convenient to assume so; see Thomson (1988) for a mathematical derivation of viscous damping). The general hysteretic behaviour of a nonlinear-viscoelastic material during a stress cycle is illustrated in **Figure 24.3**. The stress–strain curve can be considered as being composed of two curves, (1) the backbone/skeleton curve which represents the montonic loading, and (2) the hysteretic loop. The shear modulus at small strains, G_{max} can be obtained from the initial gradient of the backbone curve, whilst the secant shear modulus at any given strain is given by

$$G = \frac{\tau_a}{\gamma_a} \tag{24.2}$$

where τ_a and γ_a are the shear stress and shear strain, respectively, at the given point in the loading cycle.

The inherent damping of the soil is related to the energy dissipated during a loading cycle and is defined by

$$D = \frac{1}{4\pi}\frac{\Delta W}{W} \tag{24.3}$$

where D is the damping ratio, W is the total stored energy (given by the area under the stress–strain curve) and ΔW is the energy loss per cycle.

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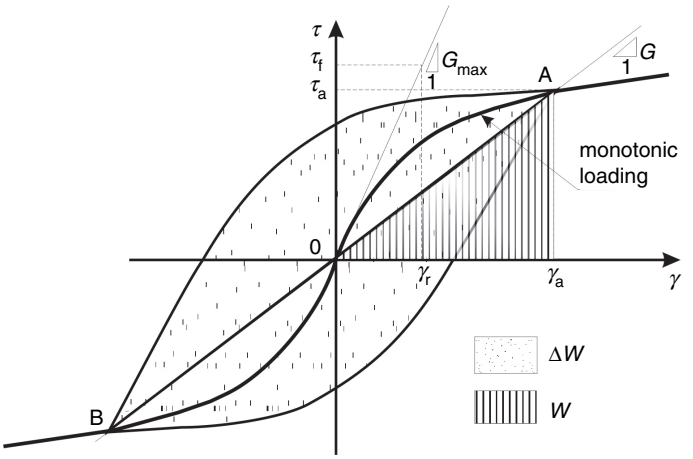


Figure 24.3 Stress–strain relationship for hysteretic soils

The two most popular constitutive models developed to describe the nonlinear stress–strain behaviour of soils are the two parameter hyperbolic model and the four parameter Ramberg–Osgood (R–O) model. The general features of both the hyperbolic and R–O models can be seen in **Figure 24.3**. For the hyperbolic model, the general forms of the equations are

$$\frac{G}{G_{max}} = \frac{1}{1 + \gamma_a / \gamma_r}, \tag{24.4}$$

$$G = \frac{\tau_a}{\gamma_a}. \tag{24.5}$$

where γ_r is a reference strain derived from the stress at failure from monotonic loading, τ_f, and assuming the soil behaves elastically.

For the R–O model, the general forms of the equations are

$$\frac{G}{G_{max}} = \frac{1}{1 + \alpha \left[\frac{G}{G_{max}} \frac{\gamma_a}{\gamma_r} \right]^{r-1}}, \tag{24.6}$$

$$\alpha = \frac{\gamma_f}{\gamma_r} - 1, \tag{24.7}$$

$$r = \frac{1 + \frac{\pi D_{min}}{2} \frac{1}{1 - G_f / G_{max}}}{1 - \frac{\pi D_{min}}{2} \frac{1}{1 - G_f / G_{max}}}, \tag{24.8}$$

$$G_f = \frac{\tau_f}{\gamma_f}, \tag{24.9}$$

where γ_f is the shear strain at failure derived from monotonic loading.

For the hyperbolic model both the initial shear modulus, G_{max}, and the shear strength, τ_f, are required, with the additional parameters α and r being required for the R–O model. These parameters are derived by fitting the respective models to experimental data.

24.4.2 Shear modulus and damping of soils

It has been previously highlighted that cyclic shear strain amplitude, γ_c, is a very important parameter for determining the dynamic behaviour of soils. Extensive laboratory testing undertaken to determine this behaviour has shown that shear modulus (G) and corresponding damping ratio of soils (D) are influenced by a range of parameters, such that

$$G; D = f(\gamma_c, \sigma'_0, e, N, Sr, \dot{\gamma}, OCR, C_m, t) \tag{24.10}$$

where γ_c = cyclic shear strain amplitude,

σ'_0 = mean effective principal stress,

e = void ratio,

N = number of loading cycles,

Sr = saturation ratio,

$\dot{\gamma}$ = strain rate (frequency of cyclic loading),

OCR = overconsolidation ratio,

m = grain characteristics – shape, size, mineralogy,

t = time of confinement under applied load.

When γ_c is around 10^{-6} it has been shown that for unconsolidated granular soils, such as sands, the dynamic behaviour is mostly independent of the variables considered above, apart from σ'_0 and e. For clayey materials, in addition to σ'_0 and e, other parameters such as OCR and t become important. At higher strains, both clay and non-clay soils are also affected by $\dot{\gamma}$ and N.

24.4.2.1 Small strain shear modulus, G_{max}

From testing of soils at very small strains (generally taken as $10^{-4}\%$), the shear modulus of soils has been found to be reasonably constant; and, for a given soil, a maximum. The value of shear modulus at these very small strains is therefore termed G_{max} or G_0. **Figure 24.4** shows the typical pattern of changes in shear modulus with isotropic stress for a range of dry granular materials. It can be seen that as σ'_0 increases, so does G_{max}. Empirically, a simple exponential relationship is found between G_{max} and σ'_0 of the form

$$G = A \sigma'^{b} \tag{24.11}$$

where A and b are constants (Hardin and Black, 1968; Hardin and Drnevich, 1972). By fitting equation (24.11) to the data in **Figure 24.5**, the b-value for the soil can be obtained (as highlighted in **Figure 24.5**). For non-clay soils the b-value is generally found to vary from 0.4 to 0.6. The b-value represents both

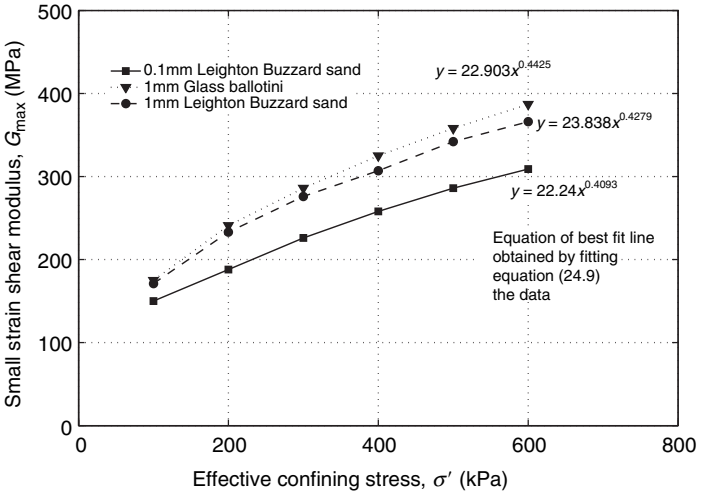


Figure 24.4 Influence of effective confining pressure (σ') on small strain shear modulus of a range of dry granular geomaterials
Data taken from Bui (2009)

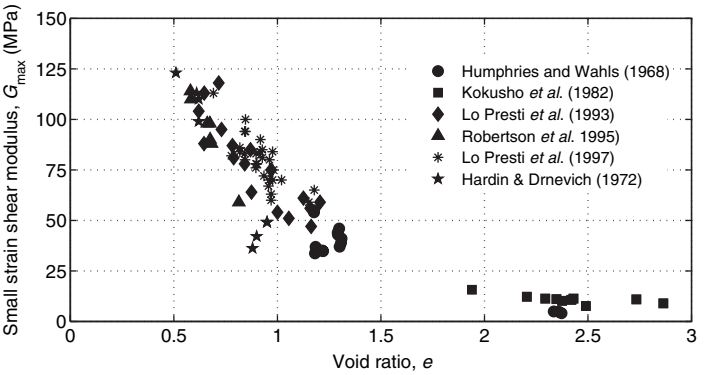


Figure 24.5 Changes in G_{max} with e for a range of soils. Data redrawn from review of literature conducted by Bui (2009)
Data obtained for samples subjected σ'_0 = 100 kPa

the nature of the contact stiffness and fabric change (Cascante *et al.*, 1998) during isotropic compression.

Based on Hertzian contact theory, the b-value for closely packed arrays of elastic spheres is 0.33 (Duffy and Mindlin, 1957), while a value of 0.5 is achieved for a random packing of elastic particles with non-spherical contacts (Goddard, 1990). Thus for non-clay soils, where changes in σ'_0 produce minor changes in fabric, aspects such as particle shape and size (which affect e) and the contact geometry influence the relationship between G_{max} and σ'_0. However, for clay soils where changes in σ'_0 can produce large changes in fabric (especially for soils with a high void ratio), the b-value may be appreciably higher than those for non-clay soils (Rampello *et al.*, 1997).

The influence of void ratio can be more clearly observed in **Figure 24.5**, which highlights changes in G_{max} with e for a range of geomaterials. It can be seen that as the void ratio reduces, G_{max} increases. For non-clay soils, the coordination number, N_c (the number of grain to grain contacts per grain), increases with reducing void ratio, leading to a reduction in average contact force under a given stress regime. Thus during dynamic loading, higher void ratio soils are likely to exhibit higher contact forces – giving rise to localized yielding at particle contacts, which reduces G_{max}. To account for the influence of void ratio on shear modulus, G is often normalized using a void ratio function, F(e) to eliminate the influence of void ratio. A number of empirically derived void ratio functions have been suggested, such as

$$F(e) = \frac{2.97 - e}{1 + e^2}, \text{ for values of } e \text{ between } 0.57 \text{ and } 0.98 \text{ (Hardin and Drnevich, 1972)} \quad (24.12)$$

$$F(e) = e^{-1.3}, \text{ for values of } e \text{ between } 0.81 \text{ and } 1.18 \text{ (Lo Presti } et\ al.\text{, 1997)}. \quad (24.13)$$

However, these functions are usually derived over a narrow void ratio range and there are no universal void ratio functions which can account for changes in G over the wide range of void ratios usually encountered.

Increasing OCR at a given value of σ'_0, primarily for clay soils, will increase G_{max}. This can be related to irrecoverable reduction in void ratio for a given soil as OCR increases, leading to increases in G_{max}. In addition, rearrangement of particle orientation inherently changes the structure of the soil causing an increase in G_{max} above that suggested by simply a reduction in void ratio (Kokusho *et al.*, 1982).

The influence of time on the measured shear modulus soils is related to the consolidation of the soil under applied stress (Humphries and Wahls, 1968), and can be characterized by two phases: (i) the initial phase which results from the primary consolidation of the soil, and (ii) a secondary phase which results from the long-term (creep) effects.

Figure 24.6 shows the effect of confinement time on G_{max} for a variety of soils. It can be seen that for clay soils both an initial phase and a secondary phase occur (**Figure 24.6(a)**), whilst for sands only a secondary phase is evident (**Figure 24.6(b)**). In addition, changes in G_{max} during the secondary phase for clay soils may be up to 15 times greater than that for sand. This observed behaviour has major implications for the testing of soils in the laboratory. For clay soils, testing must be conducted at the end of primary consolidation for each stress increment applied to the specimen. In addition, for all soils, testing must be conducted at equal intervals during the secondary phase for each sample.

24.4.2.2 Strain dependent shear modulus, G

As has been highlighted above, the dynamic behaviour of soils is highly nonlinear, and the material properties that are important such as shear modulus and damping change significantly with increasing strain during cyclic loading. Clays and sands have historically been treated individually. Seed and Idriss (1970) presented data from a wide range of test data compiled from the literature on a range of sands. They showed that when the ratio of G/G_{max} is plotted against strain, the results tended to

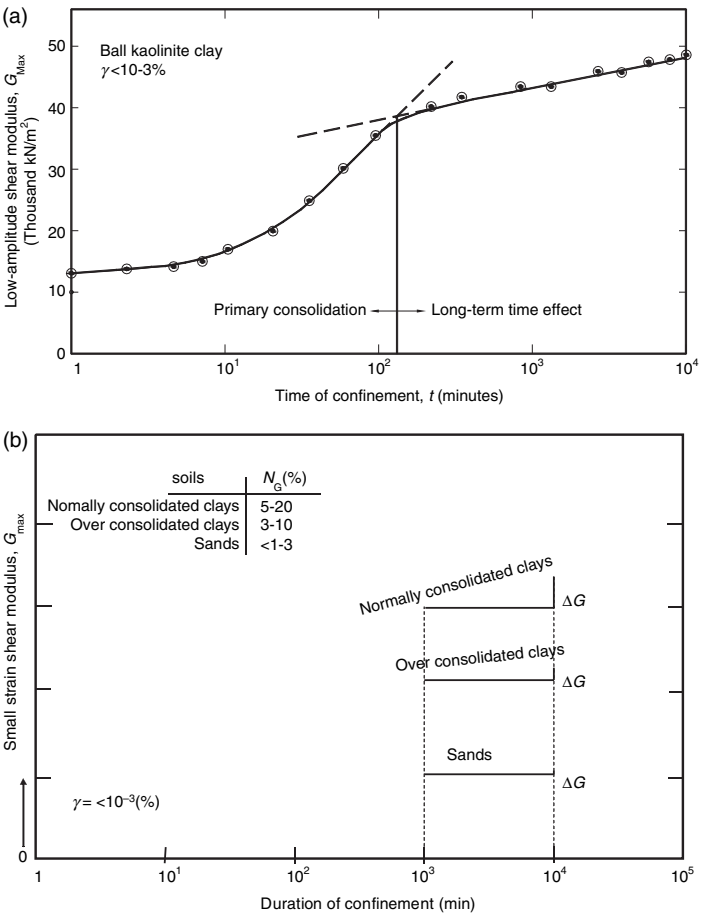


Figure 24.6 Variation in small strain shear modulus with time of confinement for (a) ball kaolinite clay sample showing primary and secondary consolidation, and (b) a range of geomaterials

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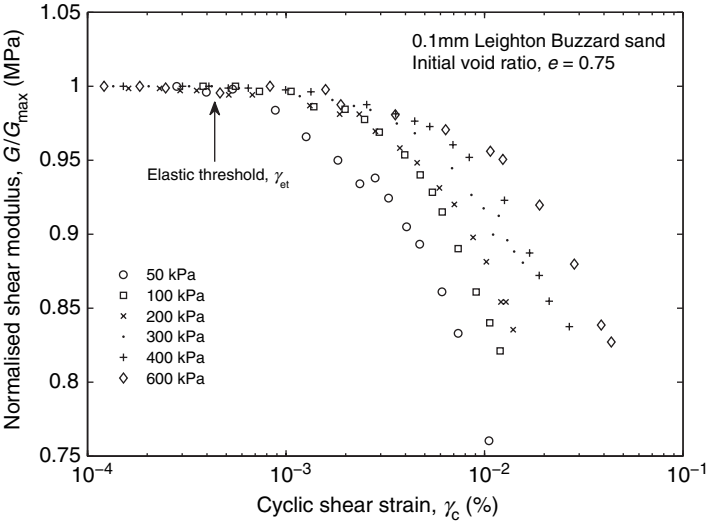


Figure 24.7 Changes in normalised shear modulus, G/G_{max}, as a function of applied cyclic shear strain at different effective confining pressures

fall within a narrow band for a given confining stress. **Figure 24.7** shows the results of resonant column tests conducted by the author using 0.1 mm diameter Leighton Buzzard sand. The results show a similar trend to that identified by Seed and Idriss (1970) and others. It can be observed that at a certain level of strain, the shear modulus starts to reduce with increasing strain. This point is referred to as the linear elastic threshold shear strain, γ_{et}. As for G_{max}, the shear modulus reduction curve is influenced by effective confining pressure. At higher confining pressures, γ_{et} increases, i.e. the linear elastic region occurs at higher strain and can be considered as shifting the whole curve to the right.

Recent research (Cho *et al.*, 2006; Bui, 2009) showed that soil attributes such as particle shape and surface roughness also influenced γ_{et}; Increased particle angularity and surface roughness increased γ_{et}, while more rounded particles such as glass ballotini gave a value of γ_{et} of around 10^{-6} (see **Figure 24.8**) such that there was little or no observable G_{max}. Vucetic and Dobry (1991) reviewed the results obtained for a broad range of materials, especially those for high plasticity clays. Their results, as presented in **Figure 24.9**, showed that γ_{et} was dependent on the plasticity of the soil and not its void ratio. They showed that as soil plasticity increased, γ_{et} increased. In addition it was shown that the influence of effective confining pressure was reduced as soil plasticity increased. It was also observed that there was a high degree of similarity between the curves for soils with a plasticity index (PI) of zero, and those obtained from Seed and Idriss (1970) on sands. Coarse gravels (Seed *et al.*, 1986) showed a similar response to sands, although the shear modulus reduction curve was slightly flatter.

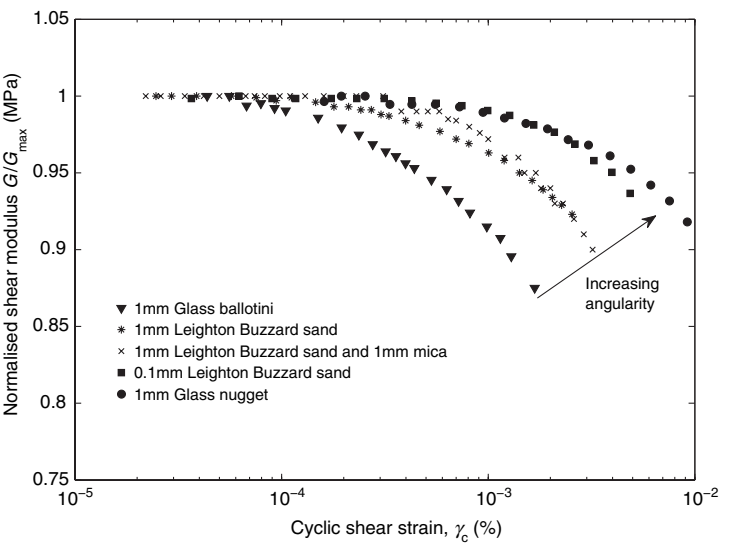


Figure 24.8 Changes in normalised shear modulus, G/G_{max}, as a function of applied cyclic shear strain at $\sigma'_0 = 100$ kPa for a range of geomaterials. Increasing particle angularity and surface roughness increase the elastic threshold strain, γ_{et}

Data taken from Bui (2009)

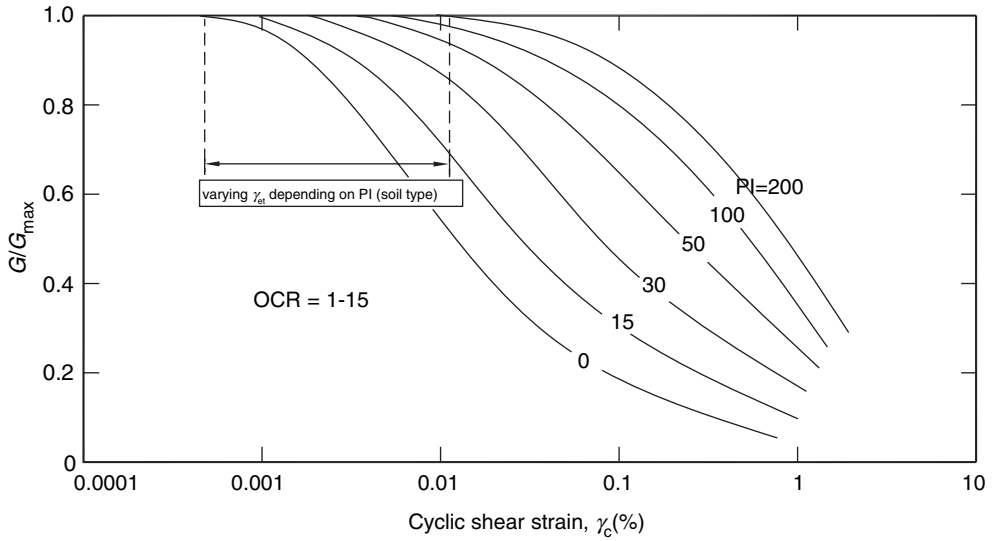


Figure 24.9 Small strain response of clays with different values of PI
Reproduced from Vucetic and Dobry (1991) with permission from ASCE

24.4.2.3 Damping ratio

Theoretically, for soils subject to shear strain within the elastic region (~10^{-6}) no hysteretic energy is dissipated, since frictional losses through grain slippage and yielding are thought not to occur (Winkler *et al.*, 1979). However, even within the elastic limit a minimum value of damping is observable; this is often termed as D_{min} and may result from fluid squirt flow at grain contacts, or from Bauschinger effects (plastic yielding of individual grains at grain contacts. It is also important to realize that 'equipment damping – due to the apparatus' is also present when measuring material damping, and therefore careful calibration is required to remove this from the measured data (Priest *et al.*, 2006). This is important at low strains where the equipment damping may be the same order of magnitude as that of the material being tested.

Studies into the factors affecting damping ratio, D, are not as extensive as those for G. In equation (24.10), several factors were highlighted as having an effect on damping. It has been shown that D is strongly influenced by γ_c and σ'_0. **Figure 24.10** highlights the measured damping ratios as a function of both γ_c and σ'_0 for the same samples whose normalized shear modulus was presented in **Figure 24.7**. It can be seen that D reduces for increasing σ'_0 and increases for increasing γ_c. As σ'_0 increases, slippage at particle contacts is restricted thereby leading to a reduction in D, while at higher strains the shear stress at particle contacts is increased, leading to increased frictional losses. **Figure 24.11** highlights the influence of particle angularity on the measured damping ratio for a range of geomaterials. The results show that increased angularity leads to reduced damping for the range of geomaterials tested. γ_c and σ'_0 have major effects on D, but the time of confinement and e have only minor influences. The number of loading cycles is important at high strain amplitudes, but becomes less important as the induced strains become small.

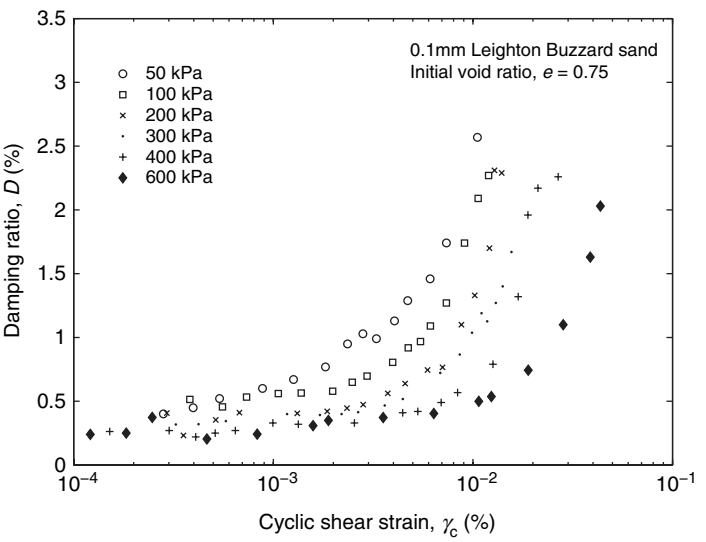


Figure 24.10 Influence of confining stress on the strain dependent damping ratio for Leighton Buzzard sand

24.5 Liquefaction of soils

The discussion above highlighted the nonlinear response of soils at strains from 10^{-6} to ~10^{-3}, where the deformation of the soil is negligible during cyclic loading. However, at higher strains, a phenomenon called soil liquefaction can occur in sands. During cyclic loading (such as that caused during an earthquake), frictional sliding and yielding occur at grain contacts. These lead to changes in soil volume, producing a build-up of pore pressure and subsequent reduction in effective stress, allowing large shear deformations to occur transforming the material from a solid to a liquefied state (Youd and Idriss, 2001). These shear deformations can be induced over short timescales and

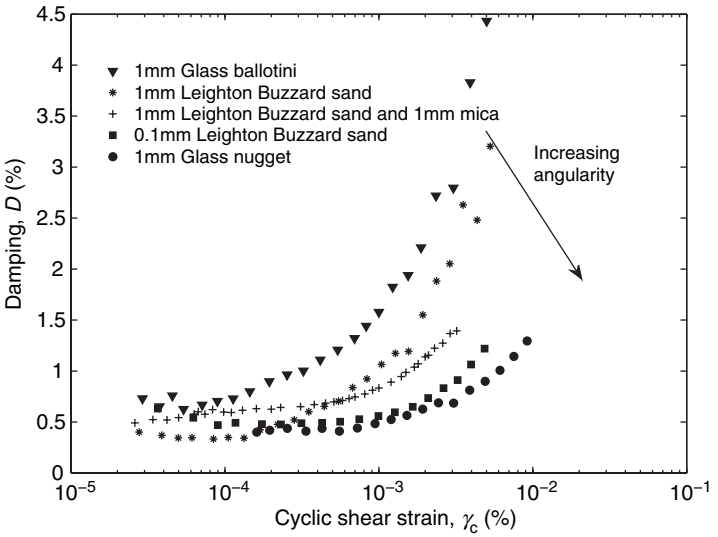


Figure 24.11 Influence of particle shape and roughness on the strain dependent damping ratio for a range of geomaterials at $\sigma'0 = 100$ kPa
Data taken from Bui (2009)

can lead to catastrophic failure of structures located on these soils (**Figure 24.12**). This dramatic change in behaviour is usually manifested in loose to moderately dense sands capped by impermeable layers, or in silty sands that have poor drainage properties; both prevent the high pore pressures induced in the sand from dissipating rapidly. For moderately dense to dense soils, liquefaction causes transient softening of the material and increases in shear strain. However, the tendency for dense soils to dilate during shearing inhibits major strength losses and restricts the development of large ground deformations. Although liquefaction has been predominantly reported in sands and silty sands, etc., significant ground deformations have also been induced in fine-grained clay soils. Boulanger and Idriss (2004) suggest that fine-grained soils can be classified as 'sand-like' (PI < 7) where the soil is liable to suffer liquefaction, or 'clay-like' (PI > 7) where the soil is liable to suffer cyclic failure. The terms 'liquefaction' and 'cyclic failure' do not imply strong differences in the observed stress–strain response of the soil during cyclic loading. They are used to identify soils where the mechanical behaviours are very different from each other and so require different procedures to be adopted to evaluate their seismic behaviours. A detailed discussion about the liquefaction or cyclic behaviour of fine-grained soils can be found in Boulanger and Idriss (2004).

The capacity of a soil to resist liquefaction is represented by the cyclic resistance ratio (*CRR*). From laboratory tests the *CRR* is defined as the point where the ratio of shear (τ) and mean effective (P′) stresses produces a 5% peak to peak amplitude shear strain, or when the buildup of pore pressure in the soil equals that of the mean effective stress. To constrain the tests, the number of load cycles to achieve liquefaction is required. An arbitrary value can be taken if relevant correction factors are applied, but given the number of significant cycles present in actual time histories, a value of 10–20 cycles is usually appropriate. **Figure 24.13** shows the typical response of saturated loose sand during cyclic loading. It can be seen that for each successive stress cycle a corresponding increase in pore pressure occurs. As the pore pressure increases and approaches that of the applied mean effective stress, large strains are manifested. Factors that influence the onset of liquefaction include initial confining stress, cyclic shear stress (intensity of shaking) and initial void ratio (relative density). In the application of constant shear stress, factors such as soil fabric are inherently important with regard to the pore pressure response. Therefore tests using constant shear strain amplitude are preferred for remolded sands, since the pore pressure response under constant strain is not significantly affected by the disturbance. **Figure 24.14** highlights the response from a cyclic triaxial test for medium dense sand with constant strain amplitude. As for constant shear stress test, pore pressure increases for each cycle with the shear stress reducing so as to maintain the constant shear strain amplitude. The stiffness response of the soil during constant strain amplitude can easily be incorporated into earthquake response programs.

Due to the inherent difficulty in obtaining undisturbed granular samples for testing and these soils being the most likely to liquefy, field tests are routinely carried out to assess liquefaction potential using techniques such as the SPT and the CPT, as well as measurement of shear wave velocity, V_s. Correlation curves have been developed over the years to relate test results to the *CRR*. Various corrections are required for these curves to take account of factors such as fines content, soil plasticity, soil type and overburden pressure, as well as test-specific correction factors. A detailed discussion of these can be found in Youd and Idriss (2001) and Idriss and Boulanger (2004).

24.6 Summary of key points

- The ground on which structures are founded can be subject to dynamic loads from both natural (earthquake, wave, water) and



Figure 24.12 Aerial view of leaning apartment houses in Niigata, Japan, produced by soil liquefaction during the 1964 earthquake
Photograph, in the public domain, courtesy of NOAA/NGDC (www.ngdc.noaa.gov)

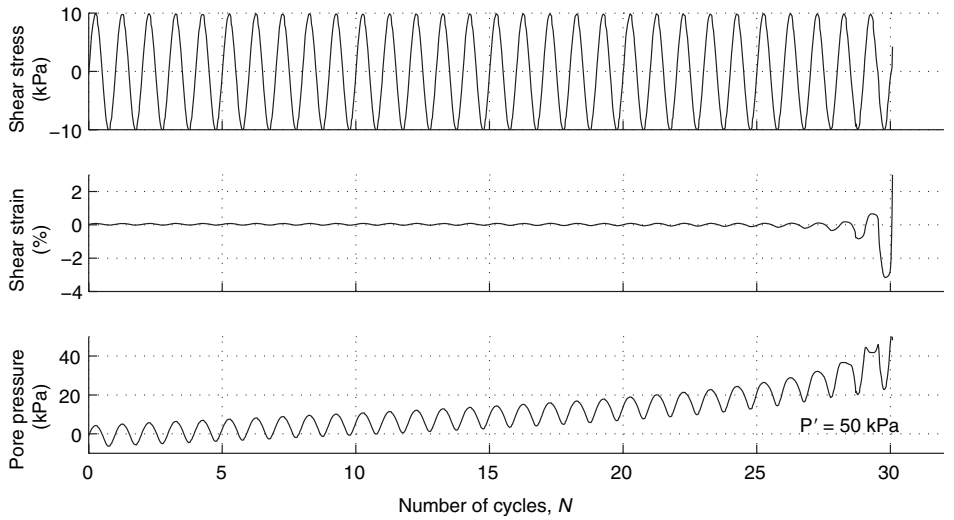


Figure 24.13 Shear stain and pore pressure response from cyclic triaxial test for applied constant stress for a loose/medium dense sand
Data courtesy of Fugro GeoConsulting Ltd

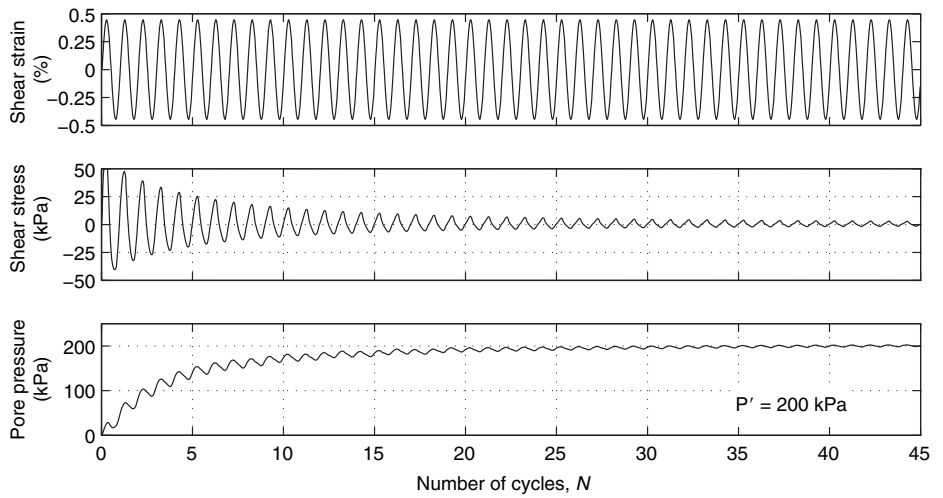


Figure 24.14 Shear stress and pore pressure response from cyclic triaxial test for applied constant strain
Data courtesy of Fugro GeoConsulting Ltd

man-made sources (vibrating machinery, traffic, bomb blasts), which generate strains in the soil (10^{-6}–10^{-2}) that are generally much lower than most static loads.

- Dynamic loading can be characterised by the frequency of the applied load (generally greater than 1 Hz) and the number of load cycles, which can vary from 1–2 (bomb blasts) up to many thousands (traffic loading and vibrating machinery).
- All soils exhibit nonlinear stress–strain behaviour where permanent, non-recoverable deformation of the soil occurs. At very low strains (10^{-6} to 10^{-4}) soil behaviour could be described as elastic, with recoverable deformations.
- Dynamic soil properties can be ascertained using field or laboratory techniques through either determining the velocity of wave propagation through the soil, or by direct measurement of the stress–strain response of the soil. The measurement techniques adopted will depend on what soil parameters are required and at what strains the measurements can be determined.
- The nonlinear behaviour of soils can be described using a number of nonlinear constitutive models, which can be adopted in numerical analysis. The key parameters for these models, such as very small strain shear modulus (G_{max}), and shear strength (τ_f) can be derived for experimental data.
- The shear modulus and damping in soils are influenced by a range of parameters, with the most important parameters being cyclic shear strain amplitude (γ_c), effective confining stress (σ') and void ratio (e). For clay soils both OCR and time of confinement (t) have also been shown to be important.
- At strains greater than 10^{-3}, frictional sliding and yielding at grain contacts can lead to changes in volume of the soil producing a build-up of pore pressures and subsequent reduction in effective stress, allowing large shear deformations to occur in the soil. This

behaviour is usually observed in loose to moderately loose sands and termed liquefaction. In dense sands, the tendency to dilate during shearing inhibits major strength loss.

- Fine-grained soils can be classified as having 'sand-like' behaviour and liable to liquefy, or exhibit 'clay-like' behaviour, where cyclic failure is likely to occur.

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24.7.1 Further reading

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

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Chapter 25

The role of ground improvement

Chris D. F. Rogers University of Birmingham, UK

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Ground improvement can take many forms to cause temporary or permanent change, usually with some specific engineering purpose in mind. Common with all geotechnical design, specific engineering purposes (e.g. stable foundation) must be defined and the ground's current state in the zone of influence fully established. Analysis of the problem in light of this information usually yields a range of solutions, some of which might be altering the ground's properties. Techniques are available to strengthen or stiffen soft or weak ground, either by physically altering its structure or by changing the ground's chemical properties. Similarly, groundwater flow regimes can be altered to achieve greater stability or to stop flow of contaminants. Any analysis of ground treatment must therefore encompass the engineering purpose(s), the ground's current state and its potential to be altered, the proposed means of its alteration, and any side effects of treatment. Physical ground improvement includes application of static, vibrationary or dynamic loading; water can be drained using gravity, increased stresses or an electrical gradient; and chemicals can be introduced as solids or liquids. Hybrids of these processes and lateral-thinking approaches (e.g. ground freezing) complete a range of alternatives to 'structural' solutions and often provide more sustainable outcomes.

25.1 Introduction

Ground improvement covers any method by which the ground, whether natural or disturbed in some way by anthropogenic processes, has its performance for any specific (geotechnical) purpose enhanced. Typically the goal is strengthening and/or stiffening of soft or weak soil or fill, either temporarily or permanently, although other specific improvements (e.g. reduction or increase of hydraulic conductivity) might bring about the desired change or be the goal of the exercise. Equally the goal might be to make the properties worse (e.g. slurries introduced at the face of tunnelling machines to facilitate excavation and transportation of soil), while some techniques have unwanted side effects (e.g. vertical stone columns provide a route for venting gases), so the term 'improvement' should be treated with caution. Nevertheless it can be generally stated that a ground improvement process has some associated desired physical or chemical consequence, such as:

- settlements of overlying or embedded structures being reduced, speeded up, or rendered unimportant;
- the risk of failure being removed or reduced;
- the need for excessive or unnecessary use of natural resources (import of virgin material to a site) and/or waste generation (poor ground disposed of in landfill) being obviated;
- the movement of chemicals being enhanced (for extraction) or inhibited (for containment);
- the excavation of contaminated ground for treatment, thereby exposing it to the environment, being avoided.

Ground improvement for physical purposes is generally an alternative to structural solutions which either bypass the soils concerned (e.g. deep foundations) or act in spite of the soils concerned (e.g. rigid raft foundations). Ground improvement for chemical purposes might be used instead of permanent physical solutions (e.g. barriers to stop chemical contaminant migration), novel semi-permanent processes (e.g. permeable reactive barriers; see Boshoff and Bone, 2005) or costly temporary operations (such as pumping for groundwater lowering to generate an inward hydraulic gradient). Phear and Harris (2008) track developments in this subject.

There are very many reasons for undertaking some form of ground improvement. As with all engineering undertakings, the costs and benefits of the alternative engineering solutions must be examined, and this should be done using the broad three pillar (environmental, social and economic) or four pillar (adding natural resources, see Braithwaite, 2007) sustainability models (Fenner *et al.*, 2006). This is a most important aspect of an engineer's role due to the wider impact of geotechnical processes (Jowitt, 2004) – in the category of resource use falls energy (embodied and directly used) and hence CO_2 emissions, water, and similar concerns. The sustainability arguments are highly germane to ground improvement; if what is in place can be made adequate by some means, then the environmental and social (and resource) benefits are usually very substantial indeed, while economic benefits are also often favourable. The civil engineering industry has now moved to a state of global awareness that demands it goes beyond the traditional 'cost, quality and time' approach to engineering design. Ground improvement alternatives to 'structural' solutions represent an excellent example of how these considerations play out.

The aim of this chapter is to introduce the fundamental principles of ground improvement techniques, the detailed manifestations of which can be seen routinely in academic or professional journals. The scope of this chapter is limited to hydraulic, physical and chemical means of improvement. Soil reinforcement via the introduction of tensile elements

in the form of grids, strips, bars, fibres or similar is covered elsewhere (see Chapter 86 *Soil reinforcement construction*). Uncommon techniques, such as the thermal treatment of clays and the use of bitumen, fly ash or other 'non-standard' additives to improve soil properties, lie outside the scope of this manual. For a more complete review of ground treatment techniques, refer to Mitchell and Jardine (2002). Hereafter the term 'soil' will be used to represent undisturbed natural, disturbed natural and artificial ground (such as fills).

25.2 Understanding the ground

Common with all geotechnical design, the specific geotechnical engineering purposes for which the site is to be used must first be defined. These purposes in turn will define what is expected of the ground. So, for example, if the site in question is to house a multi-story building, the expectation of the ground is to provide a stable foundation, whereas if a river is to be protected from contamination migrating from an adjacent industrial site, the expectation of the ground is to provide a barrier to groundwater flow. In light of these expectations, the current state of the ground in the appropriate zone of influence (the influences of the stresses imposed by the building in the case of the foundation, or the potential routes through which the contamination might flow to the river) must then be established. Moreover, the future state of the ground for the design life of the expectation must equally be anticipated.

A useful tool for this analysis is presented by the 'Burland Triangle' (**Figure 25.1**), which provides a framework in which to explore whether the expectation can be met by the ground as it exists and is likely to continue to exist on the site in question. A site investigation will be conducted to establish the ground profile and provide clues to its geological history, and this provides the first pointer to the future likely state of the ground; for example, whether exposure might lead to further weathering of the natural soil and a change in its properties. If the building is to be founded on a clay soil at a certain depth below the ground surface, for example, then the net stress increase (maximum likely stress imposed by the building minus the stress relief due to excavation of soil above the foundation depth) must be determined, although of course it could be a stress reduction were it a lightweight structure founded at depth. The make up of the soil (see Chapter 14 *Soils as particulate materials*) will usually be established by the site investigation, while the properties of the soil (strength and deformation behaviour, see Chapter 17 *Strength and deformation behaviour of soils*) will likewise be established via laboratory testing; for a clay soil the deformation may take several years before it is complete.

Some form of modelling will determine whether, in general terms, the ground can fulfil the geotechnical expectation placed on it, taking cognisance of the ground properties, while an understanding of the geotechnical context of the wider area will provide clues as to whether the ground properties are likely to remain stable in the long term. Groundwater flow regimes (see Chapter 16 *Groundwater flow*) are important here, as they

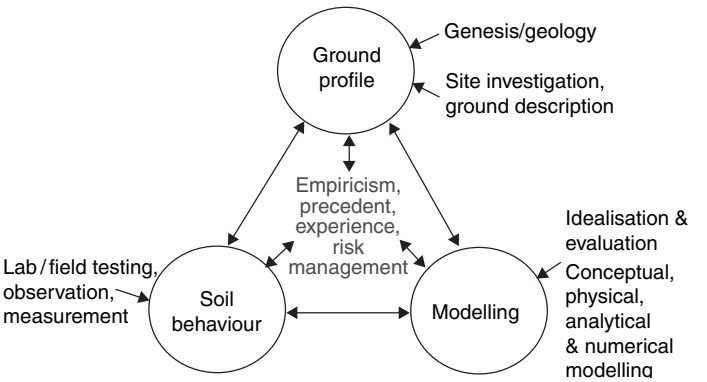


Figure 25.1 The 'Burland Triangle' for analysing the ground and how it might react for a geotechnical engineering purpose (Burland, 1987). Ground improvement processes can be analysed in the same manner

are for the second example of contaminant transport. Similarly, changes to the groundwater profile are important since they will alter the effective stress regimes operating on the site (see Chapter 15 *Groundwater profiles and effective stresses*), which in turn will alter the soil properties. The important point here is that sites are dynamic – the ground properties can change with time, and potential future changes must be accounted for in any geotechnical design, whether it is a conventional 'structural' solution (concrete foundation or cut-off wall) or a ground improvement process that is adopted. Analysis of the problem in the light of the totality of the above information will usually yield a range of solutions, some of which might be to alter the ground's properties. The remainder of this chapter is devoted to outlining the ways in which ground properties can be altered to obviate the need for more elaborate 'structural' solutions, and thereby provide additional weapons in the geotechnical engineer's arsenal.

25.3 Removal of water

25.3.1 Introduction

Water in soil is usually a nuisance (it gets in the way of construction operations) and, more importantly, the strength of a soil is usually very sensitive to both water content and water pressure (see Chapter 15 *Groundwater profiles and effective stresses*). Water is added to soil in certain situations (for example, to aid compaction of a granular soil or to weaken a clay soil to aid excavation) but on the whole, it is a hindrance. The problems posed by water in the context of ground improvement are generally that there is too much of it and/or it cannot escape quickly enough when additional, superimposed stresses are applied to the soil. Moreover, if water flows it can strengthen or weaken the soil according to the direction of flow and the pore water pressure distribution generated by the flow, which can be determined from a flownet (see Chapter 16 *Groundwater flow*). Only saturated soil (soil mineral particles and water) will be considered in terms of the need for water removal, which is the primary focus of the discussion hereafter. Natural groundwater

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flow is excluded from this discussion, whereas techniques that induce the flow of water out of the soil are central to ground improvement.

From Terzaghi's law of effective stress

$$\sigma' = \sigma - u \quad \text{or more generally} \quad p' = p - u \qquad (25.1)$$

where σ' is the effective normal stress on any given plane in a soil mass, σ is the equivalent total normal stress, u is the pore water pressure, p' is the mean normal effective stress in three dimensions and p is the equivalent mean normal total stress (all measured in kPa). The parameter σ' (or p') is the factor which determines strength of the soil on any given plane (or in three dimensions). If the pore water pressure increases above its original level, σ' (or p') reduces, and *vice versa*.

Given that the pore water pressure increases linearly with depth below the groundwater table in the majority of cases (see **Figure 15.3**, Chapter 15 *Groundwater profiles and effective stresses* and associated text for exceptions to this rule) and water is held by capillary suction such that the suction (or negative pore water pressure) increases with height above the water table until the suction can no longer be maintained (which can be many metres in fine-grained soil having very small pores), one obvious means of ground improvement is to lower the groundwater table by some means of drainage. This principle stood the Romans in good stead when they were creating their roads, or (literally) their highways, in areas with a naturally high water table which adversely influenced the properties of fine-grained soil. Of course the same principle informs our current drainage specification for roads, railways, airport runways and other such traffic-carrying infrastructure. By digging deep ditches on either side of their roads, the Romans depressed the water table, thereby increasing the suction in the soil above the water table, or making u (the pore water pressure) more negative in equation (25.1). This in turn increased σ' (or p') and made the soil stronger (see **Figure 25.2**). Not only this, but by creating a cambered surface and paving it with stone flags, they minimised the possibility of water entering the road pavement structure from above and thereby maintained the suction regime. Drainage is therefore an assured means of ground improvement and is used widely: behind retaining walls, in slope faces and adjacent to dam cores, for example.

Fine-grained soil formation typically involves soil particles transported by water and settling in slow flowing conditions, such as in the base of lakes or the sea. The soil starts off as very wet indeed, but as more soil particles are progressively laid down above it, the soil densifies as water is progressively removed from the soil – it is squeezed out as the vertical (and hence mean normal) stress builds up by a process known as consolidation. The relationship between volume and mean normal effective stress is described by the normal consolidation line (NCL) and is shown using critical state theory terminology in **Figure 25.3**. A natural soil may undergo sequences of erosion and deposition, whereby the effective stress it experiences reduces and increases. One such sequence is shown in **Figure 25.3**, in which the soil is progressively consolidated as p' increases past points A and B until point C is reached. After this, p' reduces (due to erosion) until point D is reached. The degree to which the soil swells back relative to the projection of the critical state line (CSL) is important in determining the future response of the soil to applied stress regimes. The point on the NCL at which the soil at point D (and at any point on the Swellback Line, which is idealised as a straight line here for simplicity; in truth it is curved) has experienced its maximum p', is termed the *preconsolidation pressure*. If the soil at point D is thereafter reloaded, it will recompress and its state will move down the path of the recompression line

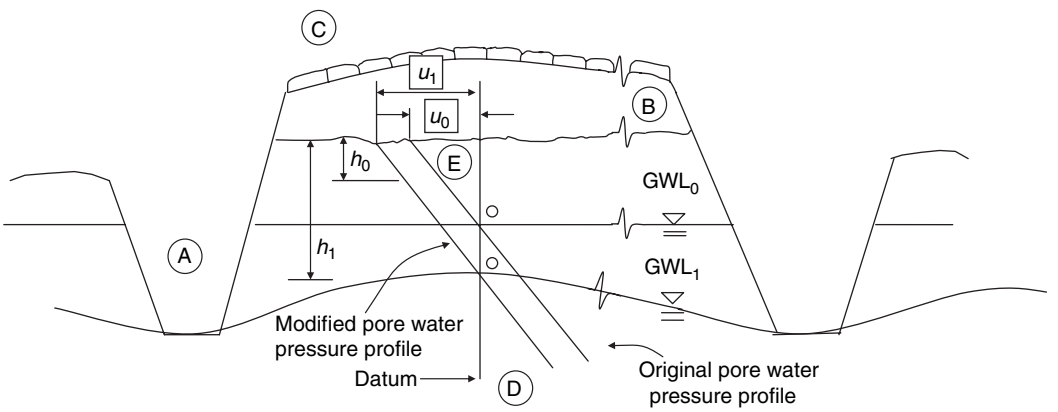


Figure 25.2 The effects on the strength of soil due to groundwater table lowering, as illustrated by a Roman road construction
A – Deep ditches on either side of road lower groundwater level (from GWL_0 to GWL_1)
B – Excavated soil creates highway
C – Stone flags on cambered surface limit surface water ingress
D – Pore water pressure increases linearly with depth below GWL
E – Suction, or negative pore water pressure, increases linearly with height h above GWL such that suction $u_1 > u_0$

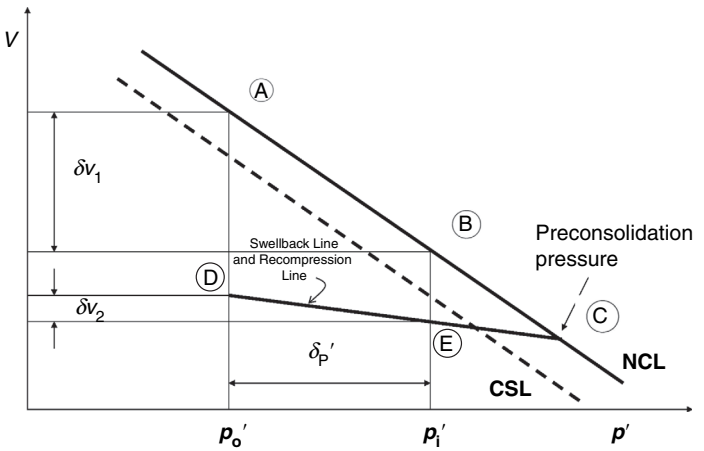


Figure 25.3 Graph of volume against mean normal effective stress

(idealised here as linear and coincident with the Swellback Line) to reach point C. It will then follow the NCL downwards once more.

In practice, if we encounter the soil at point D in equilibrium ($p' = p_0'$), and apply a superimposed stress to it of $\delta p'$, say due to a new building, according to the idealised graph the soil state would move to point E and the amount of volume change would be δv_2 once consolidation due to the imposed stress was complete, i.e. once water had been caused to flow out of the soil and a new equilibrium at $p' = p_1'$ had been reached. If, however, the soil encountered was at point A, also in equilibrium at $p' = p_0'$, and a similar stress of $\delta p'$ was imposed on it due to construction of the same building, then the far greater volume change of δv_1 would result once a new equilibrium at $p' = p_1'$ had been established as the soil state moved to point B; far more water would have been driven out of the soil to achieve this state. For practical purposes, therefore, it is desirable to ensure that the stresses imposed by construction activities remain below the preconsolidation pressure. One form of ground improvement is to create this situation, i.e. induce artificial overconsolidation.

The velocity (v in ms^{-1}) at which water flows through a saturated soil, which is a particulate material with interconnected pore spaces, is governed by the driving force (the hydraulic gradient, i, which is dimensionless) and the resistance to flow (the hydraulic conductivity, k in ms^{-1}) according to Darcy's Law:

$$v = ki. \tag{25.2}$$

When an increment of total stress (δp) is applied to a soil, this stress is instantaneously carried by the pore water and creates an excess pore water pressure of $\delta u = \delta p$. Water flows from high to low pressure and therefore away from this area of high pressure, thereby progressively reducing δu to zero, causing $\delta p'$ to rise progressively until it increases by δp. The driving force, i.e. the hydraulic gradient, is therefore due to the excess pore water pressure induced by the increment of stress (δp). In effect we have created a seepage problem, for which isochrones could be drawn to illustrate the direction of flow and progression of consolidation within the soil. In fine-grained soil, the rate of flow is small, with coefficients of hydraulic conductivity ranging from ~10^{-5} ms^{-1} for clean silts to ~10^{-11} ms^{-1} for heavy clay soils, while the distance that the water has to flow clearly governs the time required for consolidation to occur.

25.3.2 Pre-loading

Returning to the previous example illustrated in **Figure 25.3**, if we encounter a soil at point A and apply temporarily a stress equal to (applying $\delta p = \delta p'$ would take the state to point B) or greater than (say to point C) the stress subsequently to be imposed by the building, and were to allow the stress to remain in place for a sufficient length of time for consolidation of the soil to occur fully, i.e. for the water to flow out and equilibrium to be regained, then removal of the temporary stress and imposition of the new stress due to the building would cause only a small amount of volume change. Unloading (by removal of the temporary stress at points B or C) would cause the soil to swell, i.e. move upwards along parallel Swellback Lines, while the addition of the building would recompress the soil, i.e. move downwards along the relevant Recompression Line (being coincident with the Swellback Line). If this process of stress removal and replacement by the building were to take place quickly, and/or if the soil had a small coefficient of hydraulic conductivity (k), or the flow distance for the water to exit the soil is large, then the heave caused by the unloading and the compression caused by the building could be very small, and for buildings or foundations that are very sensitive to settlement, this process can be used to advantage to create negligible settlements.

This process is effective, especially for soils in which secondary compression (creep) is significant, but it is slow. One way to speed up the process is to increase the temporarily applied stress by increasing the driving force element. However, for thick soil deposits, especially where the direction of flow is upwards only (e.g. the soil overlies an impermeable rock), it can prove impractically slow; the rate of flow is proportional to the square of the drainage path. The temporary load can be earth or rubble fill, although water tanks or vacuum preloading (by pumping from beneath an impervious membrane) have been used. Placement of fill, ensuring continued drainage, is effected in layers as quickly as possible (staged loading is commonly needed to avoid overstressing the soft, weak soil being treated and avoiding failure); likewise, fill is removed as quickly as possible (staged removal is not necessary). The advantages are that this is a simple method that is easy to effect and certain to work, and is suitable for soils that undergo large volume decreases. It can, however, require a long time and much material, which has to be double handled.

25.3.3 Vertical drains

Although it adds to the cost and complicates the construction process somewhat, the time required for consolidation using

pre-loading can be greatly reduced by installing vertical drains. The rate of consolidation depends upon the length of the drainage path and the hydraulic conductivity of the soil associated with this path. In general, the water will flow vertically on loading to exit the soil and $k_{vertical}$, which is governed by the lowest coefficient of hydraulic conductivity of the substrata (e.g. interlayered fine sands, silts and clays in alluvium), will determine the rate of flow. By permitting horizontal drainage to vertical drains installed at predetermined intervals, not only is the length of the drainage path greatly reduced, but the rate of flow is governed by $k_{horizontal}$, rather than $k_{vertical}$. In general, the horizontal coefficient of hydraulic conductivity is far greater since it is the higher of the values of the substrata that dominate. For interlayered sands, silts and clays, this can mean a few orders of magnitude. This is why normally consolidated or lightly overconsolidated alluvial soils and stratified soils are most effectively improved using vertical drains. Types of vertical drains include sand-filled boreholes (though horizontal shearing would compromise their performance), sand-filled stockings that are inserted into vertical boreholes, and proprietary band drains consisting of a strong, flexible core (manufactured from polyethylene or similar) containing drainage channels and covered by a filter fabric. A horizontal free draining layer is constructed at the top of the vertical drains prior to application of the pre-load. Connection with an underlying permeable layer (e.g. sand or gravel) is also achieved where possible. Smearing of the borehole sides should be avoided so that horizontal drainage is not compromised (Hird and Moseley, 2000).

25.3.4 Electro-osmosis

When a voltage gradient is induced across a soil, the water in the pore structure of the soil moves from the anodes (often metal rods) to the cathodes (e.g. perforated metal pipes creating a series of filter wells), which are water abstraction points. The electro-osmotic flow of water through soil is expressed by:

$$v_e = k_e\, i_e \qquad (25.3)$$

where v_e is the electro-osmotic flow rate, k_e is the coefficient of electro-osmotic permeability (measured in m^2/Vs) and i_e is the potential gradient = V/L (where V is the potential difference and L is distance between the electrodes). This is analogous to Darcy's Law for hydraulic flow (equation (25.2)), yet there is a fundamental difference since k_e is commonly considered to be independent of the size of the pore spaces, while k is very strongly influenced by pore size. The technique is therefore most effective in fine-grained soils, in which k_e is typically 10^{-8} to 10^{-9} m^2/Vs (Mitchell and Soga, 2005, p. 275) whereas k can vary by orders of magnitude down to ~10^{-11} ms^{-1} (Mitchell, 1993, p. 270 provides example comparisons). Electro-osmotic flow is primarily caused by cations in the water surrounding negatively charged clay particle surfaces being transported towards the cathode drawing water (a polar molecule that is attached to the cations) with them. (Very occasionally, positively charged surfaces surrounded by anions will result in flow towards the anode.)

There are many phenomena acting (see Liaki *et al.*, 2008), and some can bring about additional improvements:

- Degradation of a metallic anode and transport of metal ions by electromigration can cause improvement of clay soils via cation exchange on the exchange sites of clay minerals.
- Transport of stabilising agents added as aqueous solutions (electrolytes) at the anode (anolyte) and cathode (catholyte) can result in stabilising reactions.
- Electrokinetic remediation can be used to draw contaminants to the electrodes where they can be removed, hence remediating chemically contaminated sites (Ottosen *et al*., 2008).

Electro-osmosis can be used for temporary dewatering in soils not readily treated by well pumping, or for permanent improvement, especially when accompanied by chemical changes. Moreover, it is fast and can be used in a confined area, though it remains little used in practice. The soil immediately adjacent to the anodes tends to dry out, causing increased resistance and current requirements, but can be countered by periodic short duration reversals of polarity or by an appropriate anolyte solution. Mitchell and Soga (2005) is a particularly helpful reference on the subject of ion movements.

25.3.5 Groundwater lowering

Well pumping is conceptually simple, being more of an accepted construction technique than a geotechnical process, and works best in soils with relatively high coefficients of hydraulic conductivity such as sands and gravels (see Chapter 80 *Groundwater control*).

25.4 Improvement of soils by mechanical means

25.4.1 Introduction

Compaction is widely used to increase the density of soils, and thereby increase their strength and stiffness, notably in road construction where surface compaction technology has reached a considerable level of sophistication. The principal goal is to cause the soil particles to pack together as closely as possible so as to minimise voids and to maximise 'particle interlock' – so that when the soil is sheared, the soil in the sheared region must dilate (i.e. particles must do work against the normal, or confining, force) as well as doing work against the frictional force. The types of rollers available include smooth-wheel dead weight rollers, vibrating rollers and sheepsfoot rollers (see Chapter 75 *Earthworks material specification, compaction and control*). Compaction technology has developed into deep compaction for more general ground improvement based on both vibratory and impact methods. Explosives have also been used to induce deep densification (Hausmann, 1990), but this approach lies outside the scope of the standard approaches and is not covered. A static form of compaction is achieved by pre-loading, which is covered in section 25.3.2.

25.4.2 Dynamic compaction

Compaction of soil by the repeated dropping of weights on the ground has been practiced for centuries with the aim of improving the bearing capacity. The extension of this technique to dynamic compaction, involving heavy weights and large drop heights, was used in the early 1970s to provide a stable foundation for structures of large plan area and for reclaimed land development. The principle of operation is simply based on the application of large vertical stresses to a considerable depth, thereby compressing the soil skeleton directly where the degree of saturation (S_r) is low (Gu and Lee, 2002). Where S_r is not low, high excess pore water pressures are generated; if they are sufficiently high they will cause failure and fissures (or cracks) to be created extending from the surface to significant depths. As the excess pore water pressures progressively dissipate, aided greatly by the fissures providing convenient drainage paths, the bearing capacity of the soil increases such that it exceeds the previous maximum value.

Dynamic compaction is effective in coarse-grained fills (including decomposed domestic waste and industrial fills) and soils, but relatively ineffective in soft clays and peats that absorb the applied energy. The weights, or pounders, typically have a mass of 10–20 tonnes (100–200 kN) and are typically dropped, using a crane, from heights of 10–20 m at wide intervals initially, the spacing being progressively reduced. Repeated application of the pounder causes craters, which are backfilled with coarse fill prior to the next series of load applications. The surface layers require rolling at the end of the process. It is a simple, rapid, relatively inexpensive method of treatment involving large equipment (though it cannot be used in confined spaces) and considerable ground vibration (hence it cannot be used adjacent to sensitive buildings or structures). A depth of compaction of 5–6 m can be easily achieved; a 'rule of thumb' for the treatment depth given by Hausmann (1990) is $0.5\sqrt{(WH)}$, where W is the mass of the weights, and H the height from which they are dropped. The same principle is adopted by *rapid impact compactors*, which were developed to provide a more controlled, rapid and thus readily applied form of the technique (see Serridge and Synac, 2006) and which typically deliver an energy input of around 8 tonne metres at a rate of 40 blows per minute. Guided or leader dynamic compaction rigs are also available.

25.4.3 Vibrocompaction

Vibrocompaction (sometimes termed vibroflotation) aims to cause deep densification, and hence strengthening and stiffening, of a specifically designed volume of ground by means of lateral vibration using a poker lowered by a crane. The poker contains an eccentric weight mounted on a vertical shaft which is rotated by a motor. The poker vibrates laterally thus compacting the ground while sinking to the design depth under its self weight. This is known as the 'dry method' of operation; the 'wet method' involves forcing water through the end of the poker while it is lowered into the ground, thereby producing what is stated to be a 'flushing effect'. The principles that underlie the standard compaction curve (described in section 25.5.2 and **Figure 25.4**) also apply here. A void is thus formed within an area of dense, compacted soil. The poker is withdrawn and a granular fill is dropped into the void and is compacted in 250–500 mm lifts by lowering the poker to refusal. The purpose of the granular fill is to compensate for the reduction in volume due to compaction of the existing soil. Vibrocompaction thus aims to create a uniformly dense, uniformly stiff block of soil onto which to locate some type of foundation. The size of each treated block of ground is designed to accommodate the bulb of pressure beneath the foundation, i.e. the plan area of the treated block is square for a pad foundation (using typically 4, 9, 16 or 25 insertions) and linear for a strip foundation. The poker typically compacts the soil to a radius of 1.0–2.0 m from the insertion point, the volume decrease being generally about 5–10%, and occasionally up to 15% (see Chapter 85 *Embedded walls*).

Loose natural soils and fills ranging from fine sand to coarse gravel are most effectively treated, while coarser material is difficult to penetrate unless it is well graded. A high fines content prevents rapid dissipation of the excess pore water pressures that are generated by vibration in soils with a high degree of saturation. It also serves to dampen the vibration, hence reducing the area of influence of compaction. Cohesionless soils with less than 15% fines are consequently commonly quoted as being suitable; see Slocombe *et al.* (2000). The technique has also been widely applied to fills consisting of brick rubble, ash and general demolition debris, the voids in these materials being broken down by regular insertion of the poker and granular fill forced into the remaining spaces.

25.4.4 Stone columns

The same equipment and construction methods are used as for vibrocompaction, although a single-sized crushed rock is used as the granular fill. The end result is a 0.9–1.2 m diameter column of dense stone within a less competent stratum – typically a saturated weak/compressible fine-grained soil, though stone columns can in principle be installed in any soil type. In this case, the vibration does not necessarily influence the soil between the columns significantly, especially when the soil has a high fines content and a high water content. This will lead to largely undrained shearing displacements since there is no time for the water to flow out of the soil (although any air within the soil voids would of course be compressed). The process of adding stone and compacting it using the poker, however, will inevitably cause stone to be forced sideways into the soil until it is resisted. Three zones of material might be considered once the process is complete: largely undisturbed soil that has undergone some displacement due to undrained shearing, a soil–stone intermediate zone, and the compacted stone column itself. This technique is sometimes known as vibroreplacement or vibrodisplacement.

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The stone columns serve to reduce settlement of overlying foundations by acting as bearing piles and to strengthen the soil by virtue of zones of granular material having a much higher shear strength than the soil. It is also suggested that the stone columns act as vertical drains for the rapid dissipation of excess pore water pressures, although this remains a matter of debate associated with the degree to which the displaced soil fills the voids between the compacted stone. The depth of stone columns rarely exceeds 12 m, although lengths approaching 30 m have been recorded. The column fill typically consists of uniform 20–50 mm crushed stone, gravel or slag. Surface disturbance (due to a lack of confinement to resist the compaction energy) means that surface rolling is required after the columns have been formed (see Chapter 85 *Embedded walls*).

25.4.5 Micro-piles or root piles

Micro-piles were originally introduced for underpinning operations. They are now used for slope stability problems, retaining structures and underground construction. Micro-piles are constructed primarily to carry tensile and compressive forces and thereby increase the strength of soil for a particular purpose. They have only a limited degree of flexural (or bending) resistance and are therefore generally constructed at appropriate angles to carry direct forces. They can be used in most types of soil.

To create a micro-pile, a casing is jetted, driven or drilled to the desired depth, reinforcement is inserted into the casing and cement mortar is pumped in to fill the hole while the casing is withdrawn. In the case of root piles, the cement mortar is forced into the holes under higher pressure using a grouting technique so that it penetrates cracks or soft layers around the void. This creates a rough pile surface, thereby gaining maximum skin friction.

25.4.6 Soil nails, ground anchors and soil reinforcement

These techniques do not really lie under the heading of ground improvement and they are mentioned here only for completeness. Their purpose is to give the soil into which they are installed or buried an enhanced strength, either by providing tensile elements, increasing the mean normal effective stress, holding in place some form of structural facing to retain soil, or a mixture of all three. Soil nails are described in Chapters 72 *Slope stabilisation methods*, 73 *Design of soil reinforced slopes and structures*, 74 *Design of soil nails* and 88 *Soil nailing construction*. Ground anchors are described in Chapters 64 *Geotechnical design of retaining walls*, 87 *Rock stabilisation* and 89 *Ground anchors construction*. Soil reinforcement is described in Chapters 62 *Types of retaining walls*, 64 *Geotechnical design of retaining walls*, 72 *Slope stabilisation methods*, 73 *Design of soil reinforced slopes and structures* and 86 *Soil reinforcement construction*.

25.5 Improvement of soils by chemical means

25.5.1 Introduction

The addition of chemicals to clay soils to bring about improvements in strength, stiffness and volume stability (i.e. to inhibit shrinkage and swelling) or for other purposes, is hardly new – the Romans were adept at using the technique. The fact that lime has been used for this purpose in stabilising clays is equally unsurprising since limestone and shale (which is highly compressed clay) are the two primary ingredients of cement; cement factories being located where limestone and shale outcrop together. Other than the use of chemicals to dry very wet soils and make sites workable following heavy rain (quicklime is particularly effective for this), chemical soil stabilisation can be divided into two categories: (i) soil improvement as a result of the combined action of the soil constituents and the stabiliser to create a cemented product (as occurs by mixing lime and clay); and (ii) addition of a stabiliser that binds the soil particles together (as is the case with a sand-cement mix). The traditional manner of introducing the stabiliser is either as a powdered solid or a liquid suspension that is mixed with the soil at the surface, laid and compacted. However, more modern techniques have evolved to mix the stabilisers at depth or to pump stabilising solutions, such as cement grouts, into the ground. There are several applications (see for example Rogers *et al*., 1996) and while all have different requirements, the basic principles remain the same. Problems can be encountered when stabilising soils having a high plasticity index due to difficulties in achieving a thorough mix and ensuring a sufficiently large volume of stabiliser to chemically modify the clay minerals, leading to some concern over long-term softening due to water uptake.

25.5.2 Lime and cement stabilisation

Traditionally, lime has been used to improve the properties of clay soils by surface mixing, wetting where necessary, laying and compaction. Mixing lime with a clay soil alters its physical properties by fundamentally changing its nature as a result of cation exchange – a process termed *modification*. If sufficient lime has been added, and the modified material is compacted and allowed to cure for a significant period of time, the material cements by a process termed *stabilisation* (or *solidification*). The technique is most commonly used as a general ground improvement process, essentially to improve soft, weak clay soils, but it can also be used to treat contaminated soils.

Lime is a general term used to describe either quicklime (calcium oxide, CaO, created by heating hydrated lime to 450°C), hydrated lime (calcium hydroxide, $Ca(OH)_2$, which is formed when CaO comes into contact with water or water vapour) or lime slurry (a mixture of hydrated lime with water). Agricultural lime refers to calcium carbonate ($CaCO_3$), which is unreactive and will form if lime comes into contact with the atmosphere; carbonation must be avoided. In its simplest application, quicklime is used as a construction expediency to dry out sites that are unworkable due to water via a strongly exothermic reaction that turns CaO into $Ca(OH)_2$; dehydration taking place due to the reaction and to steam being released. Lime is simply spread evenly on the surface to effect this improvement, perhaps with some limited surface mixing.

Lime modification occurs as a result of cation exchange, the calcium ions exchanging with the cations on the clay mineral sites (usually monovalent sodium or potassium ions), which causes:

- a reduction in the thickness of the adsorbed water layer, and hence a reduced susceptibility of the clay to water;
- flocculation of the clay particles, due to greater electrical attraction (as the water layer has thinned);
- an increased internal angle of friction, i.e. greater shear strength;
- a textural change from a plastic clay to a friable material that appears granular in nature;
- a reduced plasticity as a result primarily of the plastic limit (PL) rising considerably.

To bring about this improvement the lime is spread evenly on the surface, mixed thoroughly with the clay to the required depth using a rotavator and allowed to mellow (or cure) for 24–72 hours. This timespan will ensure that all quicklime is fully hydrated (it hydrates via an expansive reaction and this must not be allowed to happen post-compaction), and that the cation exchange reactions are complete – the surface of the mixed material having been lightly rolled to seal it from rain and to inhibit carbonation. Lime modification can be achieved with the addition of a modest quantity of lime that is determined using the initial consumption of lime (ICL) test, which establishes the quantity of lime required to fully satisfy the cation exchange reactions. The ICL test is based on measuring pH changes in the soil, the ICL value being the lime addition that causes the pH to rise to 12.4 (or failing that, 12.3). This result can be confirmed by Atterberg limit tests of clay mixed with lime at different quantities.

To bring about the lime stabilisation reactions, sufficient additional lime is needed to raise the pH to a high level (>12). Also, once the cation exchange demand of the clay has been met, additional calcium ions are needed to react with constituents of the clay minerals (which are dissolved in the high pH environment) to achieve stabilisation via pozzolanic reactions (Boardman *et al.*, 2001). It is important that, as the reactions are taking place, there is a close proximity of the clay flocs so that the reaction products can bind the particles together in a coherent cemented mass. This process takes place in an environment that is free from chemical or organic contaminants that adversely affect the reaction processes. The reactions can be superficially stated as:

$$Ca^{2+} + OH^- + Al_2O_3 + H_2O \rightarrow \text{calcium aluminate hydrate gel}$$

$$Ca^{2+} + OH^- + SiO_2 + H_2O \rightarrow \text{calcium silicate hydrate gel}$$

and are the result of alumina (Al_2O_3) and silica (SiO_2) dissolving from the clay mineral under conditions of high pH. The two gels crystallise over time to yield strong, brittle bonds. This causes a considerable increase in strength and results in a material that is subject to brittle fracture (i.e. is non-plastic).

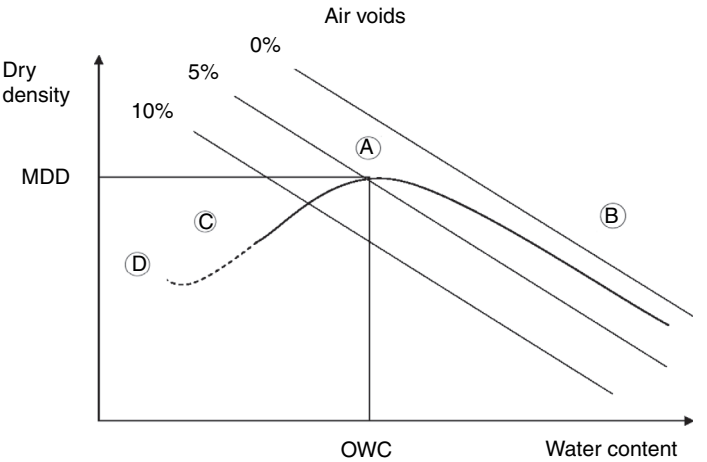


Figure 25.4 A standard compaction curve for fine-grained soil. Compaction seeks to maximise density and minimise air voids; for this application both of these compaction objectives are important
A – The maximum dry density (MDD) is achieved at the optimum water content (OWC) at which point the soil has minimum resistance to shear, hence densification
B – Additional water inhibits densification; undrained shearing occurs at progressively greater water content (hence lower dry densities)
C – As soil dries below OWC, suctions progressively increase thereby inhibiting shear under compaction stresses and the dry density falls
D – Where there is insufficient water, suction is lost and the dry density increases

The construction process is similar to that of modification, except that the mellowing process should not be as long (24 hours should be ample, and even this length of time has been considered to be detrimental to the long-term performance of the lime stabilised clay). The material should then be remixed, adding water if necessary to bring the water content up to the right level (between 0 and 2% above the optimum water content so that density is maximised while air voids are minimised, see **Figure 25.4**). It can then be placed and compacted in layers of appropriate thickness for the compaction energy applied, to ensure a density close to the maximum dry density throughout the full depth of the layer. The material should then be rolled with a smooth-wheeled roller if necessary to seal the surface while ensuring that there is a slight fall to avoid water ponding. Finally, it needs to be left to cure, free from frost and traffic until the reactions have advanced. The sulphate content must be sufficiently low (some specify <0.5%, others <1.0%) to avoid the creation of ettringite which forms via a highly expansive reaction, destroying the compacted state and compromising long-term strength. Organic material in the clay can be detrimental because it tends to be acidic, and hence lowers the pH, but more importantly calcium reacts preferentially with the organic material and thus less is available for the stabilisation reactions. The most assured method of design is to carry out laboratory trials of compacted mixes with different lime contents above the ICL value and allow them to cure for different periods before being tested for strength.

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Cement stabilisation can be used in any soil (there is no requirement for the alumina and/or silica from the clay mineral). The principles behind the technique and the processes of design and construction are similar. A larger quantity of cement than lime is usually required to provide sufficient binding agent to create a matrix sufficient to encompass the soil particles. Cement contains a significant proportion of quicklime and, like lime stabilisation, the use of cement causes a large rise in pH. This effect can be reduced if a cement replacement material, such as ground granulated blast furnace slag (GGBFS), is used.

25.5.3 Lime, lime-cement and cement columns

The modern techniques of lime column construction have been developed in Sweden and Japan, although there is now a far greater use of cement only and lime-cement mixes rather than lime only. This technique, commonly referred to as 'deep mixing', has generally only been carried out in soft/weak soils due to the difficulty of operating the mixing plant in stiff/strong soils. The principles listed above remain valid for the deeper process, notably the need for sufficient water for the reactions to take place, the mixing to be achieved thoroughly and the resulting material to be adequately densified. The mixing takes place with a purpose-designed tool that is augered into the ground; the densification occurs by counter-rotation of this tool such that there is downward pressure on the mixed material as the tool is withdrawn. The correct water content can be achieved by the inclusion of quicklime in a powder mix (nowadays consisting primarily of cement to guarantee the reaction products) to dry out wet soils, or, by using a cement slurry in cases where additional water is needed (see Chapters 84 *Ground improvement*).

25.5.4 Lime piles

An alternative stabilisation technique, commonly restricted to clay slope stabilisation, is to create lime piles, which are columns of compacted quicklime. The quicklime hydrates via the exothermic hydration reaction causing water content reduction and immediately reducing the pore water pressure and creating suctions; these have been found to arrest slope movement (Rogers *et al*., 2000a). The lime hydration releases calcium and hydroxide ions into the ground and they migrate from the quicklime pile into the surrounding soil, which is thus stabilised, to a distance that is dependent on the water content of the soil and the reactivity of the clay (Rogers and Glendinning, 1996). This distance is usually a few tens of millimetres, although clay cracking due to the rapid dehydration causes fissures that radiate from the pile and these provide conduits along which the lime migrates and stabilises the clay further away from the pile. The process can be enhanced by the inclusion of different additives, GGBFS being particularly effective (Rogers *et al*., 2000b).

25.5.5 Lime slurry pressure injection

Lime slurry pressure injection is another alternative that borders on the use of cement grout, but remains distinct since the aim of the process is to cause chemical changes in the clay soils into which the slurry is injected. A lime-water mix is injected at different depths into the clay on a grid pattern with the aim of creating an approximately uniform distribution of slurry. Migration of ions into the soil adjacent to the slurry seams results in the classical lime stabilisation reactions occurring. The technique has been used for clay embankment stabilisation in the USA, but has been little used in the UK and requires further research before its efficacy can be proved.

25.5.6 Grouting

Grouting has been practised for roughly 100 years and can be effected in many ways. The aim is generally to strengthen the soil by introducing a cementitious grout and/or by causing densification as a result of introducing the grout, or to prevent the flow of water by filling the voids in the soil. Essentially a fluid is injected into the soil and hardens at some later stage. The 'split spacing' method of group injection is generally used, whereby pre-determined quantities of grout are injected through holes spaced at two or three times the final spacing, the hole spacing gradually being reduced. The voids become progressively smaller and the ground progressively tighter. Grouting is covered in Chapters 84 *Ground improvement*, and 90 *Geotechnical grouting and soil mixing*, and so will be covered only briefly here.

25.5.6.1 Permeation grouting

Permeation grouting refers to the process in which no volume change or soil structure change occurs as the grout permeates the soil via the pore spaces. Once the grout is in position it hardens with time to strengthen the soil. Particulate grouts can be used where the soil is relatively coarse grained. Chemical grouts can penetrate smaller voids (medium silts or coarser), but are unsuitable for soils having more than 15% fines. Electrochemical injection can be used in silts and silty clays.

25.5.6.2 Displacement grouting

Displacement grouting refers to the process by which high viscosity grouts are pumped into the soil under high pressures with the aim of causing lateral or radial compaction of the soil. The grout usually consists of mixtures of cement, soil and/or clay, and water. Either spherical bulbs of grout are formed at the end of the grout pipe or columns of grout are formed by withdrawing the pipe. The process was first developed in the 1940s in France, but was of limited use until more recently developed pumping capabilities enabled the method to be more effective. The technique is applicable to soils which are readily compactable.

25.5.6.3 Jet grouting

Jet grouting is a variation that was introduced in Japan in the 1970s, the technique being based on the use of high speed water jets. The native soil is either mixed with a stabiliser (grout) *in situ* or, in the case of poor soils, the soil can be removed and grout columns formed in their place.

25.5.7 Artificial ground freezing

Artificial ground freezing is a construction expedient only. It is a temporary measure used to stabilise an area or zone of saturated soil while a particular construction operation is performed. It is a potentially dangerous construction process (significant precautions are necessary) and an expensive last resort, but it is a useful technique in certain cases.

The water within a soil mass is frozen by installing closely spaced freezing pipes through which a continuous supply of coolant is passed. The frozen soil has a greatly increased bearing capacity, a far higher strength and a permeability approaching zero. Excavation (for tunnels, underpinning, etc.) can proceed in safety once the ground is frozen.

Freezing causes a volume increase that is dependent on the water content of the soil, though a significant heave of the soil can be expected and this can cause problems similar to those of expansive clays if precautions are not taken. Conversely, adverse settlements occur during thaw and these too can create unwanted side effects. Sands and gravels are little affected because they are free draining, whereas movement can be large in silts and clays.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

Chapter 26

Building response to ground movements

John B. Burland Imperial College London, UK

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Ground movements affecting buildings can result from the loads applied by the building itself to the subsoil or from a variety of external causes including swelling or shrinkage of the subsoil, subsidence due to tunnelling or mining, and groundwater lowering. This chapter summarises an approach to assessing the response of buildings to ground movement and draws together in a coherent way a number of related studies including definitions of foundation movements, the description and classification of damage, and the factors controlling limiting distortions of buildings. The latter part of the chapter is devoted to applying the above principles to the important problem of assessing the impacts on buildings of subsidence induced by tunnelling. Methods of mitigating these impacts are described.

26.1 Introduction

Ground movements affecting buildings can result from the loads applied by the building itself to the subsoil or from external causes such as swelling or shrinkage of the subsoil, subsidence due to tunnelling or mining, and groundwater lowering. Whatever the cause of the ground movements they are likely to have an impact on the building and its function. In almost all cases, it is the differential movements (both vertical and horizontal) that are of importance – these can lead to tilting, distortion and possible damage to the building.

Compared with the literature on the prediction of ground movements, the question of allowable ground movements and their influence on the performance and serviceability of structures has received relatively little attention. Burland and Wroth (1974) listed some possible reasons for this:

(i) Serviceability is very subjective and depends both on the function of the building and the reactions of the users.

(ii) Buildings vary so much one from another, both in broad concept and in detail, that it is very difficult to lay down general guidelines as to allowable movements.

(iii) Buildings, including foundations, seldom perform as designed because construction materials display different properties from those assumed in design. Moreover, a 'total' analysis including the ground and the cladding would be impossibly complex and would still contain a number of questionable assumptions.

(iv) As well as depending on loading and settlement, movements in buildings can be attributed to a number of factors such as creep, shrinkage and temperature. As yet, there is little quantitative understanding of these factors and there is a lack of careful measurements of the performance of actual buildings.

There is tendency amongst foundation engineers, indeed structural engineers as well, to believe that the movements of the foundations are the major cause of distress in buildings, and that by controlling these the satisfactory performance of the building is guaranteed. At a conference on *Design for movement in buildings* hosted by The Concrete Society in 1969, many cases were quoted of damage to finishes which resulted from movements of the structural members rather than the foundations. Another aspect of the problem which engineers may overlook is that a certain amount of cracking is unavoidable if the building is to be economic (Peck, Deere and Capacete, 1956). It is said that it is impossible to build a structure that does not crack due to shrinkage, creep, etc. Little (1969) has estimated that in the case of one particular type of building, the cost of preventing any cracking could exceed 10% of the total building cost.

This chapter outlines the fundamental principles associated with assessing the response of buildings to ground movement. Two key aspects are: (i) the importance of clear definitions of ground and foundation movement; and (ii) the objective description and classification of building damage. These are required for an objective approach to the assessment of impacts of ground movement on buildings and are discussed in the following two sections.

26.2 Definitions of ground and foundation movement

A study of the literature reveals a wide variety of confusing symbols and terminology describing foundation movements.

Burland and Wroth (1974) proposed a consistent set of definitions based on the displacements (either measured or calculated) of a number of discrete points on the foundations of a building. Care was taken to ensure that the terms did not prejudice any conclusions about the distortions of the superstructure itself, since these depend on a large number of additional factors. The definitions appear to have been widely accepted and are illustrated in **Figure 26.1**. A few points to note are as follows:

(i) *Rotation* (or slope θ) is the change in gradient of a line joining two reference points (e.g. A, B in **Figure 26.1(a)**).

(ii) The *angular strain* α is shown in **Figure 26.1(a)**. It is positive for upward concavity (sagging) and negative for downward concavity (hogging).

(iii) *Relative deflection* Δ is the displacement of a point relative to the line connecting two reference points on either side (see **Figure 26.1(b)**). The sign convention is as for (ii).

(iv) *Deflection ratio* (sagging ratio or hogging ratio) is denoted by Δ/L where L is the distance between the two reference points defining Δ. The sign convention is as for (ii).

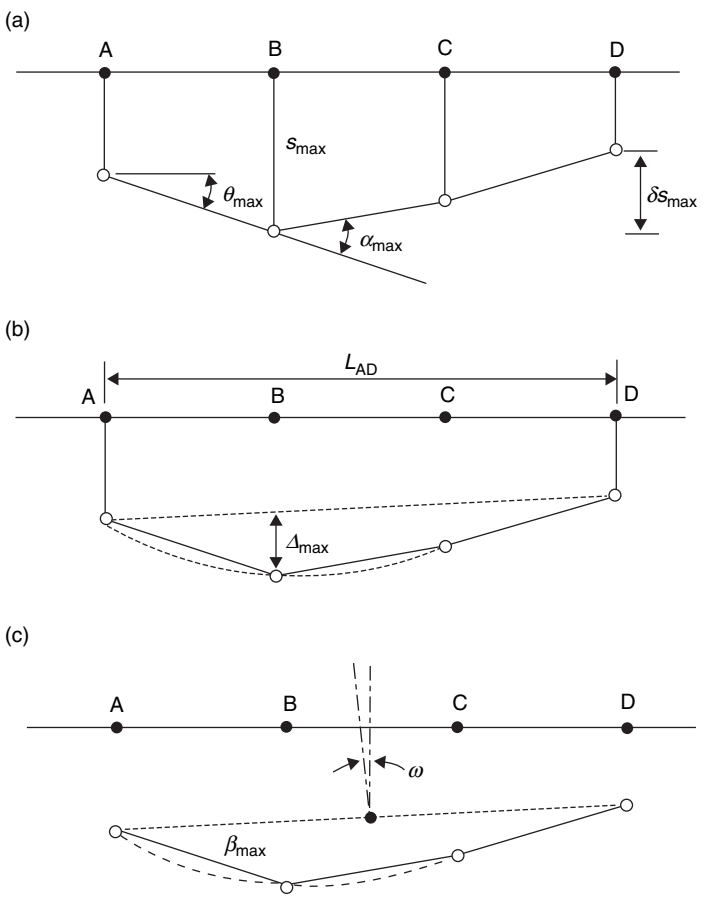


Figure 26.1 Definitions of ground and foundation movement: (a) rotation or slope θ and angular strain α; (b) relative deflection Δ and defection ratio Δ/L; (c) tilt ω and relative rotation β

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(v) *Tilt* ω describes the rigid body rotation of the structure or a well defined part of it – see **Figure 26.1(c)**.

(vi) *Relative rotation* (angular distortion) β is the rotation of the line joining two points, relative to the tilt ω – see **Figure 26.1(c)**. It is not always straightforward to identify the tilt and the evaluation of β can sometimes be difficult. It is also very important not to confuse relative rotation β with angular strain α. For these reasons, Burland and Wroth (1974) preferred the use of Δ/L as a measure of building distortion.

(vii) *Average horizontal strain* ε_h is defined as the change of length δL over the length L. In soil mechanics it is customary to take a reduction of length (compression) as positive.

The above definitions only apply to in-plane deformations and no attempt has been made to define three-dimensional behaviour.

26.3 Classification of damage

26.3.1 Introduction

Assessment of the degree of building damage can be a highly subjective, and often emotive, matter. It may be conditioned by a number of factors such as local experience, the attitude of insurers, the cautious approach of a professional engineer or surveyor who might be concerned about litigation, market value, and 'saleability' of the property. In the absence of objective guidelines based on experience, extreme attitudes and unrealistic expectations towards building performance can develop. It is worth stressing that most buildings experience a certain amount of cracking, often unrelated to foundation movement, which can be dealt with during routine maintenance and decoration.

Clearly, if an assessment of the risk of damage due to ground movement is to be made, the *classification* of damage is a key issue. In the UK, the development of an objective system of classifying damage is proving to be very beneficial in creating realistic attitudes towards building damage, and also in providing logical and objective criteria for designing for movement in buildings and other structures. This classification system will now be described.

26.3.2 Categories of damage

Three broad categories of building damage can be considered. These affect: (i) visual appearance or 'aesthetics'; (ii) serviceability or function; and (iii) stability. As foundation movements increase, damage to a building will progress successively from (i) through to (iii).

It is only a short step from the above three broad categories of damage to the six (more detailed) classifications given in **Table 26.1**. Normally, categories 0, 1 and 2 relate to aesthetic damage, 3 and 4 relate to serviceability damage, and 5 represents damage affecting stability. It was first put forward by Burland *et al.* (1977) who drew on the work of Jennings and Kerrich (1962), the UK National Coal Board (1975) and MacLeod and Littlejohn (1974). Since then the classification

has been adopted with only slight modifications by BRE (1981) and the Institution of Structural Engineers, London (1978 and 1994).

The system of damage classification in **Table 26.1** is based on 'ease of repair' of the visible damage. Thus, in order to classify visible damage it is necessary, when carrying out the survey, to assess what type of work would be required to repair the damage both externally and internally. The following important points should be noted:

(i) The classification relates only to the visible damage at a given time and not to its cause or possible progression which are separate issues.

(ii) The strong temptation to classify the damage solely on crack width must be resisted. It is the ease of repair which is the key factor in determining the category of damage.

(iii) The classification was developed for brickwork, blockwork and stone masonry. It can be adapted for other forms of cladding. It is not intended to apply to reinforced concrete structural elements.

(iv) More stringent criteria may be necessary where damage may lead to corrosion, penetration or leakage of harmful liquids and gases or structural failure.

Besides defining numerical categories of damage, **Table 26.1** also describes the 'normal degree of severity' associated with each category. These descriptions of severity relate to standard domestic and office buildings and serve as a guide to building owners and occupiers. In special circumstances, such as for a building with valuable or sensitive finishes, these descriptions of severity of damage may not be appropriate.

26.3.3 The division between categories 2 and 3 damage

The dividing line between categories 2 and 3 damage is particularly important. Studies of many case records show that damage up to category 2 can result from a variety of causes, either from within the structure itself (e.g. shrinkage or thermal effects) or be associated with the ground. Identification of the cause of damage is usually very difficult as it frequently results from a combination of causes. If the damage exceeds category 2, the cause is usually much easier to identify and it is frequently associated with ground movement. Thus the division between categories 2 and 3 damage represents an important threshold which will be referred to later.

26.4 Routine guides on limiting deformations of buildings

Perhaps the two best known studies leading to recommendations on allowable deformations of buildings are those of Skempton and MacDonald (1956) and Polshin and Tokar (1957). Skempton and MacDonald summarised settlement and damage observations on 98 buildings, 40 of which showed signs of damage. The study was limited to traditional steel and reinforced concrete frame buildings and to a few load-bearing wall buildings.

Category of damage	Normal degree of severity	Description of typical damage Ease of repair is underlined *Note*: Crack width is only one factor in assessing category of damage and should not be used on its own as a direct measure of it
0	Negligible	Hairline cracks less than about 0.1 mm
1	Very slight	Fine cracks which are easily treated during normal decoration. Damage generally restricted to internal wall finishes. Close inspection may reveal some cracks in external brickwork or masonry. Typical crack widths up to approximately 1 mm
2	Slight	Cracks easily filled. Re-decoration probably required. Recurrent cracks can be masked by suitable linings. Cracks may be visible externally and some repointing may be required to ensure weathertightness. Doors and windows may stick slightly. Typical crack widths 2–3 mm but may be up to approximately 5 mm locally
3	Moderate	The cracks require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired. Typical crack widths are approximately 5–15 mm or several closely spaced cracks > 3 mm
4	Severe	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably[1]. Walls leaning[1] or bulging noticeably, some loss of bearing in beams. Service pipes disrupted. Typical crack widths are 15–25 mm, depending on the number of cracks
5	Very severe	This requires a major repair job involving partial or complete rebuilding. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability. Typical crack widths are > 25 mm, depending on the number of cracks

[1] *Note*: Local deviation of slope, from the horizontal or vertical, of more than 1/100 will normally be clearly visible. Overall deviations in excess of 1/150 are undesirable.

Table 26.1 Classification of visible damage to walls with particular reference to ease of repair of plaster and brickwork or masonry

As their measure of building distortion, they used the ratio of the differential settlement δ and the distance l between two points after eliminating the influence of tilt of the building. They defined the ratio δ/l as the 'angular distortion' which corresponds to the 'relative rotation' β defined in section 26.2. It was concluded that the limiting value of β to cause cracking in walls and partitions is 1/300 and that values of β greater than 1/150 will cause structural damage. It was recommended that values of β greater than 1/500 should be avoided. Subsequently, Bjerrum (1963) supplemented these recommendations by relating the magnitude of β to various types of impacts and structures. It is important to appreciate that Skempton and MacDonald's conclusions relate to traditional frame buildings and that the evidence relating to load-bearing walls was not reliable. Also, no classification of damage was used other than 'architectural', 'functional' and 'structural'. The limitations of the data and the tentative nature of the conclusions were emphasised by Skempton and MacDonald in their paper, but these qualifications are seldom emphasised in textbooks and design recommendations.

Polshin and Tokar (1957) discussed the question of allowable deformations and settlements, and defined three criteria: relative rotation β; deflection ratio Δ/L; and average settlement. The limiting values of these, adopted by the 1955 Building Code of the USSR, were then listed. It is of particular interest to note that frame structures were treated separately from continuous load-bearing walls. Recommended maximum relative rotations vary from 1/500 for steel and concrete frame infilled structures to 1/200, where there is no infill and no danger of damage to cladding.

Much stricter criteria were laid down for load-bearing brick walls. For L/H ratios (where L is the length and H the height) of less than 3, the maximum values of Δ/L are 0.3×10^{-3} for sand and 0.4×10^{-3} for soft clay. For L/H ratios greater than 5, the corresponding maximum values of Δ/L are 0.5×10^{-3} and 0.7×10^{-3} respectively. In their paper, Polshin and Tokar (1957) introduced two important variables: (i) the L/H ratio of the building or wall; and (ii) the concept of limiting tensile strain before cracking. Using a limiting tensile strain of 0.05%, the limiting relationship between L/H and Δ/L was presented and was shown to be in good agreement with a number of cracked and uncracked brick buildings. The above recommendations for load-bearing masonry walls are based on a requirement for no cracking; if adhered to, the degree of damage would be unlikely to exceed category 1 (very slight) in **Table 26.1**.

The reader is referred to Chapter 52 *Foundation types and conceptual design principles* for a more detailed discussion on routine guides to allowable building movements.

26.5 Concept of limiting tensile strain

26.5.1 Onset of visible cracking

Cracking in masonry walls and finishes usually, but not always, result from tensile strain. Following the work of Polshin and Tokar (1957), Burland and Wroth (1974) investigated the idea that tensile strain might be a fundamental parameter in determining the onset of cracking. A study of the results from numerous large scale tests on masonry panels and walls carried out at the UK Building Research Establishment (BRE) showed that, for a given material, the onset of visible cracking is associated with a reasonably well defined value of average tensile strain which is not sensitive to the mode of deformation. They defined this as the critical tensile strain ε_{crit} which is measured over a gauge length of a metre or more.

Burland and Wroth (1974) made the following important observations:

(i) The average values of ε_{crit} at which visible cracking occurs are very similar for a variety of types of brickwork and blockwork and are in the range 0.05–0.1%.

(ii) For reinforced concrete beams the onset of visible cracking occurs at lower values of tensile strain in the range 0.03–0.05%.

(iii) The above values of ε_{crit} are much larger than the local tensile strains corresponding to tensile failure.

(iv) The onset of visible cracking does not necessarily represent a limit of serviceability. Provided the cracking is controlled, it may be acceptable to allow deformations well beyond the initiation of visible cracking.

Burland and Wroth (1974) showed how the concept of critical tensile strain could be used in conjunction with simple elastic beams to develop deflection criteria for the onset of visible damage. This work will be discussed in more detail later.

26.5.2 Limiting tensile strain – a serviceability parameter

Burland *et al.* (1977) replaced the concept of *critical tensile strain* with that of *limiting tensile strain* (ε_{lim}). The importance of this development is that ε_{lim} can be used as a serviceability parameter which can be varied to take account of differing materials and serviceability limit states.

Boscardin and Cording (1989) developed this concept of differing levels of tensile strain. Seventeen case records of damage due to excavation-induced subsidence were analysed. A variety of building types were involved and they showed that the categories of damage given in **Table 26.1** could be broadly related to ranges of ε_{lim}. These ranges are tabulated in **Table 26.2**, which is important as it provides the link between estimated building deformations and the possible severity of damage.

26.6 Strains in simple rectangular beams

This section describes how Burland and Wroth (1974) and Burland *et al.* (1977) used the concept of limiting tensile strain to study the onset of cracking in simple weightless elastic beams undergoing sagging and hogging modes of deformation. This simple approach gives considerable insight into the mechanisms controlling cracking. Moreover, it is shown that the criteria for initial cracking of simple beams are in very good

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Category of damage	Normal degree of severity	Limiting tensile strain (ε_{lim}) (%)
0	Negligible	0–0.05
1	Very slight	0.05–0.075
2	Slight	0.075–0.15
3	Moderate1	0.15–0.3
4–5	Severe to very severe	> 0.3

[1] *Note*: Boscardin and Cording (1989) describe the damage corresponding to ε_{lim} in the range 0.15–0.3% as 'moderate to severe'. However, none of the cases quoted by them exhibit severe damage for this range of strains. There is therefore no evidence to suggest that tensile strains up to 0.3% will result in severe damage.

Table 26.2 Relationship between category of damage and limiting tensile strain (ε_{lim})
Data taken from Boscardin and Cording (1989)

agreement with the case records of damaged and undamaged buildings undergoing settlement. Therefore, in many circumstances, it is both reasonable and instructive to represent the facade of a building by means of a simple rectangular beam.

26.6.1 The relationship between limiting values of Δ/L and limiting tensile strain

The approach adopted by Burland and Wroth (1974) is illustrated in **Figure 26.2**, where the building is represented by a rectangular beam of length L and height H. The problem is to calculate the tensile strains in the beam for a given deflected shape of the building foundations and hence obtain the sagging or hogging ratio Δ/L at which cracking is initiated. It is immediately obvious that little can be said about the distribution of strains within the beam unless we know its *mode* of deformation. Two extreme modes are: (i) bending only about a neutral axis at the centre (**Figure 26.2(d)**); and (ii) shearing only (**Figure 26.2(e)**). In the case of bending only, the maximum tensile strain occurs in the top fibre and that is where cracking will initiate as shown. In the case of shear only, the maximum tensile strains are inclined at 45°, giving rise to diagonal cracking. In general, both modes of deformation will occur simultaneously and it is necessary to calculate both bending and diagonal tensile strains to ascertain which type is limiting.

The expression for the total mid-span deflection Δ of a centrally loaded beam having both bending and shear stiffness, is given by Timoshenko (1957) as:

$$\Delta = \frac{PL^3}{48EI}\left(1+\frac{18EI}{L^2HG}\right) \tag{26.1}$$

where E is Young's modulus, G is the shear modulus, I is the second moment of area and P is the point load. Equation (26.1) can be re-written in terms of the deflection ratio Δ/L and the maximum extreme fibre strain $\varepsilon_{b\,max}$ as follows:

$$\frac{\Delta}{L} = \left(\frac{L}{12t}+\frac{3I}{2tLH}\frac{E}{G}\right)\varepsilon_{b\,max} \tag{26.2}$$

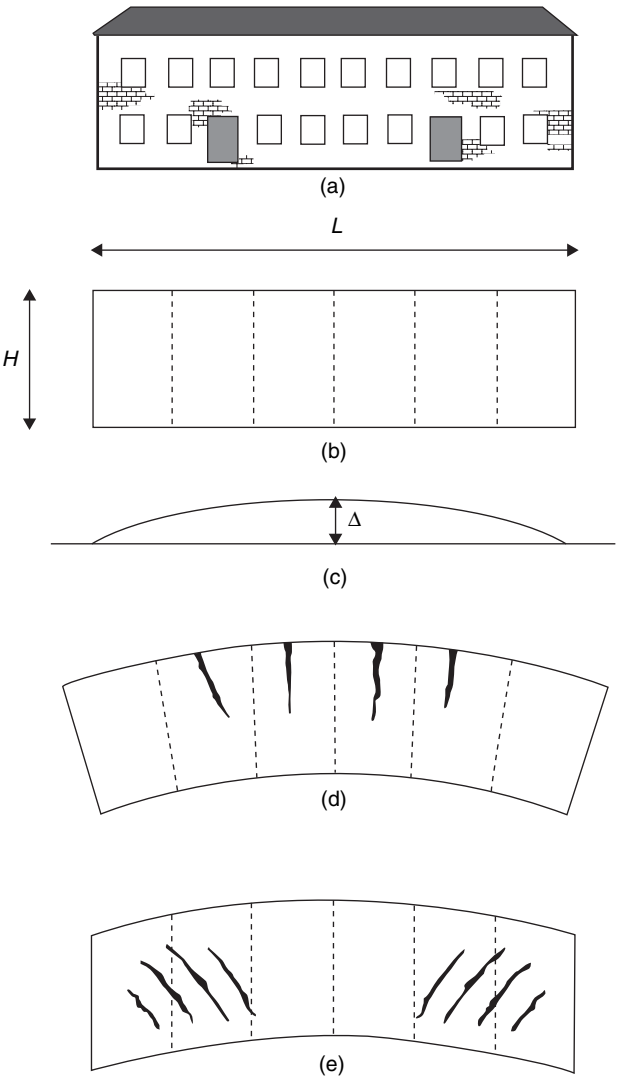


Figure 26.2 Cracking of a simple beam in bending and in shear: (a) actual building; (b) equivalent deep beam; (c) deflected shape of foundation; (d) bending deformation; (e) shear deformation
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where t is the distance of the neutral axis from the edge of the beam in tension. Similarly for the maximum diagonal strain $\varepsilon_{d\,max}$ equation (26.1) becomes:

$$\frac{\Delta}{L} = \left(1+\frac{HL^2}{18I}\frac{G}{E}\right)\varepsilon_{d\,max}. \tag{26.3}$$

Similar expressions are obtained for the case of a uniformly distributed load with the diagonal strains calculated at the quarter points of the span (i.e. at $L/4$ and $3L/4$). Burland and Wroth (1974) drew the important conclusion that the maximum tensile strains are much more sensitive to the value of Δ/L than to the precise distribution of loading.

By setting $\varepsilon_{max} = \varepsilon_{lim}$, equations (26.2) and (26.3) define the limiting values of Δ/L for the deflection of simple beams.

It is evident that, for a given value of ε_{lim}, the limiting value of Δ/L (whichever is the lowest in equations (26.2) and (26.3)) depends on L/H, E/G and the position of the neutral axis. For an isotropic beam ($E/G \approx 2.5$) with a neutral axis in the middle, the limiting relationship between $\Delta/L.\varepsilon_{lim}$. and L/H is given by curve (1) in **Figure 26.3**. For a beam which has relatively low stiffness in shear ($E/G = 12.5$), the limiting relationship is given by curve (2). A particularly important case is that of a beam which is relatively flexible in bending, and is subjected to hogging such that its neutral axis is at the bottom. Curve (3) shows the limiting relationship for such a beam having $E/G = 0.5$. These curves serve to illustrate that even for simple beams, the limiting values of Δ/L causing cracking can vary over wide limits.

26.6.2 Limiting values of Δ/L for very slight damage

Burland and Wroth (1974) carried out a preliminary survey of data for cracking of infill frames and masonry walls. They concluded that the range of values of average tensile strain at which the onset of visible cracking for a variety of common building materials was remarkably small. For brickwork and blockwork set in cement mortar ε_{lim} lies in the range 0.05–0.1%, while for reinforced concrete having a wide range of strengths, the values lie in the range of about 0.03–0.05%.

In order to assess the potential value of the limiting tensile strain approach in estimating the onset of visible cracking in buildings, Burland and Wroth compared the limiting criteria obtained from the analysis of simple beams with observations of the behaviour of a number of buildings – many of them of modern construction. For this comparison, a value of $\varepsilon_{lim} = 0.075\%$ was used. As can be seen from **Table 26.2**, this value corresponds to the threshold between category 1 (very slight) and category 2 (slight) damage. The buildings were classified as frame, load-bearing walls undergoing sagging, and load-bearing walls undergoing hogging. **Figures 26.4(a)**, **(b)** and **(c)** show the comparison with curves (2), (1) and (3) respectively from **Figure 26.3**, with $\varepsilon_{lim} = 0.075\%$. Also shown is the criterion of limiting relative rotation $\beta = 1/300$ proposed by Skempton and MacDonald (1956) and the limiting relationship proposed by Polshin and Tokar (1957) for load-bearing walls. In spite of its simplicity, the analysis reflects the major trends in the observations. In particular, the prediction is borne out that load-bearing walls, especially when subjected to hogging, are more susceptible to damage than frame buildings which are relatively flexible in shear.

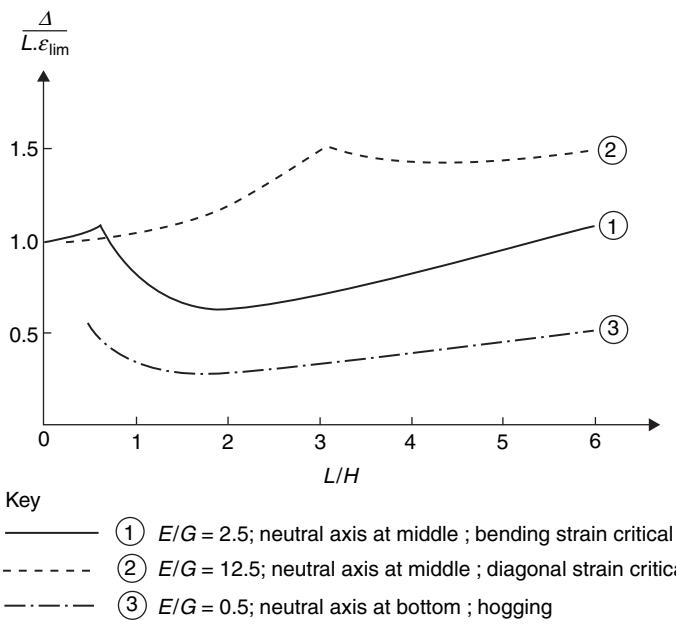


Figure 26.3 The influence of *E/G* and mode of deformation on the cracking of a simple rectangular beam
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26.6.3 The relationship between Δ/L and levels of damage

At this point it is necessary to emphasise that limiting tensile strain is not a fundamental material property like tensile strength. It should be regarded as a measure of serviceability (as set out in **Table 26.2**) which aids the engineer in deciding whether a building is likely to develop visible cracks, their severity, and where the critical locations might be. Equations (26.2) and (26.3) can be used in this way to assess what levels of damage might be expected for a given (perhaps calculated) value of Δ/L.

The advantages of this approach over traditional empirical rules limiting deformations are:

(i) It can be applied to complex structures employing well established stress analysis techniques.

(ii) It makes explicit the fact that damage can be controlled by paying attention to the modes of deformation within the building structure and its finishes.

(iii) The limiting values can be varied to take account of differing materials and serviceability limit states, e.g. the use of soft bricks and lime mortar can substantially reduce cracking as it raises the value of ε_{lim}.

It is also important to appreciate that the onset of visible cracking does not necessarily represent a limit of serviceability. Provided cracking is controlled, it may be acceptable to allow deformations to continue well beyond the initiation of cracking. Cases where the propagation of initial cracks may be fairly well controlled are framed structures with panel walls, and also reinforced load-bearing structures. Unreinforced load-bearing walls undergoing sagging under the restraining action of the foundations may also fall into this category. An important mode of deformation where uncontrolled cracking can occur is that of hogging of unreinforced free-standing load-bearing walls. Once a crack forms at the top of the wall there is nothing to stop it propagating downwards. Low-rise houses with shallow

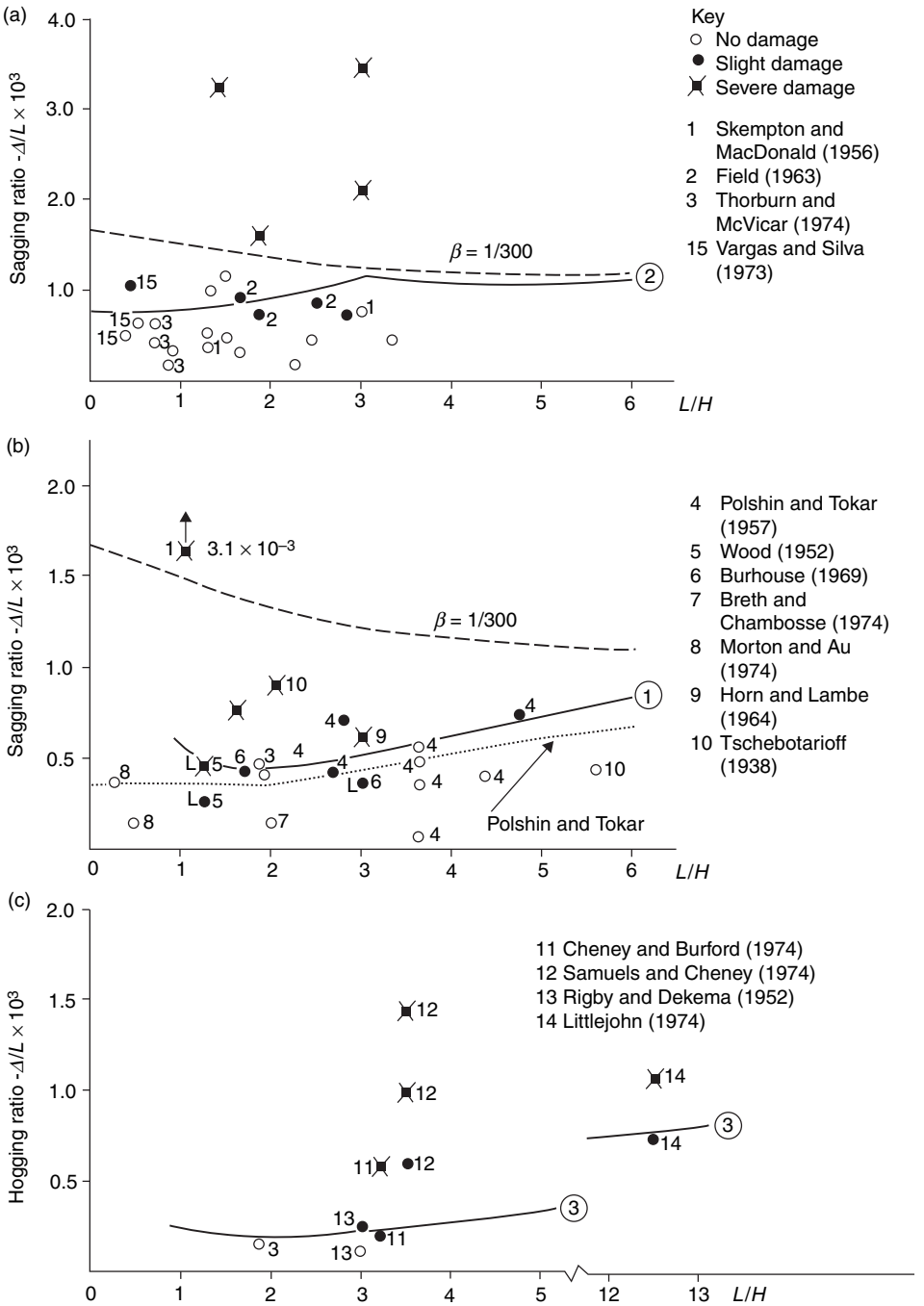


Figure 26.4 Relationship between Δ/L and L/H for buildings showing various degrees of damage: (a) frame buildings; (b) loadbearing walls; (c) hogging of loadbearing walls

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foundations on swelling or shrinking clays are particularly prone to this form of damage.

The remainder of this chapter is devoted to applying the principles set out above to the important case of assessing the impacts on buildings of subsidence induced by tunnelling and excavations.

26.7 Ground movement due to tunnelling and excavation

26.7.1 Introduction

In recent years, assessment of the environmental impact of major construction projects has become a normal and required procedure. The construction of tunnels in urban areas, while

having many long-term environmental benefits, can also create significant environmental impacts. During construction, such impacts would include construction traffic, noise, vibration and dust as well as temporary restrictions on access to certain roads and other public areas. Longer term impacts would include land and building acquisition, traffic and ventilation noise and vibration levels, and other impacts such as pollution, ground water changes and effects on ecology.

An environmental impact which is causing increasing public awareness and concern is that of subsidence and its effects on structures and services due to tunnelling and deep excavations. Construction of tunnels and deep excavations is inevitably accompanied by ground movements. It is necessary, both for engineering design and for planning and consultation, to develop rational procedures for assessing the risks of damage. Coupled with such assessments is, of course, the requirement for effective protective measures which can be deployed when predicted levels of damage are judged to be unacceptable.

The following section summarises the approach that was originally developed in the 1990s for assessing the risks of subsidence damage for the London Underground Jubilee Line Extension project, which involved tunnelling under densely developed areas of central London. The experience gained from this project has been summarised in a two volume book (Burland *et al*., 2001a and 2001b). The approach to assessing the risks of subsidence damage, which is presented here, draws on the results of the studies described previously, including the description and classification of damage and limiting distortions of brickwork and masonry walls.

The construction of tunnels or surface excavations will inevitably be accompanied by movement of the ground around them. At the ground surface, these movements manifest in what is called a 'settlement trough'. **Figure 26.5** shows diagrammatically the surface settlement trough above an advancing tunnel. For greenfield sites, the shape of this trough transverse to the axis of the tunnel approximates closely to a normal Gaussian distribution curve – an idealisation which has considerable mathematical advantages.

26.7.2 Settlements caused by tunnel excavation

Figure 26.6 shows such an idealised transverse settlement trough. Attewell *et al*. (1986) and Rankin (1988) have summarised the current widely used empirical approach to the prediction of immediate surface and near-surface ground displacements. The settlement s is given by:

$$s = s_{max} \exp\left(\frac{-y^2}{2i^2}\right) \tag{26.4}$$

where s_{max} is the maximum settlement and i is the value of y at the point of inflection. It has been found that, for most

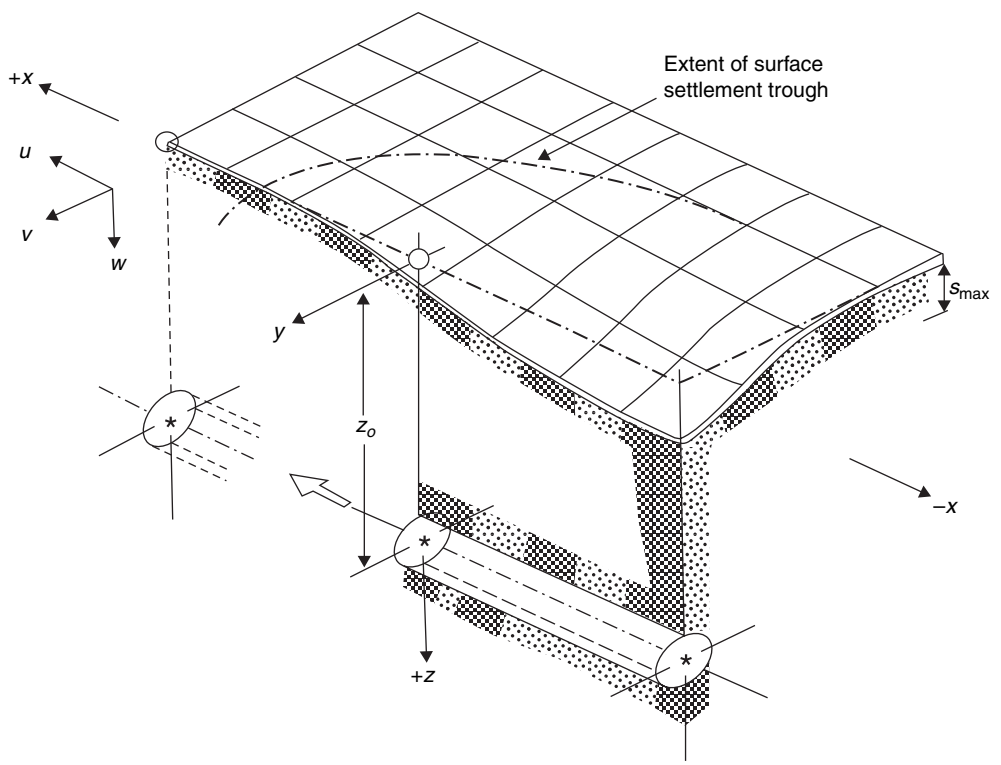


Figure 26.5 Settlement trough above an advancing tunnel
Reproduced from Attewell *et al*. (1986); Blackie and Sons Ltd. All rights reserved

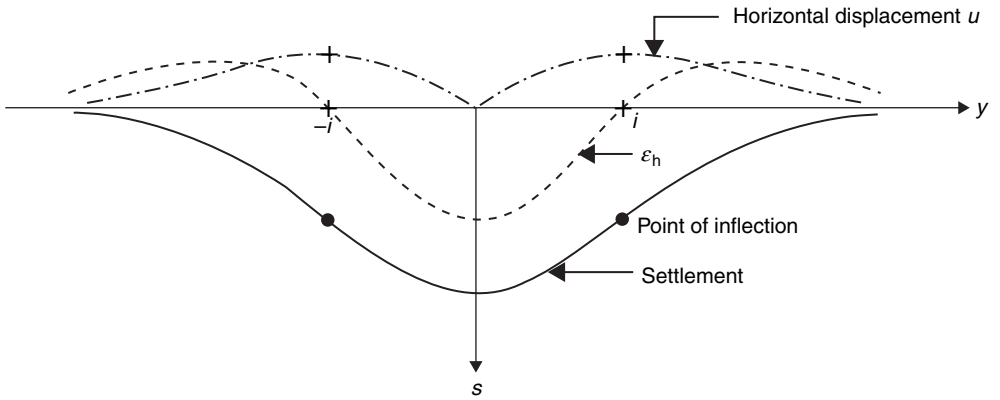


Figure 26.6 Transverse settlement trough and horizontal displacement profiles

purposes, i can be related to the depth of the tunnel axis z_o by the linear expression:

$$i = K z_o. \tag{26.5}$$

The trough width parameter K depends on the soil type. It varies from 0.2 to 0.3 for granular soils, 0.4 to 0.5 for stiff clays, to as high as 0.7 for soft silty clays. As a general rule, the width of the surface settlement trough is about three times the depth of the tunnel for tunnels in clay strata. It is important to note that, although the value of K for *surface* settlements is approximately constant for various depths of tunnel in the same ground, Mair *et al.* (1993) have shown that its value increases with depth for *subsurface* settlements.

The immediate settlements caused by tunnelling are usually characterised by the 'volume loss' V_L which is the volume of the surface settlement trough per unit length V_s expressed as a percentage of the notional excavated volume of the tunnel.

Integration of equation (26.4) gives:

$$V_s = \sqrt{2\pi}\, i\, s_{\max} \tag{26.6}$$

so that:

$$V_L = \frac{3.192\, i\, s_{\max}}{D^2} \tag{26.7}$$

where D is the diameter of the tunnel. Combining equations (26.4), (26.5) and (26.7) gives the surface settlement s at any distance y from the centreline:

$$s = \left(\frac{0.313 V_L D^2}{K z_o}\right) \exp\left(\frac{-y^2}{2K^2 z_o^2}\right). \tag{26.8}$$

26.7.3 Horizontal displacements due to tunnelling

Building damage can also result from horizontal tensile strain, and therefore predictions of horizontal movement are required. Unlike settlements, there are few case histories where horizontal movements have been measured. The data that do exist show reasonable agreement with the assumption of O'Reilly and New (1982) that the resultant vectors of ground movement are directed towards the tunnel axis. It follows that the horizontal displacement u can be related to the settlement s by the expression:

$$u = \frac{s.y}{z_o}. \tag{26.9}$$

Equation (26.9) is easily differentiated to give the horizontal strain ε_h at any location on the ground surface.

Figure 26.6 shows the relation between the settlement trough, the horizontal displacements and the horizontal strains occurring at ground level. In the region $-i < y < i$, the horizontal strains are compressive. At the points of inflection the horizontal displacements are a maximum and $\varepsilon_h = 0$. For $y > |i|$ the horizontal strains are tensile.

26.7.4 Assessment of surface displacements due to tunnelling

The above empirical equations provide a simple means for estimating the near-surface displacements due to tunnelling, assuming greenfield conditions, i.e. ignoring the presence of any building or structure.

A key parameter in this assessment is the volume loss V_L. This results from a variety of effects which include movement of ground into the face of the tunnel and radial movement towards the tunnel axis due to reductions in supporting pressures. The magnitude of V_L is critically dependent on the type of ground, the ground water conditions, the tunnelling method, the length of time in providing positive support, and the quality of supervision and control. The selection of an appropriate value of V_L for design requires experience and is greatly aided by well documented case histories in similar conditions.

A number of other assumptions are involved in the prediction of ground displacements due to tunnelling. For example,

in ground containing layers of clay and granular soils, there is uncertainty about the value of the trough width parameter K. When two or more tunnels are to be constructed in close proximity, the assumption is usually made that the estimated ground movements for each tunnel acting independently can be superimposed. In some circumstances this assumption may underestimate the movements and allowance needs to be made for this.

It is clear from the above that, even for greenfield conditions, precise prediction of ground movements due to tunnelling is not realistic. However, it is possible to make reasonable estimates of the likely range of movements provided tunnelling is carried out under the control of suitably qualified and experienced engineers.

26.7.5 Ground movements due to deep excavations

Ground movements around deep excavations are critically dependent both on the ground conditions (e.g. stratigraphy, groundwater conditions, deformation and strength properties) and the method of construction (e.g. sequence of excavation, sequence of propping, rigidity of retaining wall and supports). In general, open excavations and those supported by cantilever retaining walls give rise to larger ground movements than strutted excavations and those constructed by top-down methods. In the urban situation, the latter are clearly to be preferred if building damage is to be minimised.

The calculation of ground movements is not straightforward and much experience is required to make any sensible use of complex analyses. It is therefore essential that optimum use is made of previous experience and case histories in similar conditions. Peck (1969) presented a comprehensive survey of vertical movements around deep excavations which was updated by Clough and O'Rourke (1990). Burland *et al.* (1979) summarised the results of over ten years of research into the movements of ground around deep excavations in London Clay. The Norwegian Geotechnical Institute have published a number of case histories of excavations in soft clay in the Oslo area (e.g. Karlsrud and Myrvoll (1976)). For well-supported excavations in stiff clays, Peck's settlement envelopes are generally very conservative as settlements rarely exceed 0.15% of the depth of excavation. However, movements can extend to 3 or 4 times the excavation depth behind the basement wall. Horizontal movements are generally of similar magnitude and distribution to vertical movements, but may be much larger for open and cantilever excavations in stiff clays.

Advanced methods of numerical analysis, based on the finite element method, are widely used for prediction of ground movements around deep excavations. Such analyses can simulate the construction process, modelling the various stages of excavation and support conditions. However, comparison with field observations shows that successful prediction requires high quality soil samples with the measurement of small-strain stiffness properties using local strain transducers mounted on the sides of the samples (Jardine *et al.*, 1984).

As for tunnelling, it is essential that deep excavation work be carried out under the close supervision of an experienced engineer. Unless positive support is provided rapidly and groundwater is properly controlled, large unexpected ground movements can develop.

26.7.6 The influence of horizontal strain

It was shown in section 26.7.3 that ground surface movements associated with tunnelling and excavation not only involve sagging and hogging profiles, but significant horizontal strains as well – see **Figure 26.6**. Boscardin and Cording (1989) included the influence of horizontal extension strain ε_h in equations (26.2) and (26.3) by using simple superposition; i.e. it is assumed that the deflected beam is subjected to uniform extension over its full depth. The resultant extreme fibre strain ε_{br} is given by:

$$\varepsilon_{br} = \varepsilon_{b\,max} + \varepsilon_h \tag{26.10}$$

where $\varepsilon_{b\,max}$ is obtained from equation (26.2). In the shearing region of the beam, the resultant diagonal tensile strain ε_{dr} can be evaluated using the Mohr's circle of strain. The value of ε_{dr} is then given by:

$$\varepsilon_{dr} = \varepsilon_h\left(\frac{1-\nu}{2}\right) + \sqrt{\varepsilon_h^2\left(\frac{1+\nu}{2}\right)^2 + \varepsilon_{d\,max}^2} \tag{26.11}$$

where $\varepsilon_{d\,max}$ is obtained from equation (26.3) and ν is Poisson's ratio. The maximum tensile strain is the greater of ε_{br} and ε_{dr}. Thus, for a beam of length L and height H, it is a straightforward matter to calculate the maximum value of tensile strain ε_{max} for a given value of Δ/L, and ε_h, in terms of t, E/G and ν. This value of ε_{max} can then be used in conjunction with **Table 26.2** to assess the potential associated damage.

The physical implications of equations (26.2), (26.3), (26.10) and (26.11) can be illustrated by considering the previous case of the isotropic beam undergoing hogging, with its neutral axis at the bottom edge and $\nu = 0.3$. By combining equations (26.2) and (26.10), the influence of ε_h on the limiting values of Δ/L can be examined for bending strains only by setting $\varepsilon_{bmax} = \varepsilon_{lim}$. **Figure 26.7(a)** shows the normalised relationship between Δ/L and ε_h. For $\varepsilon_h = 0$, the limiting values of Δ/L are the same as given in **Figure 26.3** for various values of L/H. It can be seen that as ε_h increases towards the value of ε_{lim}, the limiting values of Δ/L for a given L/H reduce linearly, becoming zero when $\varepsilon_h = \varepsilon_{lim}$.

Similarly, **Figure 26.7(b)** has been derived from equations (26.3) and (26.11) for the diagonal strains only. Once again, for $\varepsilon_h = 0$ the limiting values of Δ/L are recovered from **Figure 26.3**. As ε_h increases, the limiting values of Δ/L decrease non-linearly at an increasing rate towards zero. It is of interest to note that the values of Δ/L are not very sensitive to the values of L/H between 0 and 1.5.

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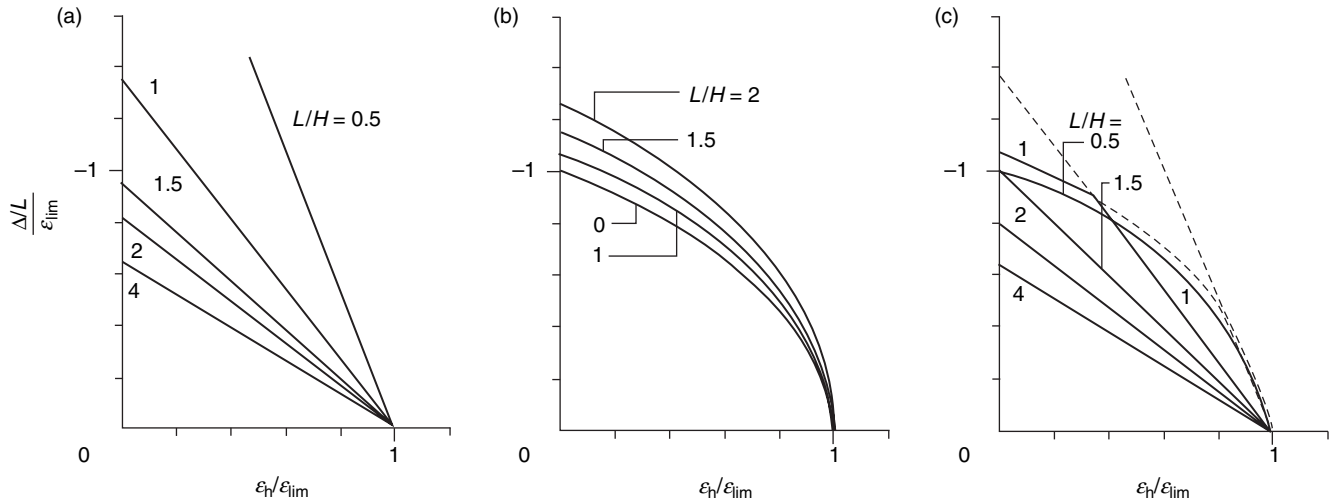


Figure 26.7 Influence of horizontal strain on $(\Delta/L)/\varepsilon_{lim}$ for (a) bending strain controlling; (b) diagonal strain controlling; and (c) limiting combinations of (a) and (b). Note that $E/G = 2.5$ and the mode of deformation is hogging

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Figures 26.7(a) and **(b)** can be combined to give the resultant normalised limiting relationships between Δ/L and ε_h for various values of L/H, as shown in **Figure 26.7(c)**. It can be seen that, for $L/H > 1.5$, the bending strains always control. Also, for lower values of L/H, as ε_h increases, the controlling strain changes from diagonal to bending. It must be emphasised that **Figure 26.7** relates to the specific case of hogging with the neutral axis at the lower face and with $E/G = 2.5$.

By adopting the values of ε_{lim} associated with the various categories of damage given in **Table 26.2**, **Figure 26.7(c)** can be developed into an interaction diagram showing the relationship between Δ/L and ε_h for a particular value of L/H. **Figure 26.8** shows such a diagram for $L/H = 1$.

There are similarities between **Figure 26.8** and the well-known Boscardin and Cording (1989) chart of angular distortion β versus ε_h. The latter chart has the following limitations:

(i) it only relates to $L/H = 1$;

(ii) maximum bending strains ε_{bmax} are ignored;

(iii) β was assumed to be proportional to Δ/L whereas Burland *et al.* (2004) have shown that the relationship is in fact very sensitive to the load distribution and to the value of E/G;

(iv) as mentioned in section 26.2, the evaluation of β is not always straightforward.

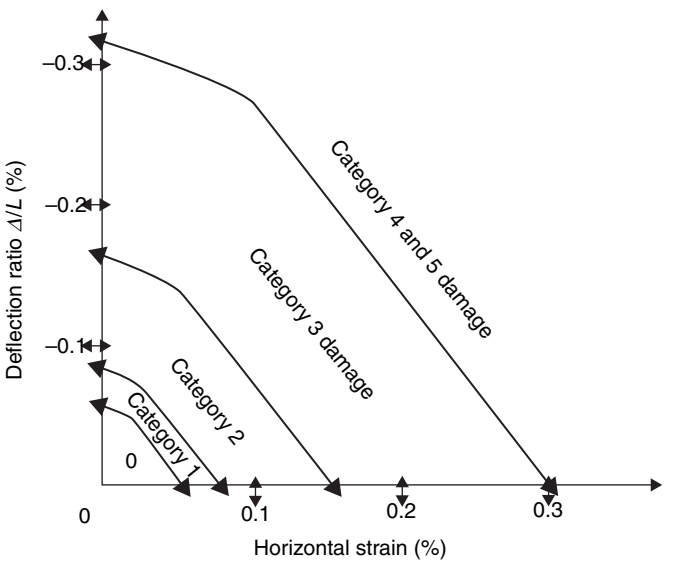


Figure 26.8 Relationship of damage category to deflection ratio and horizontal strain for hogging ($L/H = 1$, $E/G = 2.5$)

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26.7.7 Relevant building dimensions

An important consideration is the definition of the relevant height and length of the building. A typical case of a building affected by a single tunnel settlement trough is shown in **Figure 26.9**. The height H is taken as the height from foundation level to the eaves. The roof is usually ignored. It is assumed that a building can be considered separately either side of a point of inflexion, i.e. points of inflexion of the settlement profile (at foundation level) will be used to partition the building. The length of building is not considered beyond the practical limit of the settlement trough, which for a single tunnel can be taken as $2.5i$ (where $s/s_{max} = 0.044$). In a calculation of building strain, the building span length is required; it is defined as the length of building in a hogging or sagging zone (shown as L_h or L_s in **Figure 26.9**) and limited by a point of inflexion or extent of settlement trough.

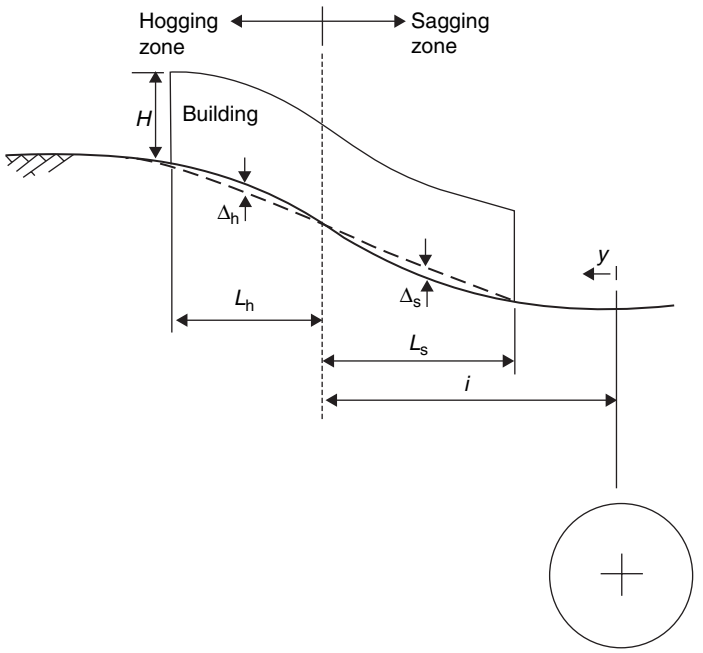


Figure 26.9 Building deformation – partitioning between sagging and hogging
Reproduced from Mair *et al.* (1996); all rights reserved

26.8 Evaluation of risk of damage to buildings due to subsidence

The various concepts discussed in the previous sections can be combined to develop a rational approach to the assessment of risk of damage to buildings due to tunnelling and excavation. The following broadly describes the approach that was adopted during the planning and enquiry stages of the Jubilee Line Extension underground railway in London, and which is now widely used internationally with minor variations (Burland, 1997).

26.8.1 Level of risk

The term 'the level of risk', or simply 'the risk', of damage refers to the *possible degree of damage* as defined in **Table 26.1**. Most buildings are considered to be at low risk if the predicted degree of damage falls into the first three categories 0 to 2 (i.e. negligible to slight). At these degrees of damage, structural integrity is not at risk and damage can be readily and economically repaired. It will be recalled from section 26.3.3 that the threshold between categories 2 and 3 damage is a particularly important one. A major objective of design and construction is to maintain the level of risk below this threshold for all buildings. It should be noted that special consideration has to be given to buildings judged to be of particular sensitivity, such as those in poor condition, containing sensitive equipment, or of particular historical or architectural significance.

Because of the large number of buildings that are usually involved, the method of assessing risk is a staged process as follows: preliminary assessment; second stage assessment; detailed evaluation. These three stages will now be described briefly.

26.8.2 Preliminary assessment

So as to avoid a large number of complex and unnecessary calculations, a very simple and conservative approach is adopted for the preliminary assessment. It is based on a consideration of both maximum slope and maximum settlement of the ground surface at the location of each building. According to Rankin (1988), a building experiencing a maximum slope θ of 1/500 and a settlement of less than 10 mm has negligible risk of any damage. By drawing contours of ground surface settlement along the route of the proposed tunnel and its associated excavations, it is possible to eliminate all buildings having negligible risk. This approach is conservative because it uses ground surface displacements rather than foundation level ones. It also neglects any interaction between the stiffness of the buildings and the ground. For particularly sensitive buildings it may be necessary to adopt more stringent slope and settlement criteria.

26.8.3 Second stage assessment

The preliminary assessment described above is based on the slope and settlement of the ground surface and provides a conservative initial basis for identifying those buildings along the route requiring further study. The second stage assessment makes use of the work described in the previous sections of this chapter. In this approach, the façade of a building is represented by a simple beam whose foundations are assumed to follow the displacements of the ground in accordance with the greenfield site assumption mentioned in section 26.7. The maximum resultant tensile strains are calculated from the pairs of equations (26.2), (26.10) and (26.3), (26.11). The corresponding potential category of damage, or level of risk, is then obtained from **Table 26.2**.

The above approach, though considerably more detailed than the preliminary assessment, is usually still very conservative. Thus the derived categories of damage refer only to *possible* degrees of damage. In the majority of cases, the likely *actual* damage will be less than the assessed category because in calculating the tensile strains, the building is assumed to have no stiffness – so that it conforms to the greenfield site subsidence trough. In practice, however, the inherent stiffness of the building will be such that its foundations will interact with the supporting ground and tend to reduce both the deflection ratio and the horizontal strains.

Potts and Addenbrooke (1996; 1997) carried out a parametric study of the influence of building stiffness on ground movements induced by tunnelling using finite element methods, incorporating a non-linear elastic-plastic soil model. The building was represented by an equivalent beam having axial and bending stiffness EA and EI (where E is the Young's modulus, A the cross-sectional area and I the moment of inertia of

the beam). The relative axial stiffness α^* and bending stiffness ρ^* are defined as:

$$\alpha^* = EA/E_sH \text{ and } \rho^* = EI/E_sH^4 \quad (26.12)$$

where H is the half-width of the beam (=B/2) and E_s is a representative soil stiffness.

Figure 26.10 shows the influence of relative bending stiffness on the settlement profile for a 20 m deep tunnel excavated beneath a 60 m wide building with zero eccentricity. **Figure 26.11** shows the modification factors MDR that are applied to the green field site deflection ratios (Δ/L) for sagging and hogging modes of deformation at different e/B ratios, where e is the eccentricity of the tunnel centreline. It can be seen that, for hogging in particular, the building changes from relatively very flexible to relatively very stiff over quite a small range in relative bending stiffness.

Potts and Addenbrooke's assessment of the influence of global stiffness of a building is a most valuable addition to the existing methodology for assessing the risk of damage. The results can be used to make more realistic assessments of relative deflection, and hence average strains within a building.

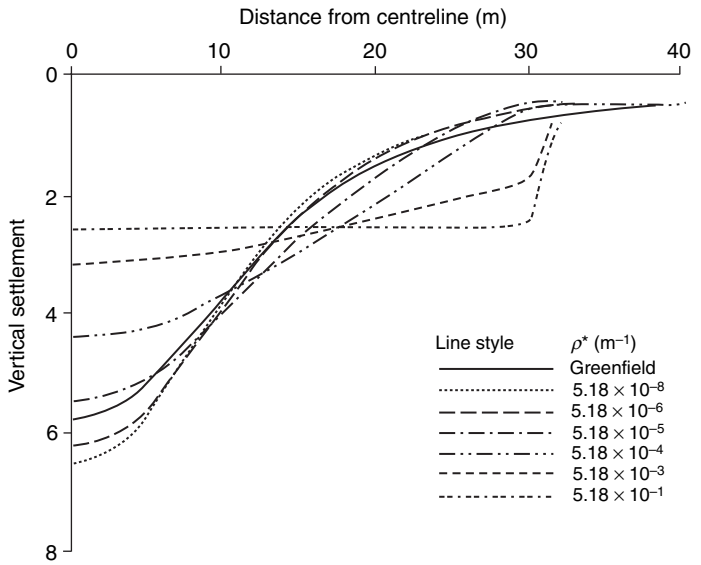


Figure 26.10 Influence of relative bending stiffness on settlement profile
Reproduced from Potts and Addenbrooke (1997)

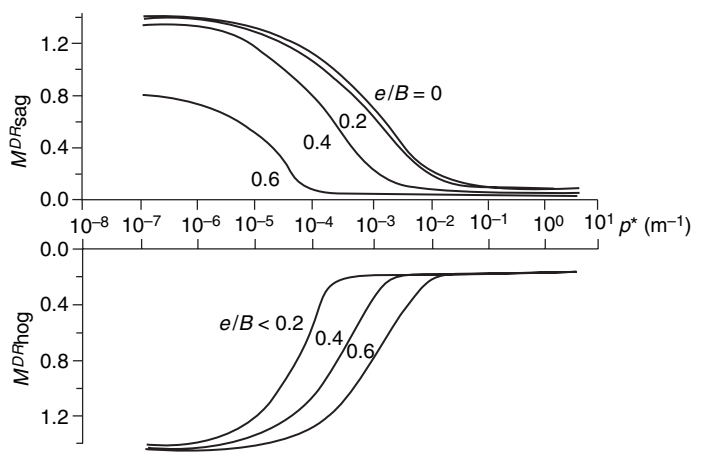


Figure 26.11 Bending stiffness modification factors
Reproduced from Potts and Addenbrooke (1997)

26.8.4 Detailed evaluation

Detailed evaluation is carried out on those buildings that, as a result of the second stage assessment, are classified as being at risk of category 3 damage, or above (see **Table 26.1**). The approach is a refinement of the second stage assessment in which the particular features of the building and the tunnelling and/or excavation scheme are considered in detail. Because each case is different and has to be treated on its own merits, it is not possible to lay down detailed guidelines and procedures. Factors that are taken more closely into account are discussed below.

26.8.4.1 Tunnelling and excavation

The sequence and method of tunnel and excavation construction should be given detailed consideration, with a view to reducing volume loss and minimising ground movements as far as is practical.

26.8.4.2 Structural continuity

Buildings possessing structural continuity, such as those of steel and concrete frame construction, are less likely to suffer damage than those without it, such as load-bearing masonry and brick buildings.

26.8.4.3 Foundations

Buildings on continuous foundations such as strip footings and rafts are less prone to damaging differential movements (both vertical and horizontal) than those on separate individual foundations, or where there is a mixture of foundations (e.g. piles and spread footings).

26.8.4.4 Orientation of the building

Buildings oriented at a significant skew to the axis of a tunnel may be subjected to warping or twisting effects. These may be accentuated if the tunnel axis passes close to the corner of the building.

26.8.4.5 Soil/structure interaction

The predicted greenfield displacements will be modified by the stiffness of the building. The detailed analysis of this problem is exceedingly complex and resort is usually made to simplified procedures – some of which are described in the report published by The Institution of Structural Engineers (1989). The beneficial effects of building stiffness can be considerable, as demonstrated by some measurements on the Mansion House in the City of London during tunnelling

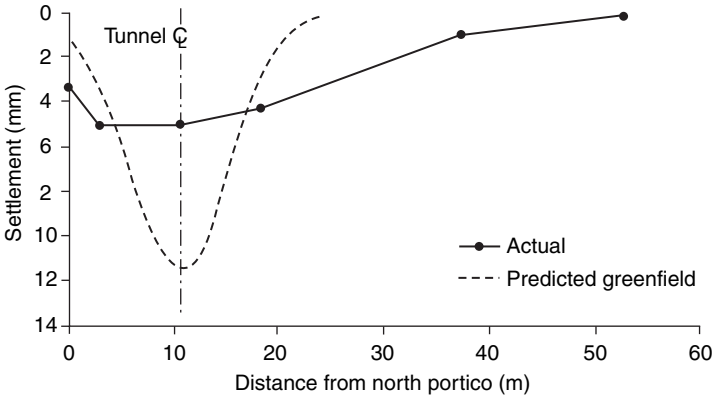


Figure 26.12 Comparison of observed and greenfield site settlements of the Mansion House, London, from driving a 3.05 m diameter tunnel at 15 m depth
Reproduced from Frischman *et al.* (1994)

beneath and nearby – see **Figure 26.12** from a paper by Frischman *et al.* (1994).

26.8.4.6 Previous movements

The building may have experienced movements due to a variety of causes such as construction settlement, ground water lowering and nearby previous construction activity. It is important that these effects be assessed as they may reduce the tolerance of the building to future movements. If the building is already cracked, an assessment should be made as to whether the tunnelling-induced movement will focus on these cracks. Sometimes this can be an advantage, as other parts of the building are then at less risk of damage.

As many factors are not amenable to precise calculations, the final assessment of possible degree of damage requires engineering judgement based on informed interpretation of available information and empirical guidelines. Because of the inherently conservative assumptions used in the second stage assessment, the detailed evaluation will usually result in a reduction in the possible degree of damage. Following the detailed evaluation, consideration is given as to whether protective measures need to be adopted. These will usually only be required for buildings remaining in damage categories 3 or above (see **Table 26.1**).

26.9 Protective measures

Before considering near-surface measures, consideration should be given to measures that can be applied from within the tunnel to reduce the volume loss. There are a variety of such measures, for instance increasing support at or near the face, reducing the time to provide such support, the use of forepoling, soil nailing in the tunnel face and the use of a pilot tunnel. These approaches tackle the root cause of the problem and may prove much less costly and disruptive than near-surface measures.

If, for a particular building, tunnelling protective measures are considered either technically ineffective or too expensive, then it will be necessary to consider applying protective measures near the surface or to the building itself. However, it must be emphasised that such measures are generally disruptive and can have a significant environmental impact. The main forms of protective measures currently available fall into the following six broad groups.

26.9.1 Strengthening of the ground

This can be achieved by means of grout injection (cement or chemical) or by ground freezing. It is normally undertaken in granular water-bearing soils. Its primary purpose is to provide a layer of increased stiffness below foundation level or to prevent loss of ground at the tunnel face during excavation.

26.9.2 Strengthening of the building

Strengthening measures are occasionally undertaken in order that the building may safely sustain the additional stresses or accommodate deformations induced by ground movements. Such measures include the use of tie rods and temporary or permanent propping. Caution is needed when adopting such an approach as the work can be very intrusive and may lead to greater impacts than simply allowing some cracking to take place which can subsequently be repaired.

26.9.3 Structural jacking

In some special situations (e.g. sensitive equipment) it may be possible to insert jacks and use them to compensate for tunnelling-induced settlement as it takes place.

26.9.4 Underpinning

There may be circumstances in which the introduction of an alternative foundation system can be used to eliminate or minimize differential movements caused by tunnelling. However, because the ground movements are usually deep-seated, such an approach may not be possible. If the existing foundations are inadequate or in a poor condition, underpinning may be used to strengthen them and provide a more robust and stiffer support system.

26.9.5 Installation of a physical barrier

Occasionally, consideration has been given to the installation of a physical barrier between the building foundation and the tunnel. This might take the form of a slurry trench wall or a row of secant bored piles. Such a barrier is not structurally connected to the building's foundation and therefore does not provide direct load transfer. The intention is to modify the shape of the settlement trough and reduce ground displacements adjacent to and beneath the building.

26.9.6 Compensation grouting

Compensation grouting is the controlled injection of grout between the tunnel and the building foundations in response to observations of ground and building movements during tunnelling. As its name implies, the purpose is to compensate for ground loss. The technique requires detailed instrumentation

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to monitor the movements of the ground and the building. Experience of compensation grouting is reported by, amongst others, Harris *et al.* (1994) and Harris (2001). The technique was successfully used on the Jubilee Line Extension Project in London for the protection of many historic buildings, including the Big Ben clock tower at the Palace of Westminster (Harris *et al.*, 1999). More recently, Mair (2008) described the successful application of compensation grouting in granular soils and the innovative application of directional drilling for installation of the grout tubes.

It cannot be emphasised too strongly that all of the above measures are expensive and disruptive, and should not be regarded as a substitute for good quality tunnelling and excavation practice aimed at minimising settlement.

26.10 Conclusions

Damage due to foundation movements is only one aspect of the wider problem of serviceability of buildings. The problem of coping with differential settlement, as with creep, shrinkage and structural deflection, may frequently be solved by designing the building, and in particular the cladding and partitions, to accommodate movement rather than resist it. Successful and economic design and construction of the total structure requires cooperation between the geotechnical engineer, the structural engineer and the architect from the earliest stages of planning.

In this chapter, clear definitions have been given of foundation movement; it is vital that these are understood and used. Also, a widely accepted classification of damage based on ease of repair is described. These two aspects form the basis of a coherent and objective approach to assessing limiting distortions of buildings. The concept of limiting tensile strain as a serviceability parameter is introduced as a means of gaining insight into some of the factors influencing limiting distortions in buildings.

The latter part of the chapter summarises briefly a rational and coherent approach to the assessment of risk of damage to buildings due to tunnelling and excavation. A three-stage approach is recommended for assessing the potential for damage due to subsidence. The first two stages neglect the stiffness of the building in assessing the magnitude of the induced strains within it. The assessments are therefore very conservative. The advantages of this staged approach are that it is simple to use and the buildings which require detailed analysis are identified without recourse to a large amount of unproductive complex analysis.

A wide variety of protective and mitigation measures are described to minimise impacts of ground movement. In general, it is much better to control ground movements by controlling the tunnelling operation (so as to minimise ground loss) than to embark on expensive building protective measures which can themselves be disruptive and damaging.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

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Chapter 27

Geotechnical parameters and safety factors

John B. Burland Imperial College London, UK
Tim Chapman Arup, London, UK
Paul Morrison Arup, London, UK
Stuart Pennington Arup, London, UK

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A key aspect of geotechnical design involves the choice of appropriate parameters for characterising the ground. This chapter describes how the engineer should set about this important aspect of design. Great emphasis is placed on the fact that the selection of the appropriate geotechnical design parameters must be undertaken in the context of overall considerations of risk. A number of examples are given to illustrate how the characteristic values and design values of various geotechnical parameters are selected for a variety of problems. Much of the discussion relates to the requirements of Eurocode 7 but reference is also made to other more traditional approaches.

27.1 Introduction

27.1.1 The design process

This chapter presents the process that should be followed for choosing the appropriate geotechnical parameter for a particular safety factor regime – the two should always be chosen in tandem. The chapter generally follows the approach proposed by Part 1 of Eurocode 7 (BS EN1997-1:2004). Other design codes are also referenced to illustrate how different approaches to design can be used to achieve a similar result. Eurocode design is based on a cautious estimate of the parameters which are used along with partial factors to calculate design values. These design values are then used to assess whether a limit state is being satisfied (e.g. the design axial load on a pile is less than the design resistance to axial load of the pile).

The choice of geotechnical parameters and partial factors is a balance between economy and reliability. (Design to the Eurocode uses partial factors rather than factors of safety. Within geotechnical design to Eurocode 7 partial factors are applied to both material strengths (or resistances in piles) and applied actions, so as to factor each element in the design according to the degree of uncertainty to which the parameter/action is known and according to the implication of variation in the parameter's/action's value.) A cautious approach is generally safer and the resulting structure will be more robust with a much lower risk of under-performing than if a less cautious approach is taken. However, few clients are prepared to pay for an excessive level of redundancy resulting in design being the art of making elements safe and functional as well as economical. In geotechnical design especially, where the risk of failure is very real and the ability to predict failure accurately is not easy, the art of design is to select a design basis that is robust, but not excessively so.

Design is usually carried out within the constraints of a code, e.g. Eurocode 7. The preparation of a code allows the experience of others to be encapsulated in a concise manner. As such the choice of appropriate ground parameters must be compatible with the intent of the code writers for the values of partial factors that are to be used. Further, the choice of geotechnical parameters must be made in the context of the complete structural design being embarked upon. In order for geotechnical parameters to be identified and assessed through the design and construction process the following actions are recommended (see **Table 27.1**).

In addition to the actions detailed in **Table 27.1**, the following considerations are important in ensuring good design:

- Consideration of how a design can be robust to accommodate unforeseen or unexpected loading events, this consideration is important to both ground and structural elements.
- Utilisation of a procurement processes in which the allocation of risk is carried out appropriately and which rewards experienced contractors who make proper allowance for those risks for which they are responsible. To deal with unexpected conditions the contract should place an onus on the contractor to identify such conditions at as early a stage as possible and should thereafter allow for reasonable compensation (time/money).

The confidence that errors in design or construction have been avoided can always be increased by additional third-party checking, both of the design process (such as Category 3 checks proposed by the Highways Agency standards) or on construction sites (such as independent supervision, full-time on-site auditing of the processes and verification of records). These factors are explored by Chapman and Marcetteau (2004).

The management of temporary works risks during construction can include use of the Observational Method – this permits the adoption of less pessimistic parameters and replaces the otherwise lost confidence by more robust management processes, supported by construction monitoring and a set of

Action	Output (Eurocode 7)
■ A well-planned desk study should be carried out, this may include a site walkover (recommended) as well as a preliminary site investigation – see section 27.4.	Desk Study Report with preliminary Ground Model
■ Carrying out a well-planned ground investigation(s) that responds to the risks identified in the desk study. Interpretation of the findings of the investigation allowing for sufficient time to monitor groundwater and environmental gasses to be completed. Part 2 of Eurocode 7 'Ground investigation and testing' (BS EN 1997–2:2007) give suggested scopes, etc.	Factual and Ground Investigation Report with refined Ground Model
■ Use of experienced and knowledgeable designers who take part in a coherent design process using proven models which include consideration of geotechnical, structural and performance requirements	Geotechnical Design Report with recommendations for construction and monitoring
■ Clear design documentation (Geotechnical Design Report) must state the assumptions made in the design in a clear manner to allow the professional involved during the construction (execution in Eurocode terminology) phase to assess ground conditions against the design assumptions. Re-evaluation of the geotechnical parameters/conditions may be necessary at any stage of design and construction. In the design of geotechnical elements it is important for the design to anticipate the construction processes to be used, as those processes may affect the magnitude of the design parameter that is used – for instance different methods of piling cause different values of skin friction.	
■ Where appropriate, testing of trial constructions (e.g. preliminary test piles) may be necessary to validate the design models and geotechnical parameters adopted. Testing samples of the works (e.g. working pile load tests) may also be necessary to verify that the constructed elements do not differ from the trial constructions or design intent.	Refinement of Geotechnical Design Report.

Table 27.1 Recommended actions for identifying and assessing geotechnical parameters

feasible predetermined contingency measures. This is covered by Nicholson *et al*. (1999).

Within the stricture of modern design codes there is limited opportunity for waiver from the partial factors (or factors of safety) stated in design codes. It is only for design of the most basic of structures that design based on comparable experience is permissible. (It is noted that for structures of high importance, such as hospitals in earthquake regions, there is a logical need for increased safety in these structures due to the consequences of failure being so great.) As such it is vitally important that the choice of ground parameter values is appropriate to the level of safety provided for in the code's stated partial factors; the designer must maintain control of the design process by informed choice of the geotechnical parameter to use within a framework of partial factors.

27.1.2 Design codes

In design to Eurocode 7 the value of the partial factors (e.g. in the UK NA to BS EN 1997-1:2004) is based on the assumption that the designer will adopt a ground parameter value that is a cautious estimate of the parameter's value relevant to the limit stage being considered (Clause 2.4.5.2 of BS EN1997-1:2004). Failure to adopt a cautious estimate of the ground strength would result in a potentially unsafe construction. In contrast to a cautious estimate approach, the LDSA note for pile design in London (LDSA, 2009) stipulates that the undrained strength of London Clay should be taken as an average value (and not a cautious estimate). Such an approach is reasonable based on the great knowledge from long history of pile testing to failure in the area and due to the size of factor of safety applied to pile resistance; the combination of mean ground strength and factor of safety combine to provide a level of confidence deemed reasonable. Both of these approaches, Eurocode 7 and the LDSA, and many others, are valid when applied consistently across a design; they must not, however, be mixed.

The list of codes in **Table 27.2**, including Eurocode 7 and the LDSA note, shows how the degree of conservatism used in the geotechnical parameter is balanced by the value of the partial factor on the geotechnical parameter. The methods are all different but aim at achieving similar end constructions.

Mean value (LDSA guidelines)	High value of partial factor/factor safety
Moderately conservative line or conservative best estimate (CIRIA R185 *The Observational Method in Ground Engineering*)	
Representative line or conservative estimate (BS8002 'Code of Practice for earth retaining structures')	Median values of partial factor/ factor safety
Characteristic line or a cautious estimate (Eurocode 7)	
Worst case line or a realistic worst value (CIRIA C580 *Embedded Retaining Walls – Guidance for Economic Design*)	Low value of partial factor/factor safety

Table 27.2 Code comparison

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27.2 Overall consideration of risk

27.2.1 An example

When considering the choices for partial factors and ground parameters it is necessary to consider the overall risk regime. Eurocode 7 Part 1, Clause 2.1, introduces the concept of safety categories for design, where Category 1 is for small and relatively simple structures with negligible risk, Category 2 is for normal structures and Category 3 is for special structures where the consequences of failure are worse than normal.

Some of these concepts are illustrated in **Figure 27.1**.

The idealised example of a 'Project' in **Figure 27.1** is provided to highlight the need to design not only for the 'Project' but also for the surrounding context of the project. It is suggested that the design of the foundations for the project building need have no more than typical safety requirements (as supplied by the combination of caution in choice of geotechnical parameters and partial factors) on the assumption that local failure of the building is not anticipated to impact on the nuclear power plant below. However, the design of the retaining wall above the nuclear plant will need to have enhanced safety due to the consequences of its failure. This enhanced safety would logically include larger partial factors on the building loads from the 'Project' than would be used to design the building itself.

Considering the retaining wall, the design for the retaining wall supporting the building will be subservient to the design of the wall protecting the nuclear plant. The implication of this is that the combination of geotechnical parameter and partial factor used to design the wall to protect the plant will be the controlling feature and will result in lower design soil strength than would be used for design of the 'Project' foundations (same ground different levels of safety).

In section 27.2.2 below the concept of risk category is presented; in this context the 'Project' could be a category 2 structure, while the wall should be a category 3 structure. It must be recognised that the stability of the building depends crucially on that for the retaining wall, so the building designer should be satisfied that the retaining wall is sufficiently stable, and has a design life commensurate with that intended for the building.

As a parting comment on this example, what category would be placed on assessment of the slope uphill of the project building? If a slope failure could impact on the nuclear plant then its category would be 3, if not, then 2.

27.2.2 Risk factors

The factors which may affect the level of caution are summarised in **Table 27.3**.

Precautionary measures to reduce the risk for geotechnical elements can be applied by some of the following ways:

- robust design processes including ground investigations;
- checks on design;
- checks on construction;
- some redundancy in the structures (e.g. many piles per column).

27.2.3 Geotechnical categories to Eurocode 7

The concept of geotechnical category to take account of risk as presented in Eurocode 7, Part 1, as introduced in the preceding section is useful in defining the level of caution that is

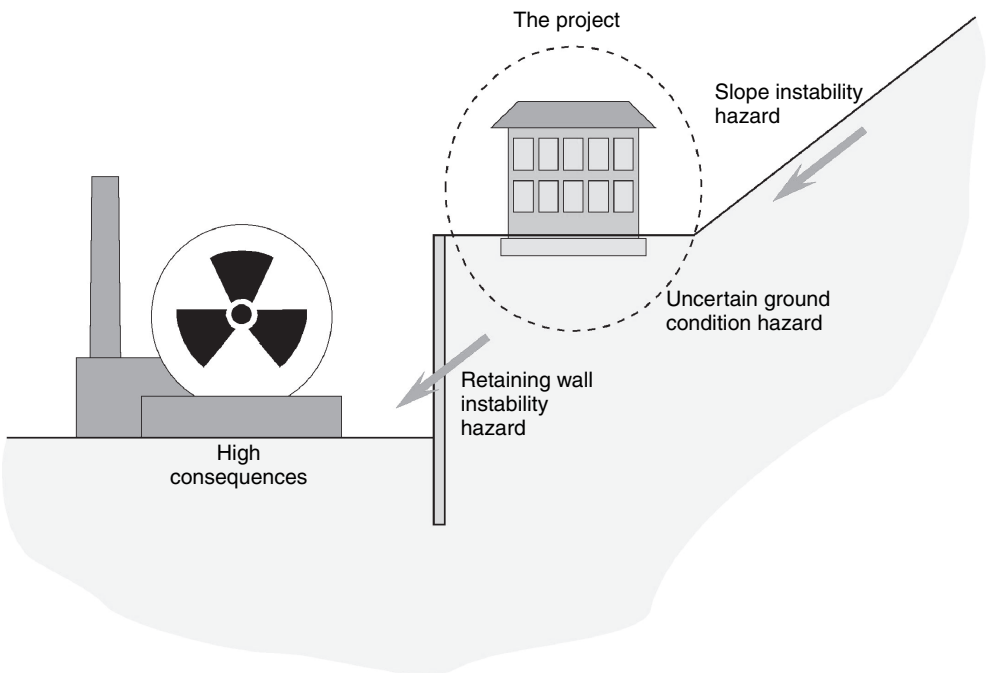


Figure 27.1 The context of a design problem

appropriate when considering the design of a structure. These geotechnical categories are summarised in **Table 27.4**.

An analogous assessment of risk is presented in the Eurocode (BS EN1990:2002+A1:2005).

27.3 Geotechnical parameters

27.3.1 General

Geotechnical conditions are represented in design by series of geotechnical parameters which are used to characterise the complex properties of the ground and groundwater such that they can be modelled for the purposes of that design. Geotechnical parameters can be measured directly or obtained through the use of correlation, theory and empiricism or they can be sourced from published data; it is often best to obtain geotechnical parameters by more than one way to allow confidence in the variability of the parameter to be developed.

In everyday usage, the terms 'geotechnical parameters' and 'design parameters' are often used interchangeably. However, in the context of this chapter and in the Eurocodes, a design value is the particular value of a geotechnical parameter selected for a specific design purpose. The numeric value of the design value will vary depending on the limit state being considered as a function of the particular level of safety required in the specific calculation being carried out and the manner in which safety is achieved in the calculation (factoring of actions or factoring of resistances or a combination of both). For ultimate limit state calculations the design parameter is typically the factored version of the geotechnical parameter. For serviceability limit state the design and geotechnical parameters are typically equal. In all cases the partial factor is applied to adjust the geotechnical parameter detrimentally. Sometimes an upper value of a parameter may result in a less safe condition than a lower one – hence just picking a lowerbound value is not always appropriate without understanding the calculation to be carried out. This may mean that it is necessary to increase the 'Cautious upper' estimate of the geotechnical upwards rather than downwards to obtain a design line to use in ultimate limit state assessments.

In certain circumstances, rather than determining a unique value of a geotechnical parameter or a design parameter it may be appropriate to consider a range of parameters to see how these influence the design being carried out; such a situation may be applicable to ground strength but is more typically applicable to ground stiffness. The estimate may have varying levels of caution depending on the limit state being considered and this should accommodate the balance between caution in choice of parameter and caution inherent in the magnitude of factor of safety or partial factor being applied.

A geotechnical parameter is the culmination of an interpretation process in which geotechnical data are critically examined through a process which considers code requirements and issues such as:

- Ground model
  - geological history;
  - uncertainty with respect to the ground conditions;

Factors due to the supported structure	Factors due to the project or its situation
Structural form very vulnerable to damage or collapse, e.g. statically determinate.	Difficult or complex ground, or poorly understood ground conditions.
Columns supported by single piles.	High or very variable groundwater conditions and groundwater flows.
High economic consequences of failure, e.g. the structure is vital to the cash flow or financial well-being of a major organisation.	Seismic conditions or ground otherwise unstable, e.g. on a hillside suspected as prone to failure.
High safety consequences of a failure, e.g. risk that many people may be injured or killed by a geotechnical failure, such as a concert hall or sports stadium.	High live loads give a greater level of uncertainty of the maximum load likely to be applied. Cyclic loading resulting in large settlement.
High safety consequences of a failure, due to the processes carried out within, such as a chemical production or electricity generation from nuclear power.	Ground conditions where accurate prediction is difficult. Complex ground, or poorly understood ground conditions; attention to settlement at working loads and how these vary over time.
Monitoring and maintenance regimes important.	
Choice of foundations which are unproven in the circumstances or where comparable experience suggested a high rate of defects.	Appointment of an inexperienced or poorly equipped foundation contractor.

Table 27.3 Consideration of risk

	Category 1	Category 2	Category 3
Risk level	Negligible	Not exceptional	Abnormal
Groundwater level	Construction above the water table only	Construction above or below the water table	Construction above or below the water table
Ground	Straightforward with good record of performance for type of structure proposed	Routine	Exceptionally difficult
Type of analysis	Qualitative	Quantitative	Special (quantitative)
Typical structures	Small and relatively simple	Normal spread footings, piles, rafts, retaining walls, etc.	Large or unusual; or in highly seismic areas, etc.

Table 27.4 Geotechnical categories
Data taken from BS EN1997-1:2004

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- scope of ground investigation – is it adequately thorough?
- groundwater conditions and variation with time;
- treatment of anomalous data;
- anisotropic or heterogeneous ground;
- brittle or strain softening ground.

- Risks associated with proposed structure and design simplifications:
  - type and complexity of the intended structure;
  - required design life;
  - geotechnical design models;
  - scale/rate/strain effects;
  - comparable experience.
- Construction related factors:
  - anticipated construction methods and workmanship;
  - adjacent site activities.
- Social considerations:
  - level of acceptable risk/importance of the intended structure.

27.3.2 Eurocode 7

Eurocode 7 uses four terms when referring to geotechnical data:

- raw data;
- derived values;
- characteristic values;
- design values.

Raw data are, as the name suggests, data collected in their most basic format, for example, standard penetration test (SPT) N values.

Derived values are ground properties interpreted from raw data directly or indirectly (e.g. correlation, theory and empiricism), and from other sources such as published data. An example of an indirectly derived ground property could be undrained shear strength based upon SPT data.

Characteristic values are the basic form of geotechnical parameters used in design and are defined in Eurocode 7 as 'a cautious estimate of the value affecting the occurrence of the limit state'. They are selected from derived values.

Design values are characteristic values that have been modified by a partial factor determined according to the limit state being considered and the design approach being adopted. It is noted that in some instances (e.g. when dealing with water pressures) design values may also be selected directly without the use of partial factors.

27.3.3 Traditional codes

Traditional codes such as British Standards are typically based on working stress design approaches (with global factors of safety rather than partial factors) though some are based on limit state design principles. They are not as explicit with their definitions of geotechnical data as Eurocode 7 is, however, they do have similar terms to the Eurocode 7 characteristic value as previously mentioned in section 27.1.2 above.

27.3.4 Considerations for parameter choice

27.3.4.1 General

The partial factors in Eurocode 7 are based on a geotechnical parameter being a cautious estimate of the value controlling the limit state. Within Eurocode 7 this value of the geotechnical parameter is called a 'characteristic value'. Clearly this cautious estimate of the geotechnical parameter implies a value that is offset from what is deduced to be the most probable value by a margin to provide a level of caution.

Choosing a characteristic value is not about putting a line through a set of data. It is about understanding the issues that lead to the choice. Once these are understood, the selection of a characteristic value or line should be straightforward, and whilst it could be curved, straight, stepped, constant with depth or increasing with depth, an overly complex line is generally not necessary or helpful. An overly complex choice more frequently leads to errors in calculations.

Whilst the use of statistical methods to determine a characteristic line, by attributing a statistical definition to the characteristic value, has been promoted by various authors, they are often a trap. It should be kept in mind that the ground is not a well-behaved material (e.g. concrete), data sets are often small, not of normal distribution, and they can contain significant scatter – not an ideal situation for the application of rigorous statistical approaches.

Rather than try to describe how a characteristic line could be selected, it seems more appropriate to communicate the idea via an example. The following example provides a simple illustration of how such an assessment could be made in the context of Eurocode 7.

27.3.4.2 Illustrative considerations for shallow foundations

For shallow foundations, design parameters are very dependent on the construction process and in particular, the preparation of the bearing surface. They also typically engage far less ground than, for example, a pile, and can be affected by localised changes in ground properties or by non-typical soils such as sensitive clays, loose wet sands or collapsible soils. Sometimes those more onerous conditions may lurk just below the exposed formation and may not be observed in the formation preparation. Geotechnical parameters need to be selected with these issues in mind.

27.3.4.3 Illustrative considerations for piled foundations

For piles, design parameters are very dependent on the construction processes used to form the piles and the clear separation of design and construction responsibilities is not as simple as in other branches of engineering. For instance, if a pile fails is it because the geotechnical parameters were too optimistic for the

likely construction methods, or because the construction methods failed to live up to the standards expected by the designer?

Geotechnical parameters need to be selected in full cognizance of:

- the likely construction methods;
- the range of results of load tests carried out for the particular pile type and ground conditions;
- the specification to be employed, to ensure that processes known to be detrimental are not used, or where it would be impractical for them not to be used, that the design parameters are reduced accordingly.

Furthermore, the detailed choice of construction methodology needs to be made in full cognizance of the design assumptions and parameters.

27.3.4.4 Illustrative considerations for slopes

For slope stability, it is often a thin layer of soil that is most critical for the stability of a slope. For example, the back analysis of the failure at the Carsington dam by Potts *et al.* (1990) showed that a thin layer of soil had been neglected in the analyses. Furthermore, understanding the groundwater regime is equally important and in particular, accidental events such as burst water pipes should not be forgotten.

The selection of design parameters is absolutely critical for slopes and it may be that just one data point (perhaps an outlier) is most representative for the behaviour of the overall slope.

For slopes, too often a computer is used to plot results of several drained triaxial tests, and the choice of what should be characteristic values for effective cohesion (c′) and effective

Box 27.1 Case study 1

This example is one of a pile embedded in a stiff clay formation which is subjected to a fixed lateral displacement at the top of the pile 1 m above ground level as shown in **Figure 27.2**.

The undrained shear strength, measured on 100 mm diameter samples tested in quick undrained triaxial compression, with depth is shown in **Figure 27.3**.

The measurement of undrained shear strength has been carried out by means of undrained triaxial compression tests. The clay stratum was logged as being uniform with the exception of a sand layer at 16.5 m depth. Prior to lines being *drawn* on the data it is necessary to review the data to isolate any results that are not representative of the ground. Notes are provided in **Figure 27.3** for three of the data points which are considered to be atypical of the *in situ* state. The high value at approximately 6.5 m depth is noted in the laboratory test log as having dried out; such data may be excluded from further assessments. The two data points at 16 to 17 m depth appear to be soft and coincide with a sand layer; it is likely that sand and water from the sand layer have been included in the samples obtained allowing the clay to swell within the sample tube prior to testing. While the measurement of undrained strength is therefore not representative of *in situ* conditions the observation that the sand layer may have implications for pile construction should be carried through to the design process. The end product of this review is that three data points have been removed from the ground data prior to assessing the characteristic strength of the clay.

It is usual to assume that a characteristic design line (cautious estimate) through the data would be in keeping with the 'Cautious lower' line. By adopting this line the strength of the soil will be towards a lower bound. However when considering the limit state relevant to the structural design of the pile adopting this line as the characteristic property of the ground will likely underestimate the bending moment and shear force in the pile as it would result in a low assessment of mobilised resistance of the ground to pile head movement (δh). Hence for this particular design case the correct characteristic design line through the undrained shear strength data will be the 'Cautious upper' line as only this will provide the appropriate resistance to the pile head movement for carrying forward to structural design of the pile.

The lines shown for the 'Cautious lower' and 'Cautious upper' are open to debate and discussion and tweaking. For the data set used the lines are close to being parallel; this is clearly not a requirement and the splay from the average line should be a function of the scatter in the data and the confidence that the designer has in the data.

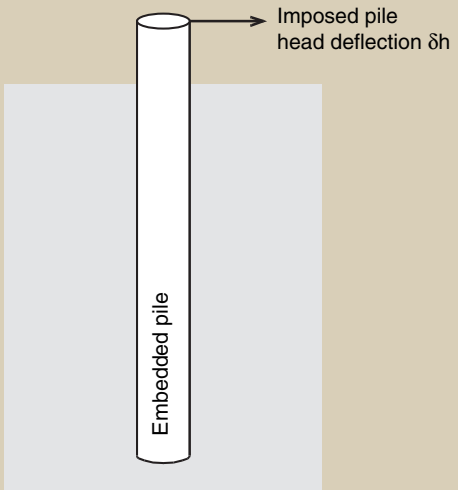


Figure 27.2 Laterally loaded pile

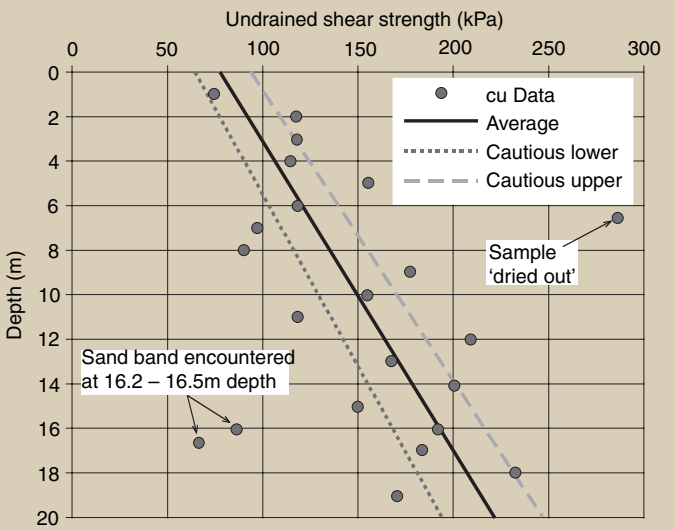


Figure 27.3 Selection of characteristic line

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angle of internal friction (φ') is not made with sufficient understanding of the true soil behaviour. c′ is unlikely to persist for shallow cut slopes as it will be lost due to weathering and φ' should be chosen over a stress range that is representative of that which will occur in practice.

Box 27.2 Case study 2

An example of how the construction process can lead to problems is seen in CFA piles. A good CFA pile may corkscrew its way into the ground, thereby creating a pile with a ribbed shape. This creates a high capacity pile with a shaft resistance which obtains its resistance from a failure interface between soil and soil rather than a failure interface which is between remoulded soil and smooth concrete. Continued auger rotation during the construction process can, however, quite quickly remove the beneficial ribs by a low rate of penetration per revolution. Hence parameters derived from a test on a very well-constructed pile may not apply to a pile built by the same equipment but with less control, or where circumstances have led to greater auger rotations.

An example of how design parameters could be too optimistic is illustrated by shaft-grouting. For a shaft-grouted pile, it is sometimes difficult to discern what has been improved – has the interface been made rougher, has the pile diameter increased or have the horizontal soil stresses been increased? Learning lessons from load tests requires an understanding of the physical processes in order to decide which parameters have been improved.

27.3.4.5 Illustrative considerations for retaining walls

For retaining walls, critical geotechnical parameter choices can include:

- appropriate selection of water pressures and cognizance of the effect of seepage throughout construction stages, and design life;
- allowance for locked-in earth pressures caused by compaction plant or integral bridges;
- knowing when undrained behaviour can be relied upon;
- allowing for construction mistakes such as overdig.

27.4 Factors of safety, partial factors and design parameters

27.4.1 General

Geotechnical design falls into two main categories: design against failure and design against unacceptable movement. In limit state design theory, these two concepts are known as breaching an ultimate limit state and breaching a serviceability limit state.

In many instances the historical choice of the value of a factor of safety allowed for settlements or lateral movements

Box 27.3 Case study 3

In the following example a situation is described along with discussion of how a designer may accommodate the ground conditions in the calculation process.

Situation: Open cut excavation in a stiff high plasticity clay stratum (see **Figure 27.4**).

Context: Long-term design case critical.

Consideration: Case history records of open excavations in stiff overconsolidated high plasticity clays exhibit progressive failure whereby a range of mobilised strengths (c′, φ') values are mobilised at different parts of the resulting failure mechanism.

- What design parameters should be used in design (long-term conditions)? How is progressive failure accommodated?
- Measurements of shear strength parameters may be made in the laboratory (triaxial, shear box, ring shear, etc). These typically return values of peak resistance and may also show strength reduction with increasing shear strain. How can these results be carried into the analysis of slope stability?
- One option would be to incorporate in a sophisticated numerical analysis model of the slope and ground a soil model which fully accounted for the strain softening of the ground – in this manner a geotechnical parameter model could be populated with appropriately cautious ground parameters. In subsequent calculations partial factors can then be applied to achieve the appropriate level of risk. This is not a standard approach and not one advocated for general design.
- An alternative solution would be to look at the worst possible strength parameters (e.g. residual strength) that are considered possible and to use them with reduced partial factors, possible equal to unity, to provide an equally coherent design approach.
- A third approach would be to use case history data from similar slopes to derive 'safe' parameters to bring forward into the design process. Clearly for this approach to be valid a large set of data for slopes with age similar to the design life of the project slope would be necessary; extrapolation for time-dependent failure is not viable.
- Adjustments will be necessary to any design input to allow for environmental changes in the future that are not replicated by past experience (e.g. global warming and associated changes in precipitation and temperature variations).
- Clearly, consideration of strength parameters is only one part of the stability assessment. Equally important is the consideration of water pressures in the slope. Such considerations need to look at anisotropy of permeability and recharge boundaries. Any drainage provisions that are put in place will need to be maintained if they are to be relied upon in design (it is sometimes the case that drainage provisions are assumed to fail due to risk of not carrying out maintenance).

In this example three possible approaches to the slope stability are proposed. They are quite different in approach (cautious estimate of strength to worst possible estimate of strength) but by using an appropriate value of partial factor in each case the design can be shown to be coherent. As always, water pressures need to be chosen correctly.

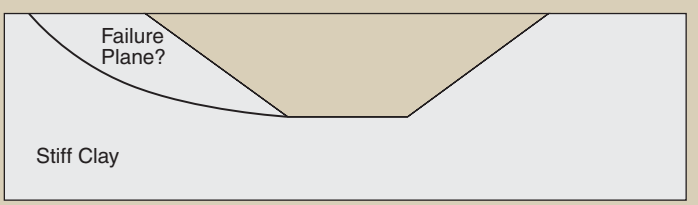


Figure 27.4 Open cut excavation

of a structure that were acceptable for typical performance requirements. Use of such all-encompassing factors of safety is acceptable (even in Eurocode, for example, Geotechnical Category 1) where there is well-established experience allowing 'rules of thumb' to be used. Where such rules do not exist, or where they are not suitable for the level of rigour required, then the designer must assess serviceability explicitly.

In other situations the factor of safety has simply been a factor to create a margin against a real failure, such as rupture, or constant movement. Given the provenance of historic factors it is necessary for the designer to be aware of the implications in choosing a particular value to ensure that the resulting performance is satisfactory in terms of serviceability and ultimate limit states for the particular design being undertaken.

Often geotechnical factors are composite and are created so as to be sufficiently high to prevent both ultimate and serviceability failure. This is one reason why geotechnical factors are sometimes higher than in other branches of engineering. Another is that ground engineering is an inexact subject and utilises materials with wide natural variation and hence the extent of uncertainty in any parameter is much larger than for manufactured materials.

27.4.2 Eurocode basis

The Eurocode use of partial factors is significantly different from a factor of safety approach, in that they are written in terms of limit state principles rather than those of working stress design. The creation of Eurocode 7 brings geotechnical engineers into line with the longer established structural engineering limit state approach to design which will hopefully lead to a more efficient and robust design process and better understanding between geotechnical and structural engineers.

In limit state design the designer is obliged to think about the problem at hand in terms of a particular limit mode and level of risk. Safety is applied in the form of partial factors on individual parameters in a way that more rigorously takes account of uncertainty compared with the lumped factor of safety approach. However, caution should be exercised given the complexity of design to Eurocode 7. There is a risk that the more intricate design concepts and processes will cloud the designer's mind.

The Eurocodes divide limit state design into ultimate limit states and serviceability limit states:

- Ultimate limit states (ULS) are those concerned with collapse and safety; they are not intended to be realistic in terms of the day-to-day

Box 27.4 Case study 4

In the following example a situation is described along with discussion of how a designer may accommodate the ground conditions in the calculation process.

Situation: Basement formed in a retained excavation in stiff clay with layers of water-bearing sand (see **Figure 27.5**).

Context: Design for short- and long-term conditions.

Consideration: The presence of water-bearing sand layers suggests that the overall behaviour of the ground to a retained excavation will be dominated by the presence of the sand layers.

What design parameters should be used in temporary and permanent design?

- Typically in stiff clay undrained parameters are assumed for temporary works design. This is possible due to the drainage path length preventing all but the exposed formation from swelling. However, in this situation, what impact do the sand layers have? The designer must assess the effect that these layers have on the rate that the clay layers can swell and thus the length of time that the undrained strength can be relied upon. Is the geotechnical parameter a drained or undrained parameter? Caution would indicate a drained parameter in the absence of other case histories or additional data.
- The presence of a sand layer close to the toe of the wall requires consideration of uplift of the soil below the excavation. Could the base of the excavation 'blow'? Has the GI investigated both water levels and also supply rate to allow such a design decision to be made? What design conditions should be attributed to the sand layer (permeability and recharge potential)?
- The assumption of the retaining wall forming a cut-off below formation level cannot be taken for granted without careful construction and trial dewatering. If the wall is not continuous then pore pressure in the upper sand band may be recharged through the retaining wall and cause loss of passive resistance in the zone immediately below formation level (this is the zone that controls retaining wall movement the most). What design assumptions are made to the retaining wall, and how can these be validated?

While design parameters are usually considered in terms of strength and stiffness, in this case in order for these to be useful they need to be understood in the context of permeability of the ground, the anisotropy of the ground and the boundary conditions. It is as important to make cautious assessment of these other factors as it is to strength and stiffness.

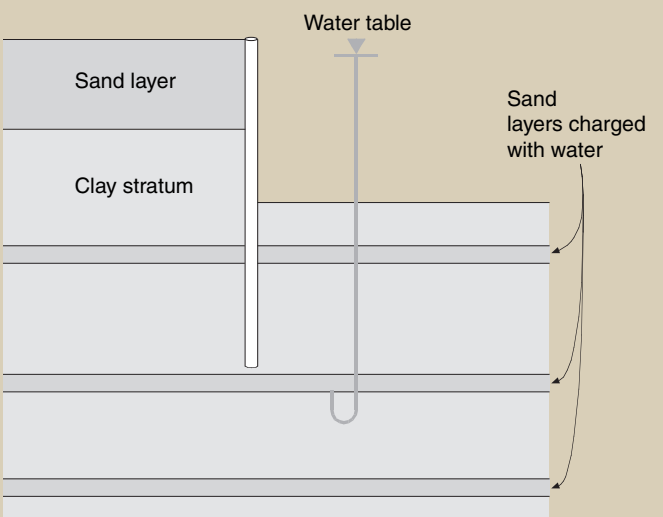


Figure 27.5 Retaining wall in layered soil

performance of a structure. Accidental cases are also covered by the ULS condition.

- Serviceability limit states (SLS) are those concerned with the day-to-day requirements for a structure such as comfort of people and cracking.

In order to verify that potential limit states will not occur, Eurocodes typically adopt the method of partial factors to achieve a suitable level of safety. Eurocode 7 goes one step further to define the following ways by which limit states may be verified:

- by prescriptive methods ('rules of thumb');
- using calculation methods;
- through models and load tests;
- by observational methods.

In the partial factor method, actions, materials, resistances and geometry are factored using prescribed, partial (γ) and combination (ψ) factors. The majority of factors contained within the Eurocodes have been determined by calibration to well-established experience so a design undertaken using them should be similar to conventional designs undertaken with a single lumped factor of safety. Some factors have, however, been determined in a probabilistic manner from laboratory and field data.

27.4.3 Traditional code basis

Traditional British Standards for geotechnical design (note: the Eurocodes are British Standards) are generally now withdrawn and replaced by Eurocode 7 in the UK. However, the traditional standards often remain as non-contradictory-complementary-information (NCCI) referenced in the Eurocodes. This means that the information within them can be used in conjunction with Eurocodes provided it does not contradict the Eurocodes. A further point to note is that Eurocode 7 is currently only compulsory for public sector projects.

The traditional approach to factors of safety can be illustrated as follows:

> BS 8004, Clause 7.3.8: 'In general, an appropriate factor of safety for a single pile would be between two and three. Low values within this range may be applied where the ultimate bearing capacity has been determined by a sufficient number of loading tests or where they may be justified by local experience; higher values should be used when there is less certainty of the value of the ultimate bearing capacity.'

In addition to the traditional codes, some cities adopt their own building codes. One example is the London District Surveyors Association (LDSA 2009) guidance notes for pile design. These 'local' codes can take precedence over the Eurocodes in the appropriate locations.

27.4.4 Consideration of safety

27.4.4.1 General

Historically, factors of safety in geotechnical engineering have incorporated both ULS and SLS considerations without being explicit about these limit states. For foundations, as per Eurocode, it is better practice for these to be considered

Box 27.5 Case study 5

For design against failure the factor of safety has historically been defined as the ratio of resisting actions to disturbing actions. The units of these actions can be force, moment or indeed pressure or stress depending on the calculation being carried out. In order to best understand the factor of safety, the following example is presented.

In **Figure 27.6** the factor of safety for the gravity wall in sliding is:

FOS = $\Sigma_{resisting} / \Sigma_{disturbing} = (P_{P1} + T_{S1}) / P_{A1}$. (Typically a value of 2 would be required.)

Where all actions are based on unfactored parameters rather than factored parameters.

It is noted that the 'resisting actions' should be taken as the sum of all resisting actions and the 'disturbing actions' the sum of all disturbing actions. When calculating a bulked factor of safety it is not admissible to consider the net disturbing or restoring action as part of the calculation.

With the introduction of limit state design, the term 'factor of safety' has been dispensed with and the level of safety is now accommodated through independent partial factors on actions, materials, geometry and resistances to arrive at design values. The resulting verification of safety for a particular ultimate limit state calculation becomes:

Design value of the effect of actions (E_d) ≤ Design value of the resistance (R_d).

Considering the loading conditions in **Figure 27.6** and using Eurocode 7 calculations requirements for the GEO/STR condition of the Ultimate Limit State (Design Approach 1 Combination 2 as per the UK NA to 1997-1:2004), all actions and material strength parameters attract unique partial factors. Safety is introduced by reducing the soil parameters by a partial factor which increases P_{A1} and reduces both T_{S1} and P_{P1} as well as increasing the detrimental influence of transient surcharge loading. Verification that the limit state has not been exceeded can be achieved by showing that the design value of P_{A1} is ≤ the sum of the design values of P_{P1} and T_{S1} where the design values of all three forces are based on factored soil parameters and surcharge loading.

Note: for Design Approach 1, Combination 2 of BS EN 1997-1:2004 the partial factors that must be applied are as follows: $\gamma_{\varphi'} = 1.25$ (applied to tan φ'); and $\gamma_c' = 1.25$; $\gamma_{cu} = 1.4$.

For all permanent loads $\gamma_G = 1.0$, while for variable loads only unfavourable loads are considered and for these $\gamma_{Q;unfav} = 1.3$ ($\gamma_{Q;fav} = 0.0$).

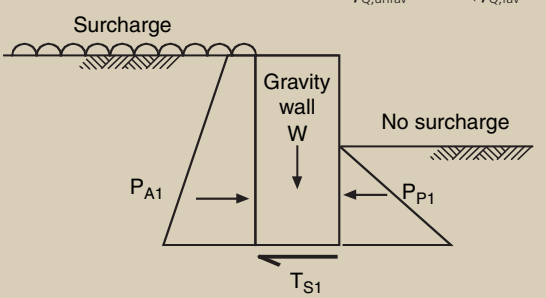


Figure 27.6 Factor of safety against sliding

separately. Factors just providing a margin against failure can be lower than those seeking to control movements, for instance.

27.4.4.2 Illustrative considerations for shallow foundations

For shallow foundations designed to Eurocode 7, the design bearing resistance is calculated using ground parameters modified by partial factor. This design bearing resistance is then compared with the design effect of actions which are also factored (note: in traditional design the ultimate bearing resistance would be factored and referred to as the safe bearing pressure).

The values of the partial factors for the two main Eurocode 7 ULS design situations for foundation design, namely GEO and STR, vary as to where partial factors are applied. For GEO the ground strength values are factored leaving density or surcharge effects largely unfactored or with small partial factors while for STR the actions or effects of the actions are factored (e.g. surcharge loading and calculated shear stress or bending moment in an element). By splitting into these two design cases it is possible to be confident of the geotechnical stability (usually this would be assessed first) and then the structural forces (usually calculated after the geotechnical stability is addressed).

Settlements (SLS) are considered separately and computed using partial factors of unity for the stresses imposed. They are compared to the values of settlement that it is thought that the structure of the function in the structure can withstand. If the calculated settlement is too high then the design is modified (note: in traditional design the safe bearing resistance would be reduced to an allowable bearing pressure if settlements controlled the design).

27.4.4.3 Illustrative considerations for piled foundations

In traditional design for piles, two criteria are often applied:

- An overall factor on the combined shaft and base resistances, giving a margin against failure.
- A reduced factor on the stiffer and normally more reliable shaft capacity alone, to limit movements to the stiffer part of the pile load/settlement curve, giving control on movements of the supported structure.

There are cases when this approach can be too conservative, such as when the pile base performance is known to be both stiff and reliable; in these cases the factor on shaft capacity is not used.

Under Eurocode 7, the capacity and settlement of a pile are usually dealt with separately; the design resistance being calculated by modifying the characteristic resistance by partial factors. However, where serviceability is not considered explicitly, an alternative set of partial factors on resistance are used.

For temporary works, sometimes lower safety factors are considered. However, their use is only appropriate where the higher resulting risks can be accommodated and where larger displacements are acceptable.

27.4.4.4 Illustrative considerations for slopes

For slope stability, overall moment and vertical stability are the key ultimate limit state concerns. Serviceability is generally related to the effect that slope displacement has on surrounding structures rather than internal displacements, unless there are aesthetic constraints to consider.

27.4.4.5 Illustrative considerations for retaining walls

For retaining walls, the geotechnical modes of failure are more varied and typically require separate calculations, for example, rotation about the top, rotation about the base, vertical displacement and horizontal displacement. Whilst safety is typically accommodated in partial factors on actions and ground parameters, geometrical variation and progressive failure checks are also important.

Serviceability is typically addressed in terms of lateral movement and the effect this has on supported structures, although excessive displacement of a wall into a basement, for example, may impact upon its amenity despite it not causing problems for structures that it supports.

27.5 Concluding remark

Geotechnical parameters and safety factors must be viewed holistically: the manner by which one is chosen needs to mirror the level of risk in the other. Beyond ground strength and designing against failure, the assessment of soil structure interaction, movement and performance of structures associated with the ground elements need also to be assessed with an understanding of the level of certainty in the design parameters.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.

Section 3: Problematic soils and their issues

Section editor: **Ian Jefferson**

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Chapter 28

Introduction to Section 3

Ian Jefferson University of Birmingham, UK

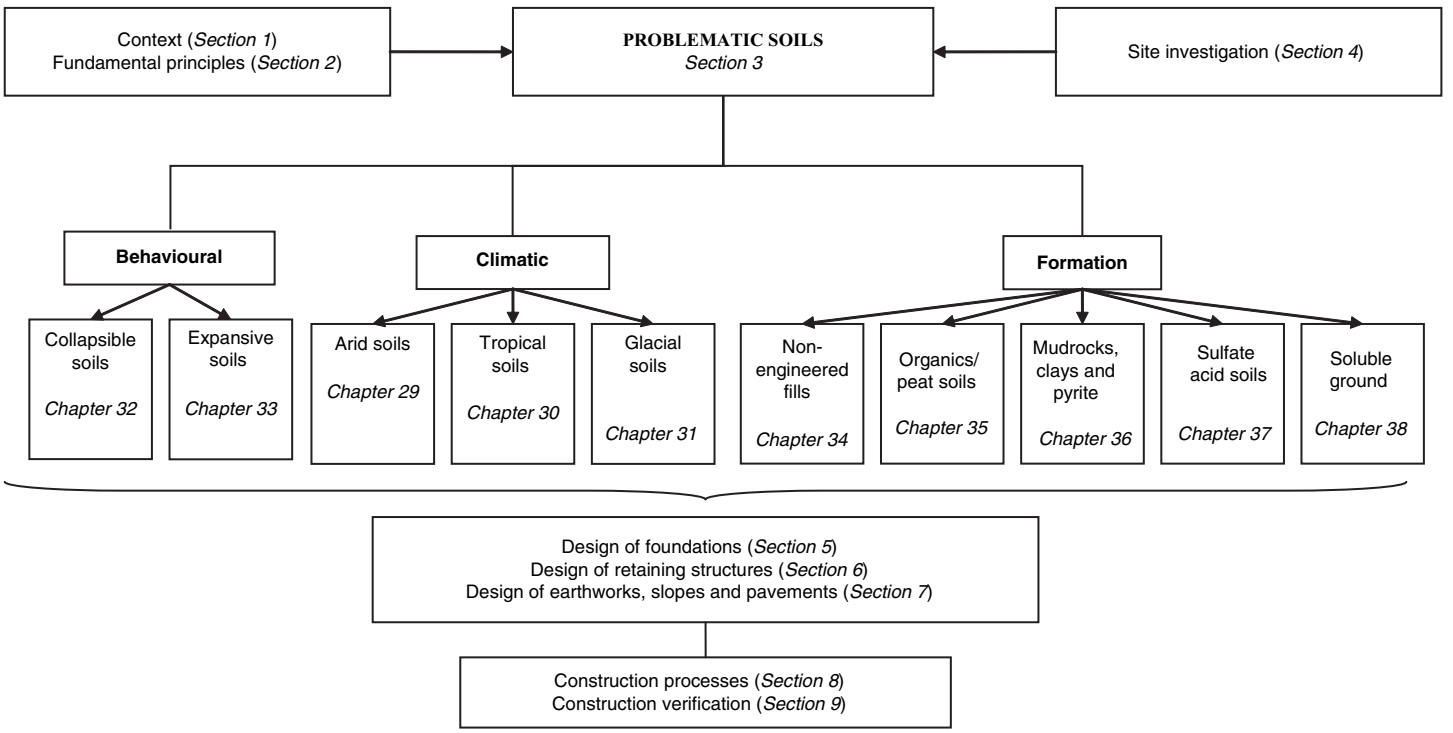


Figure 28.1 Layout of chapters in Section 3

Many soils can prove problematic to geotechnical engineering. Problematic characteristics include significant volume change, a distinct lack of strength, or soils that are potentially corrosive. This creates difficulties across built and urban environments and continues to cause billions of pounds of financial loss annually across the world: costs that often exceed those associated with earthquakes, floods, hurricanes and tornadoes combined.

The degree to which a soil is problematic to engineering is a function of:

(1) the nature of the soil itself (mineralogy, micro-fabric and geotechnical and other properties);

(2) the geological processes that caused it to be formed (whether fluvial, glacial, aeolian, or others);

(3) the processes that are acting on it currently (weathering, erosion and human activity).

However, problematic soils are also profoundly influenced by the climatic regime in which they were developed. This, creates many types of problematic soils and a number of key challenges when dealing with them. Nevertheless, these can be overcome by paying careful attention to:

- recognition (through geology and geomorphology);

- classification and assessment (through correlations, laboratory and field testing);
- evaluation (via use of appropriate models – empirical, analytical or numerical);
- treatment by application of suitable and appropriately targeted engineering solutions.

Thus problematic soils can be engineered effectively through observation, understanding and technological development, allowing soils that are considered problematical to be assessed in a rational manner using the approaches detailed throughout this manual. This will overcome what Leroueil (2001) considered as problematic engineering, designs, specifications or use of technologies that occur through a lack of appropriate or detailed understanding of the situation in hand. We are moving to ever-better understanding of soil behaviour; however, as Vaughan (1999) suggests, many of our geotechnical materials seem unusual or 'problematic' because their behaviour does not fit the classic theories of soil mechanics. Thus, modification of basic soil mechanics is needed to incorporate problematic soils. However, as yet this has not been fully accomplished and many soils continue to cause problems globally.

Section 3 *Problematic soils and their issues* covers the main problematic soils/ground encountered globally. These have been grouped to reflect the main process in their formation (see **Figure 28.1**):

(1) Behavioural:

- Chapter 32 *Collapsible soils*
- Chapter 33 *Expansive soils*

(2) Climatic:

- Chapter 29 *Arid soils*
- Chapter 30 *Tropical soils*
- Chapter 31 *Glacial soils*

(3) Formation:

- Chapter 34 *Non-engineered fills*
- Chapter 35 *Organics/peat soils*
- Chapter 36 *Mudrocks, clays and pyrite*
- Chapter 37 *Sulfate acid soils*
- Chapter 38 *Soluble ground*

These soil types or formations, which continually cause significant challenges, are associated with many of the difficulties experienced with problematic ground, both across the UK and throughout the world. Moreover, these constitute the problematic soils/ground commonly encountered in geotechnical engineering.While this does not represent all possible types of problematic soils, the approaches highlighted throughout Section 3 *Problematic soils and their issues* provide a useful benchmark from which all problematic soils can be dealt with.

The ultimate aims of this section are therefore to:

- highlight the importance of understanding geological and geomorphological aspects associated with problematic soils;
- identify methods to assess and evaluate problematic soils and their behaviour, and
- present methods to treat and engineer problematic soils effectively.

Overall this Section provides a sound basis of engineering in problematic soils and thus aims to help prevent many of the costly mistakes that continue to plague engineers dealing them.

28.1 References

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Chapter 29
Arid soils

Alexander C. D. Royal School of Civil Engineering, University of Birmingham, UK

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Arid soil forms when more water evaporates than enters the ground from precipitation. It is this process that generates specific problematic soils occurring at specific locations on the Earth's surface. Arid soils present a number of challenges to engineers due to the highly variable nature of their engineering properties associated with active and ancient geomorphic processes; the considerable diurnal temperature ranges experienced, potentially resulting in accelerated disintegration of soil and rock formations as well as man-made structures; lithification processes can result in cementation of the soil particles, making excavation problematic; and various problematic behaviour occurs in arid conditions, such as soils with significant shrink–swell or collapsing potentials, or aggressive salt environments. Key to understanding how arid soils behave and how these soils can be engineered is achieved through an appreciation of the geomorphology of their formation. Associated with these processes are soils that can be cemented or uncemented, whilst being unsaturated. Once geomorphologic effects are understood, appropriate engineering assessment and mitigation can be applied. A number of other logistical challenges also occur with arid environments owing to the limited availability of suitable water sources for construction; only through careful planning can these problems be avoided.

29.1 Introduction

Arid soils can be described as those soils that are conditioned by an arid climate; such climates occur when the potential annual evaporation exceeds annual precipitation and a soil moisture deficit is the result (Atkinson, 1994). Overall, it is estimated that a third of the Earth's land mass currently experiences arid conditions (Warren, 1994a) yet only approximately 25% of the soils found in hyper-arid and arid regions comprises active sand dunes (see **Figure 29.1**) (Warren, 1994b). Arid conditions can be found in both hot and cold locations on the Earth, e.g. arid soils can comprise hot desert soils such as the Sahara, cold desert soils such as the Gobi, even locations within the polar regions are considered arid.

In addition, movements of the tectonic plates result in soils and rock previously formed in arid conditions being found in temperate climates (such as the dune-bedded sandstones, **Figure 29.2**, in the UK's midlands), and soils not commonly associated with arid conditions being found in modern-day arid environments.

Arid soils present a number of challenges to engineers owing to:

- the highly variable nature of their engineering properties associated with active and ancient geomorphic processes;
- the considerable diurnal temperature ranges experienced, potentially resulting in accelerated disintegration of soil and rock formations as well as man-made structures;
- lithification processes, which can result in cementation of the soil particles, making excavation problematic;



Figure 29.1 Sand dune in the Namib Desert, Namibia
Image courtesy of Mrs Jackson-Royal



Figure 29.2 Dune-bedded Permian sandstone, Shropshire, UK
Reproduced from Toghill (2006); Airlife Publishing

- various problematic soils, which occur in arid conditions, such as those with significant shrink–swell or collapse potentials (see Section 3 *Problematic soils and their issues* of this manual for related chapters).

Arid conditions can mean there is a limited availability of water for construction, potentially affecting the suitability of geotechnical processes during construction. This may necessitate the use of locally sourced water, which can contain high concentrations of dissolved salts and will affect construction activities such as the casting of concretes. Limited availability of water can create logistical problems, which must be surmounted if the project is to be completed on time, or it can raise the cost of the project.

The mechanics describing the behaviour of arid soils are not unique to these deposits; soils not currently experiencing arid conditions may still be unsaturated, cemented, prone to shrink-swell or be metastable, although the effects of these mechanisms can be pronounced in arid deposits. However, the climatic and geomorphologic processes that shape arid soils must be considered in order to understand the nature and engineering behaviour of soils in arid regions. This chapter will, therefore, give the key aspects of soil mechanics and geotechnical engineering associated with arid soils along with a description of climatic and geomorphologic processes that shape arid soil conditions (and are likely to continue affecting a site after the completion of a construction project).

29.2 Arid climates

Arid climates form when more water evaporates and transpirates than enters the ground from precipitation. It is this process that generates specific problematic soils occurring at specific locations on the Earth's surface, therefore this section briefly introduces:

- the development of arid conditions in hot and cold regions;
- the characterisation of arid regions into hyper-arid, arid and semi-arid zones.

29.2.1 Development of arid conditions

Arid conditions develop through the global circulation patterns (or macroclimate) of the atmosphere, the location of the land in relation to the sea and the prevailing winds. This is governed by a number of complex processes driven by differences in air temperature and pressure across the globe and the rotation of the Earth. The distribution of arid environments around the world is shown in **Figure 29.3**.

The equatorial regions of the Earth experience the greatest concentration of solar radiation and the poles the least; this imbalance creates a number of convection currents within the macroclimate on either side of the equator (see Wallen and Stockholm, 1966, for further details). The associated climate and atmospheric patterns are the principle reasons why the majority of hot arid conditions are located around the 30° line of latitude on either side of the equator (see **Figure 29.3**).

Cold arid landscapes tend to form because of their location within a land mass in relation to the sea and the prevailing winds (Bell, 1998; Wallen and Stockholm, 1966; **Figure 29.3**); the further from the sea the more likely the water transported in the atmosphere has already precipitated, leaving the air relatively dry (see Lee and Fookes, 2005a, for more details). Topographic structures, such as mountain ranges, can also interrupt the transport of water from sea to land by inducing precipitation

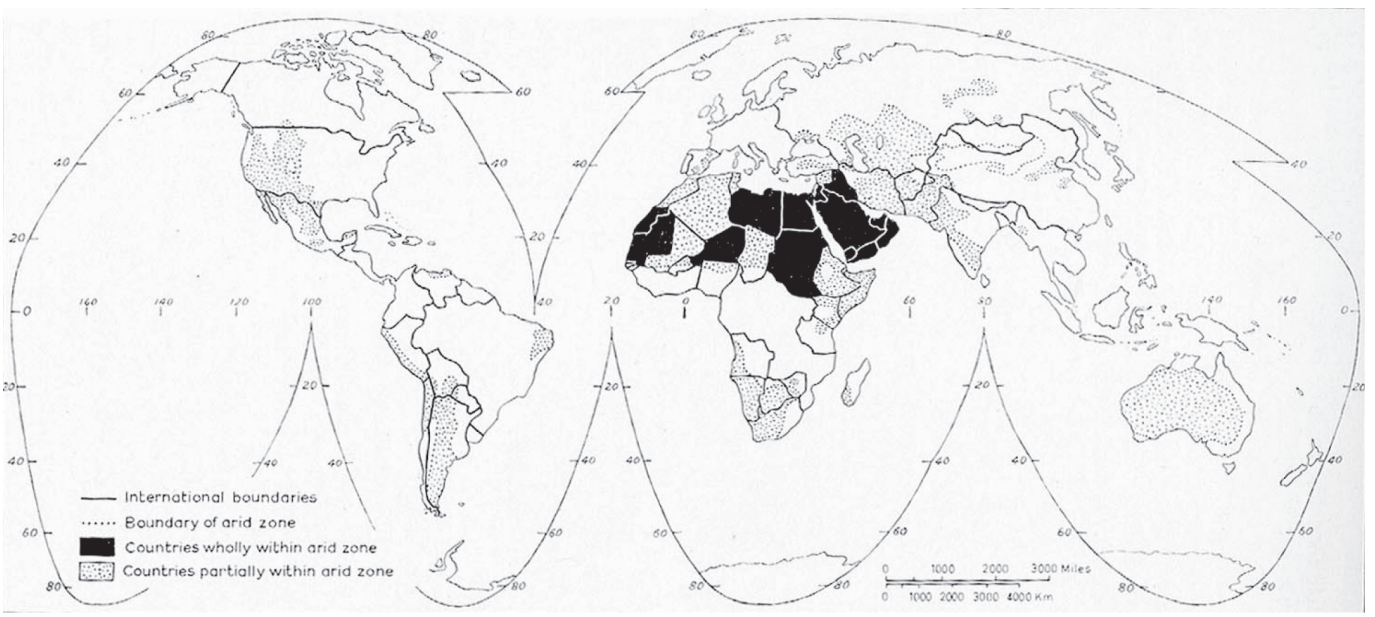


Figure 29.3 The distribution of arid environments
Reproduced from White (1966); Methuen and Co Ltd

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on the topographic structure and leaving the air flowing into the lands behind relatively dry (Bell, 1998; Wallen and Stockholm, 1966).

Cold sea currents that approach the surface and flow near to land (such as the Humboldt Current, situated off Chile's coast) can exacerbate the effect of atmospheric circulation on an arid environment by creating stable and cool air conditions at sea, making precipitation unlikely to occur (although fog and clouds can still form) (Wallen and Stockholm, 1966). The Atacama Desert, Chile, is considered one of the driest places on Earth due to the combined effects of the atmospheric circulation, the presence of a topographical structure along the coastline and the Humboldt Current (Wallen and Stockholm, 1966; Lee and Fookes, 2005a); Lee and Fookes (2005a) give data indicating that the Atacama Desert receives a mean annual precipitation of less than 0.3 mm.

29.2.2 Precipitation in arid environments

Arid environments are not necessarily under constant drought conditions; although there are locations in hyper-arid regions that may be in drought for years, if precipitation occurs at all. However, by definition, arid environments do experience a moisture deficit. Regions can be classified as arid (or subdivided into hyper-arid and arid, depending on the classification system used), sub-humid (where the magnitude of the deficit is low) or semi-arid (transitional tracts of land between arid and sub-humid lands); see **Figure 29.4**.

Lee and Fookes (2005a) give a simple relationship between mean annual precipitation and annual potential evapotranspiration (the combination of water vapour entering the atmosphere via evaporation and transpiration, Fetter, 2001), see **Table 29.1**, to determine the aridity of a region. Calculation of the annual potential evapotranspiration for a region can be determined using a number of approaches, although the Penman equation (not considered here) is considered the most suitable (see Lee and Fookes, 2005b, and Wallen and Stockholm, 1966, for further details). Deriving the annual potential evapotranspiration requires the quantification of a number of environmental parameters and Wallen and Stockholm (1966) give an alternative method (developed by Köppen) that relates aridity to mean annual precipitation and mean annual temperature (for a winter, summer or undefined season); see **Table 29.1**.



Figure 29.4 Semi-arid desert conditions in Botswana during the summer 'wet season'; note the profusion of green vegetation indicating the occurrence of recent rains
Image courtesy of Mrs Jackson-Royal

Arid regions can be described as having wet and dry seasons, although such an approach is more applicable to semi-arid regions than arid or hyper-arid regions. The duration of the wet season can be very short. Predicting actual precipitation patterns in hyper-arid and arid conditions can be very difficult as the annual precipitation can vary considerably, whereas semi-arid regions can experience more predictable conditions (Bell, 1998; Lee and Fookes, 2005b). Taking the mean precipitation over a reasonable time frame will result in the values quoted in the literature, though the annual precipitation will vary considerably throughout this period; some years very little precipitation, if any, will occur whilst in other years far greater amounts will be recorded (Lee and Fookes, 2005b). Lee and Fookes (2005b) give further details of the distribution of arid regions, including indications of aridity, periods of drought and associated temperature and the occurrence of wet and dry seasons.

Precipitation, when it occurs, can take the form of short-lived very heavy storms known as convection-cell cloudbursts. These storms can occur as infrequently as once in a decade or even once in a century in certain arid environments (Hills *et al*., 1966). However, they are very intense events and they can cause significant changes to localised topography and can damage earthworks (as illustrated in the subsequent sections of this chapter).

29.3 Geomorphology of arid soils and the effect of geomorphic processes on the geotechnical properties of arid soils

An overview of the geomorphology of arid soils has been included in this chapter as the geomorphic processes active in arid regions are fundamentally related to the geotechnical properties of the soils found within those areas. Failure to understand the geomorphic processes, which have acted upon, and will continue to act upon, the arid soils found on site, will limit the ability to successfully engineer those soils. Therefore, this section of the chapter considers:

- the commonly encountered types of arid regions and the associated ground conditions;
- erosion and deposition processes within arid regions (alluvial, chemical, fluvial and mechanical) and the potential effect these process have upon construction processes;

Humidity province	Characteristic vegetation[1]	*P*/ETP[2]	Summer[3]	Winter[3]	Undefined season[3]
Sub-humid zone	Grassland	0.50–0.75	-	-	-
Semi-arid zone	Steppe	0.20–0.50	$P \leq 20T$	$P \leq 20(T + 14)$	$P \leq 20(T + 7)$
Arid zone	Desert	0.03–0.20	$P \leq 10T$	$P \leq 10(T + 14)$	$P \leq 10(T + 7)$
Hyper-arid zone	Desert	<0.03	-	-	-

Where P is the annual precipitation (mm), ETP is the annual potential evapotranspiration and T is the annual mean temperature (°C)
1. Lee and Fookes (2005a)
2. Lee and Fookes (2005b)
3. Hills *et al.* (1966)

Table 29.1 Methods to determine if arid conditions apply for a given region
Data taken from Lee and Fookes (2005a, b) and Hills *et al.* (1966)

- landforms that are found in arid regions and their potential for use as aggregates;
- the formation of duricrusts and cemented soils and the problems these present to construction;
- the formation of hammada, dunes and other aeolian features, the conditions associated with these landforms and the geohazards these landforms present when undertaking construction;
- the formation of fans, wadis and other fluvial landforms, the conditions associated with these landforms and the geohazards these landforms present when undertaking construction;
- the formation of salinas, sabkhas and playas, the conditions associated with these landforms and the geohazards these landforms present when undertaking construction;
- considerations when undertaking site investigations in arid environments and potential sources of aggregates.

The geomorphic processes affecting arid environments are similar to those affecting other regions; arid environments are not governed by special circumstances (Hills *et al.*, 1966). However, the lack of vegetation and the aridity of the ground conditions makes aeolian erosion more prominent in arid regions than in other regions: aeolian deposits can be prone to collapse (as described in detail in Section 3 *Problematic soils and their issues* of this manual), affecting the design of structures built in such conditions. Stabilisation may be necessary before constructing on these deposits. Aeolian deposits can also be extremely mobile, potentially swamping a site and making it unfit for purpose. Arid conditions can also experience extreme changes in diurnal temperature, resulting in both increased rates of mechanical and chemical weathering of soils and man-made structures, such as concrete, and can significantly reduce the engineering life of a structure. Certain geomorphic processes will only occur infrequently, such as fluvial erosion, yet can have a pronounced effect upon conditions on site.

29.3.1 Landforms within arid environments

There are two distinct types of arid region: shields and basins (or plains) (Lee and Fookes, 2005b), with four distinct zones (from a geomorphologic viewpoint) that control the topography within an arid region: uplands, foot-slopes, plains and base level plains (Lee and Fookes 2005b). Each of these zones exhibits different geotechnical problems as a direct result of their geomorphologic environments. Shields are defined as a 'major structural unit of the Earth's crust, consisting of large mass of Precambrian rocks, both metamorphic and igneous, which have remained unaffected by later mountain building' (Whitten and Brooks, 1973, p. 411). Such features are remarkably stable geological formations having survived intact since the Precambrian eon (Precambrian rocks were formed at least 570 million years ago and can be billions of years old). Therefore, many shield deserts do not contain active volcanoes (although there are examples of recently active volcanoes in arid regions of America; Hills *et al.*, 1966) and many of the topographic features within a shield that were created by tectonic actions in a previous age have been affected by hundreds of millions of years of geomorphic processes (i.e. mountain ranges become reduced to hills by weathering and erosion). A shield may be overlain by younger rock formations, or soil, that will erode preferentially to the shield. Conversely a basin (defined as a 'depression of large size, may be of structural or erosional origin'; Whitten and Brooks, 1973, p. 49) will act as a sink for debris eroded from higher ground via fluvial and aeolian actions and will contain soils with a considerable range of particle sizes. A basin may have dunes.

29.3.1.1 Uplands and foot-slopes

Uplands are regions of hills, plateaus and canyons, characterised as having flat tops with scarps and ramparts (**Figure 29.5**). The topography of upland regions (with the exception of the ramparts) tends to be denuded of soil deposits by fluvial and aeolian processes leaving barren rock or boulder fields (Lee and Fookes, 2005b). The topography of upland regions is shaped by mechanical, chemical and thermal weathering and eroded by fluvial and aeolian action (and is also weathered by these processes), with weaker rock formations eroding preferentially to harder ones. This creates the prominent flat-topped, steeply sided features synonymous with arid environments (Monument Valley in the USA being a prime example).

Foot-slopes (**Figure 29.6**) are transitional regions between uplands and a basin. Gently dipping plateaus that form between

two regions are known as piedmonts. Eroded materials from uplands are deposited, by fluvial and aeolian actions, within a piedmont to form transitionary topographic structures such as alluvial fans and pediment slopes, which can exhibit a significant potential for collapse.

The same geomorphic processes that created deposits on foot-slopes will also erode them, transporting the eroded material into a basin. Whilst chemical, mechanical and thermal weathering occurs in these regions, aeolian and fluvial processes dominate the shaping of the topography in foot-slope environments. The actual geomorphologic processes involved are not universally agreed. However, from a geotechnical engineering viewpoint, piedmonts can be valuable sources of readily available aggregates and other resources for construction projects (Lee and Fookes, 2005b), as illustrated in section 29.3.7.5.

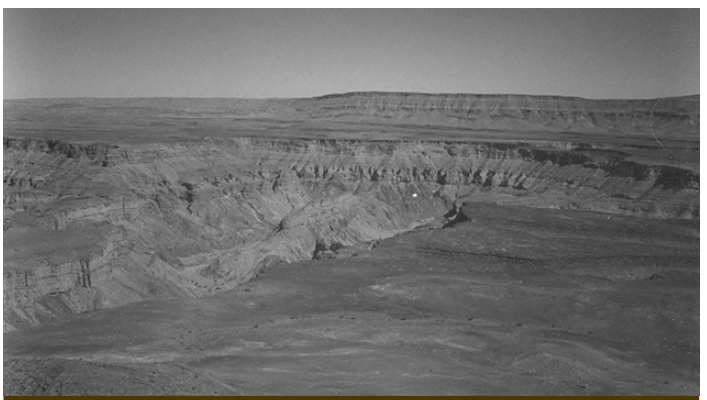


Figure 29.5 Fish River Canyon, Namibia; note the flat and barren tops, steep scarps, establishment of ramparts at the base of the scarps and the development of a mesa and a butte (centre) within the canyon
Image courtesy of Mrs Jackson-Royal

29.3.1.2 Plains and base level plains

The topography of plains tends to be dominated by the deposition and continued transport of previously weathered and eroded materials from uplands and foot-slopes. Aeolian and fluvial transport mechanisms dominate the changing topography of these regions, creating dunes and wadis, for example, and segregating the transported particles with respect to size and density depending on the location of the source material and the orientation of weathering process. Base level plains are naturally occurring depressions within a basin and they are also affected by aeolian and fluvial actions (akin to plains), although the groundwater level can be relatively near the surface. The proximity of groundwater, the moisture deficit experienced by the soils in the vadose zone and the chemistry of the pore water can result in an accumulation of salts within the vadose zone. These highly salty conditions can accelerate the weathering of soil and rock deposits within base level plains and can cause damage to man-made structures.

Whilst upland features such as scarps and inselbergs (Ayers Rock being a prime example, Hills *et al*., 1966) can be ancient, features such as pediment slopes, fans and dunes (**Figure 29.6**) are thought to be products of the Quaternary and are likely to be reshaped by future geomorphic processes (Lee and Fookes, 2005b). The lack of vegetation and predominantly dry surface soil result in coarse-grained soils being readily eroded by

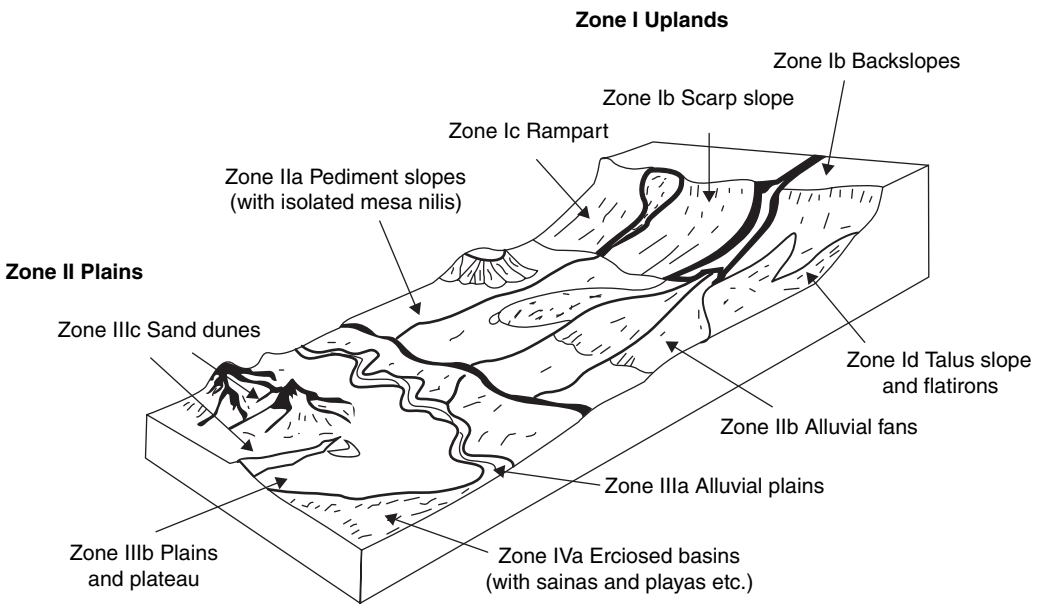


Figure 29.6 Illustration of hot desert topography
Reproduced from Lee and Fookes (2005b) Whittles Publishing

geomorphic processes; phreatic surfaces can be found in arid environments although the depth of the water table can vary considerably and can be very deep; Lee and Fookes (2005b) cite an example of a phreatic surface being 50 m below the ground surface in the Sahara, and this is echoed by Blight (1994) who cites similar groundwater levels in semi-arid conditions in southern Africa. The geomorphic processes that dominate arid environments include aeolian and fluvial erosion as well as chemical, mechanical and thermal weathering.

The transitional, and desiccated, nature of superficial deposits within arid landscapes, along with the bedrock being very close to the surface in upland areas, can prove to be a significant resource of aggregates for construction purposes, as illustrated in section 29.3.7.5.

29.3.2 Development of arid soils through weathering

Weathering by aeolian, chemical, fluvial, mechanical and thermal processes occurs in arid environments. These processes produce a complex array of landforms and associated ground conditions, many of which present a number of significant challenges to geotechnical engineers. An understanding of this behaviour is essential. Further details and other useful references can be found in Fookes *et al*. (2005b).

29.3.2.1 Mechanical weathering processes in arid environments

The process of eroding an upland is relatively slow; Lee and Fookes (2005b) cite rates for a scarp to recede as being of the order of millimetres per year. The resilience of upland formations is related to the nature of the upper layers of the topography, with rocks such as sandstone, or cemented soils (known as a duricrust), forming barriers that resist the erosive processes. These layers are called capping layers and they can slowly be eroded at the scarp by mechanical weathering. If the formations of uplands are jointed then a capping layer may be undermined by block-by-block failure (Lee and Fookes, 2005b; Howard and Selby, 2009); if the formations below the capping layer are susceptible to erosion by water (if they contain smectites, gypsum, etc.) then these formations can soften when inundated by water, resulting in the collapse of the capping layer blocks above (Howard and Selby, 2009). The capping layer may also be weathered if the rock underlying the capping layer is permeable. Water seeping through this rock may weaken it; eventually voids will form within the underlying rock and ultimately lead to collapse of the weakened capping layer above (Lee and Fookes, 2005b).

Mechanically weathered material will fall from the scarp and accumulate on the ramparts where it continues to weather, forming a debris slope (known as a talus slope). Talus (commonly known as scree) will form at or near the angle of repose; it can form metastable collapsible materials, and in response to changes in environmental conditions can collapse down the slope. Thus, talus slopes are dangerous places for construction and may require treatment to prevent slips causing disruption. Degradation of a scarp can result in a gradual reduction in the scarp angle (and height), producing smoother slopes, with the upland eventually being reduced to piedmont slopes (see Lee and Fookes, 2005b, for details).

Cyclic changes in climate experienced by the uplands, alternating from wet to dry, can result in the development of flatirons (wedge-shaped features, which can be of considerable size, that have detached from the rampart and slipped down the talus slope; for more information see Howard and Selby, 2009) within a talus slope. Repeated wedge failure, instigated by weakening of internal boundaries (joints, slip surfaces, dipping rock boundaries, etc.), with a change in climate (from dry to wet), may result in rapid degradation of a scarp and result in a number of flatirons being created on the ramparts.

29.3.2.2 Acceleration of mechanical weathering processes with changes in temperature

The mechanical weathering of arid landscapes, and man-made structures within these arid environments, can be accelerated by changes in temperature. Arid environments experience considerable changes in temperature throughout the year and more importantly diurnally, for example when working on a site in a hot arid region during the summer, the daily temperature would change from low twenties (°C), or less, in the early morning to the high forties (°C), or greater, in a matter of hours; therefore, the acceleration in weathering can be considerable. Rapid changes in temperature can result in the degradation of man-made structures; this is particularly true for composite structures containing materials with different thermal expansion coefficients (such as reinforced concrete), as differential expansion and contraction induce stresses within the material. The material need not be composite to experience such weathering, differential heating and cooling rates (i.e. the outer face compared to the core) can equally result in increased rates of weathering; the outer layers of the material can spall via a process known as exfoliation. Hills *et al*. (1966) suggest that this method of weathering can be very slow and that unconfined rocks, free to expand and contract, do not readily fail via exfoliation. However, rocks, or structures, that are partially buried in a different medium will experience differential rates of change in temperature (the surface exposed to the ambient temperature will expand or contract at a greater rate than the buried surface), the differential movements within the rock will result in additional stresses developing and lead to accelerated rates of weathering.

29.3.3 Degradation of arid environments and man-made structures through chemical weathering

Diurnal changes in temperature can cause chemical weathering of rocks, and man-made structures, via the precipitation of salts within the joints or pores of the rock or soil, or by chemical attack, as illustrated in **Figure 29.7**.

Miscible salts are often found naturally occurring in soils and sedimentary rocks, either as crystals or dissolved salts within the pore water. Vertical seepage of groundwater, induced by

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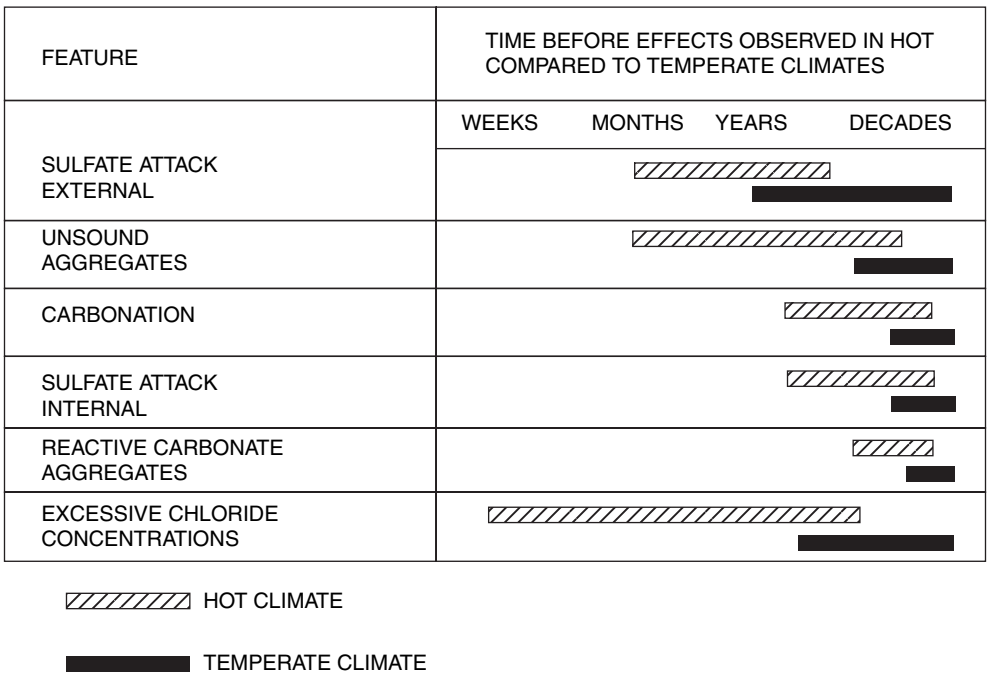


Figure 29.7 Deterioration of concrete due to environmental factors over time
Reproduced from Khan (1982), with permission from Elsevier

suction within the vadose zone, will transport dissolved compounds upwards. If the groundwater table is sufficiently close to the ground level, and suction due to capillarity acts on the groundwater, then the transport of water vertically upwards (see section 29.4.6 for more information) will increase the concentration of soluble compounds within the pores of the near-surface layers of soil.

The concentration at which water can no longer support additional quantities of miscible compounds is known as the saturation concentration. The solubility of a compound is related to a number of parameters including temperature (for more information please refer to Fetter, 1999) (**Figure 29.8**); thus, diurnal changes in temperature can be sufficient to result in a reduction in the saturation concentration and the precipitation of compounds above this threshold, resulting in crystal growth and pressures being exerted on the soil/rock fabric. If the ambient temperature is sufficiently high, evapotranspiration can dry out the pores causing the precipitation of all salts previously dissolved in the pore water. However, Goudie (1994) states that it is not necessary for pores to dry out before the precipitation of miscible compounds is an issue for naturally occurring rock, soil and man-made structures alike. Diurnal changes in temperature will result in cycles of precipitation and dissolution, and this is sufficient to weather the host material. Man-made structures can be particularly at risk with concretes spalling and steel reinforcing experiencing accelerated rates of corrosion. Al-Amoudi *et al*. (1995) suggest there is also the potential for solutes to enter placed fills, such as the layers of a highway, via capillarity and this can result in damage occurring to these structures with time.

Changes in environmental conditions, including a change in temperature, a change in the concentration of dissolved salts, a change in the volume of pore water, can result in the precipitation of compounds out of solution. The volume of a solid form of a salt is likely to be greater than that for the compound when dissolved in water, particularly if the compound hydrates on precipitation, applying a pressure in the pore or fissure and causing degradation of the soil or rock structure.

Bell (1998) gives crystallisation pressures for various salts: gypsum ($CaSO_4.2H_2O$) at 100 MPa, kieserite ($MgSO_4.H_2O$) at 100 MPa and halite (NaCl) at 200 MPa, and cites a study illustrating that the salts Na_2SO_4 and $MgSO_4$ caused a far more pronounced degradation of sandstone cubes that were exposed to solutions containing them, and that a mixture of salts within a solution can be far more destructive than when present individually. The pressures exerted by precipitation of miscible compounds can be considerable and man-made structures are equally at risk of degradation via this process (Goudie, 1994). Thus, detailed investigation is necessary before undertaking construction in highly salty ground conditions to ascertain the likely effect the salts will have on the engineering life of the structure. Ground conditions with high concentrations of salts can cause physical weathering of man-made structures (due to the expansion of precipitated crystals within pore spaces, cracks, etc.) and chemical weathering (such as the accelerated corrosion of steel reinforcing) (Goudie, 1994).

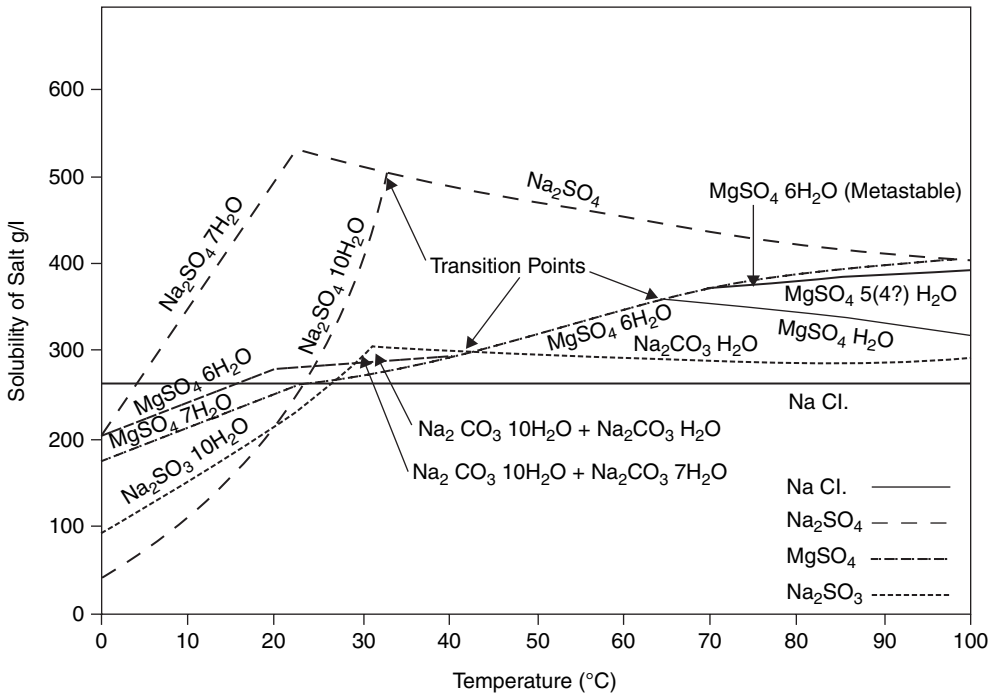


Figure 29.8 Temperature/solubility relationship for some common salts
Reproduced from Obika *et al.* (1989) © The Geological Society

The saturation concentration for a compound is also related to the presence of other compounds dissolved within the pore water. Pore water can only undergo a finite degree of restructuring to accommodate miscible compounds; thus, if more than one compound is present the saturation concentration for each will be reduced and the concentrations of salts precipitated out of solution will be greater than predicted by summing the saturation concentrations for each compound present. This situation is colloquially known as 'salt-out'. Understanding how dissolved compounds will behave within a pore is vital if the design of a proposed foundation is to include protection against this type of weathering. Numerical or analytical modelling, based on the Gibbs free energy, can be used to predict solubility, which can also be modelled within a laboratory using batch tests (Fetter, 1999).

29.3.4 Chemical bonding of soil creating duricrusts and cemented soils

Duricrusts (defined as 'a layer of strongly cemented material occurring in unconsolidated sediments, often found a short distance below the surface; usually formed as a result of water action; the cementing material may be calcareous, siliceous or ferruginous', Whitten and Brooks, 1973, p. 222) are cemented soils, and often form below ground and are exposed by erosion. Duricrusts are commonly a few hundred millimetres thick (300–500 mm), although the development of multiple crusts can result in significant duricrust thicknesses (3–5 m) (Lee and Fookes, 2005b), and are common in arid conditions (and other environments); for example calcrete duricrusts are estimated to cover 13% of the Earth's land mass (Dixon and McLaren, 2009).

The cementation process stabilises the soil and makes duricrusts resilient to weathering and provides strengths greater than those associated with unaffected soils. The types of duricrust commonly encountered in arid conditions include: aluminous, calcareous, gypseous and siliceous duricrusts; siliceous and aluminous duricrusts are considered to be stronger than calcareous duricrusts (Hills *et al.*, 1966; Dixon and McLaren, 2009) although any of the duricrust formations found in arid conditions can prove to be very difficult to excavate. Excavation of a duricrust may necessitate drill and blast techniques or powerful plant (Lee and Fookes, 2005b). Once through the duricrust, the subsequent strata may require less effort to excavate. Therefore, not only do duricrusts create rock structures resilient to erosion, they can also create problematic conditions for construction and must not be underestimated when planning.

The formation of lightly cemented soil also occurs in arid environments, whilst not producing materials as resilient to erosion as the heavily cemented duricrusts, there may be an increase in stiffness and they may be self-supporting when excavated (the sides of an excavation in cemented, desiccated sands may not collapse with time if the degree of cementation is sufficient to resist movement). The soils may be cemented by precipitation of salts via the same mechanisms creating duricrusts (Dixon and McLaren, 2009) or they may be cemented via clay bridging, although soils cemented by clay structures are considered weaker than those chemically stabilised.

29.3.5 Fluvial erosion, the effect of flooding and the creation of challenging engineering environments including salinas, playas and sabkhas

The majority of watercourses found in arid regions are ephemeral: they only contain water directly after a storm, and the water rapidly evaporates or permeates the soil leaving the watercourse dry once more. The drainage path of the water from a storm will dictate how it interacts with the surrounding environment (**Figure 29.9**); flood water will readily migrate across rocky piedmont slopes to wadi drainage channels, forming ephemeral streams; conversely flood water entering tracts of loose sandy soil will rapidly diminish due to infiltration of the water into the soil and ephemeral streams are unlikely to form.

Lee and Fookes (2005b) cite a study indicating that 90% of the erosion on a site over a ten-year period occurred over five days involving seven storms. This is because the intensity of convective cell storms, which take place over a relatively short time frame, results in large volumes of water migrating through previously established drainage networks (or creates new networks when draining). The velocity of this water is sufficient to transport significant quantities of sediment as it migrates through the drainage pathways, creating new topographic features as the sediment is deposited (for more information, refer to Parsons and Abrahams, 2009). The infrequent storms encountered in arid conditions can also damage earthworks (particularly during construction when the earthworks and their drainage provision are incomplete); discharging water can erode the surface of an earthwork; and water permeating into an earthwork can reduce suction within the soil, thus, reducing the stiffness of the soil and inducing ground movement. The former can result in damage to the earthwork, necessitating remedial measures to reinstate the earthwork, and any ground movement can cause serviceability issues or the failure of structures on the earthwork, again necessitating remedial action. Puttock *et al.* (2011) describe a construction project in Morocco that experienced storms during the construction of an earthwork. **Figure 29.10** illustrates a drainage channel neighbouring the construction site in flood, this occurred after a storm, and jeopardised a section of the newly compacted earthwork.

There is often a lack of flood risk assessment for sites within arid environments and such an assessment should have been included during the planning stages for this project, as a matter of course, if potential flood risks were to be properly managed.

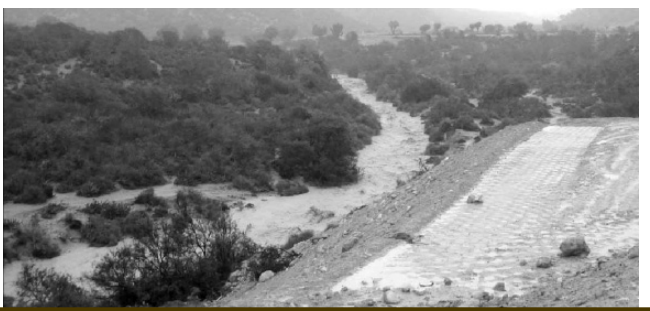


Figure 29.10 Ephemeral drainage channel in flood after a storm; note the proximity of the drainage channel to the newly placed earthwork places the earthwork at risk

Reproduced from Truslove (2010); all rights reserved

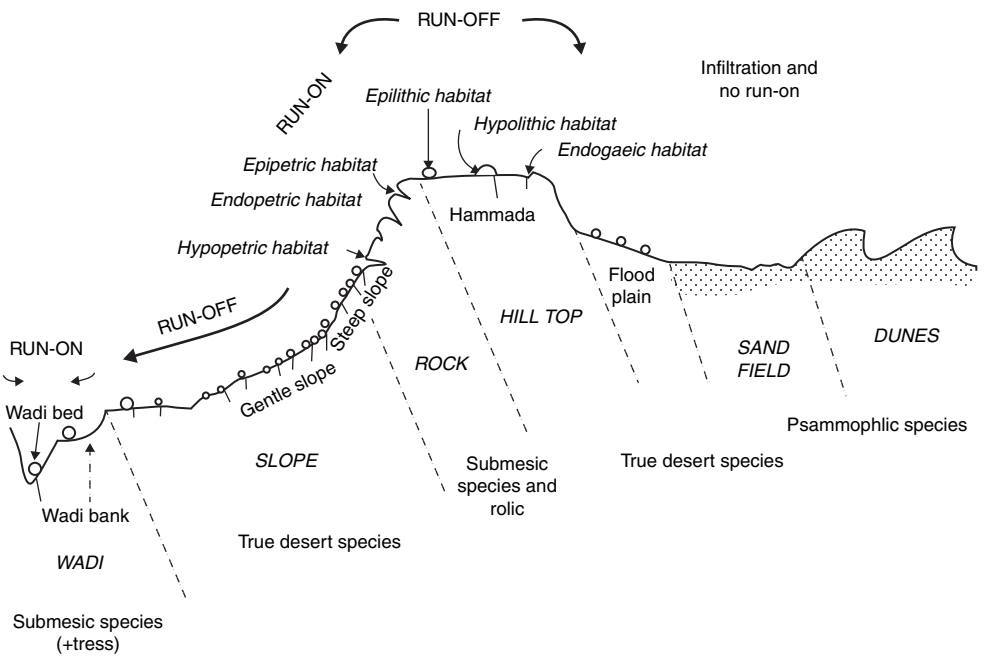


Figure 29.9 Elements of the hydrological cycle in arid lands

Thornes (2009), reproduced from Shmida *et al.* (1986)

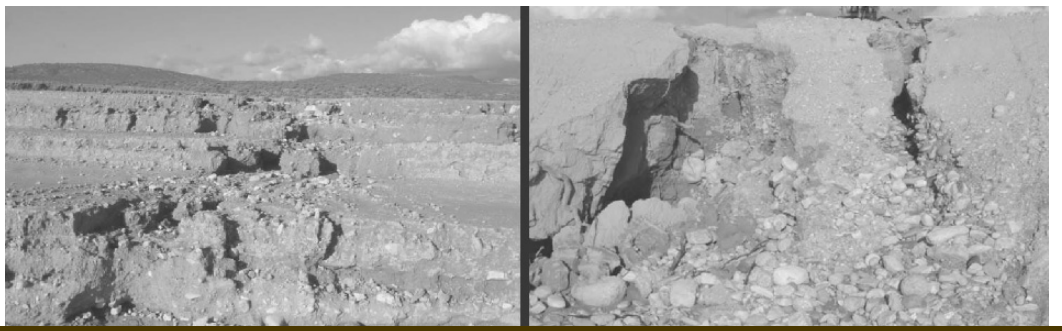


Figure 29.11 Damage to compacted earthwork
(left) Reproduced from Truslove (2010); (right) reproduced from Puttock *et al*. (2011). All rights reserved

Whilst the occurrence of a flood might be significantly lower in arid environments, the potential damage arising from such an event could still be significant. This requires suitable drainage provision for the placed earthwork. If storm water collects on the surface of an earthwork until it reaches a drainage pathway and then dissipates, damage to the earthwork will occur (**Figure 29.11**), requiring remedial measures to reinstate it to a functional condition.

29.3.5.1 Sediment transport and deposition

When precipitation falls on uplands, where the upper surfaces are commonly barren or comprise boulder fields, the water will rapidly drain through a discrete network of drainage pathways, via gullies, into the plains below. The limited nature of the drainage channels and gullies eroded into uplands creates locations for the repeated deposition of eroded materials around the base of the uplands. The materials deposited within these regions can vary greatly but can prove an excellent source of aggregates for the construction industry (Lee and Fookes, 2005b).

29.3.5.2 Alluvial fans

Sediment load is a function of velocity and as this decreases the water will start to deposit sediment, resulting in poorly graded fan-shaped deposits. These fan deposits (alluvial fans) are prominent features on piedmont slopes. Repeated storms will create newly deposited alluvial fans, with the flood water flowing through preferential pathways within previously established fans; the flood water will erode sections of a previously created fan, depositing it further away from the uplands and creating distinctive 'lobe' shapes within the fan. Alluvial fans can cover significant tracts of land; Hill *et al*. (1966) reported the extent of an alluvial fan in the Atacama Desert as stretching 60 km from the upland source. Further details can be found in Blair and McPherson (2009).

29.3.5.3 Wadis

Wadis (valleys with intermittent streams) are ephemeral in nature and form braided streams. Whitten and Brooks (1973, p. 62) define these as 'a stream consisting of interwoven channels, constantly shifting through islands of alluvium and sandbanks; the banks are comprised of soft sediments (often self-deposited), which are easily eroded; the stream beds are wider and shallower than where meander occurs, and are more liable to flood, which in itself helps the constant shifting, widening process'. This creates new alluvium deposits along the river valley in what can seem like haphazard locations (when compared to deposition sites of meandering watercourses). Thus, subsequent patchy lavers of poorly graded, loose alluvium deposits (overlying previously deposited alluvium) can be encountered when working in these conditions. Loose deposits can be eroded and redeposited in subsequent storms; thus, the topography of wadis are transitory. These conditions can also be dangerous places to work during the 'rainy' season when flash floods can occur very quickly and with little warning (Waltham, 2009).

29.3.5.4 Sabkhas, salinas and playas

Fine-grained particles are the sediment likely to settle out of suspension last, and so they can be transported considerable distances by flood water. In base level plains the flood water will pond to form ephemeral lakes. As the lakes recede, silts and clays settle out and form a crust (an example of a crust formed as lake water evaporated can be seen in **Figure 29.12**, although this was not formed by receding flood water but due to changes in river pathways that previously fed the lake). These clay or silt crusts will accumulate with subsequent floods, forming additional layers and will thicken the crust (Lee and Fookes, 2005b). These layers may be interspersed (or overlain) by layers of sand, deposited by aeolian action between periods of flooding, or contain evaporites (in the case of sabkhas and salinas). The surfaces of these crusts can be resilient to settlement induced by light loading or inundation of water during flooding, and Al-Amoudi and Abduljawad (1995) suggest that (in the case of sabkha deposits) this is due to the desiccated and cemented nature of the crusts.

If the crusts form on plains adjacent to the sea then sabkhas or salt playa can form (see **Figure 29.13**). Sabkhas experience



Figure 29.12 Nxai Pan in Botswana, one of a series of salt pans created with the drying of a major lake (Makgadikgadi) due to regional changes in river pathways in southern Africa (caused by tectonic uplift)
© A. Royal

coastal flooding and are likely to have a water table close to the surface. The salt concentrations accumulated within these crusts can be extremely high (**Table 29.2**).

Salt playas can also have high salt concentrations (although not the same magnitude as sabkha) due to the precipitation of salt with the silts and clays as the flood water recedes (Lee and Fookes, 2005b). Sabkhas and salt playas may experience a degree of cementation with the precipitation of solutes from the pore water. Such precipitation can also lead to the creation of gypsum, and other evaporite lenses, developing within these landforms (Al-Amoudi and Abduljawad, 1995) – see also Chapters 37 *Sulfate acid soils* and 38 *Soluble ground* of this manual.

However, the soil below the crust can be metastable, it might contain minerals susceptible to dissolution (e.g. gypsum and halite), and it might be very soft, and, hence, potentially susceptible to ground movement once the upper surface of the crust is compromised (Al-Amoudi and Abduljawad, 1995).

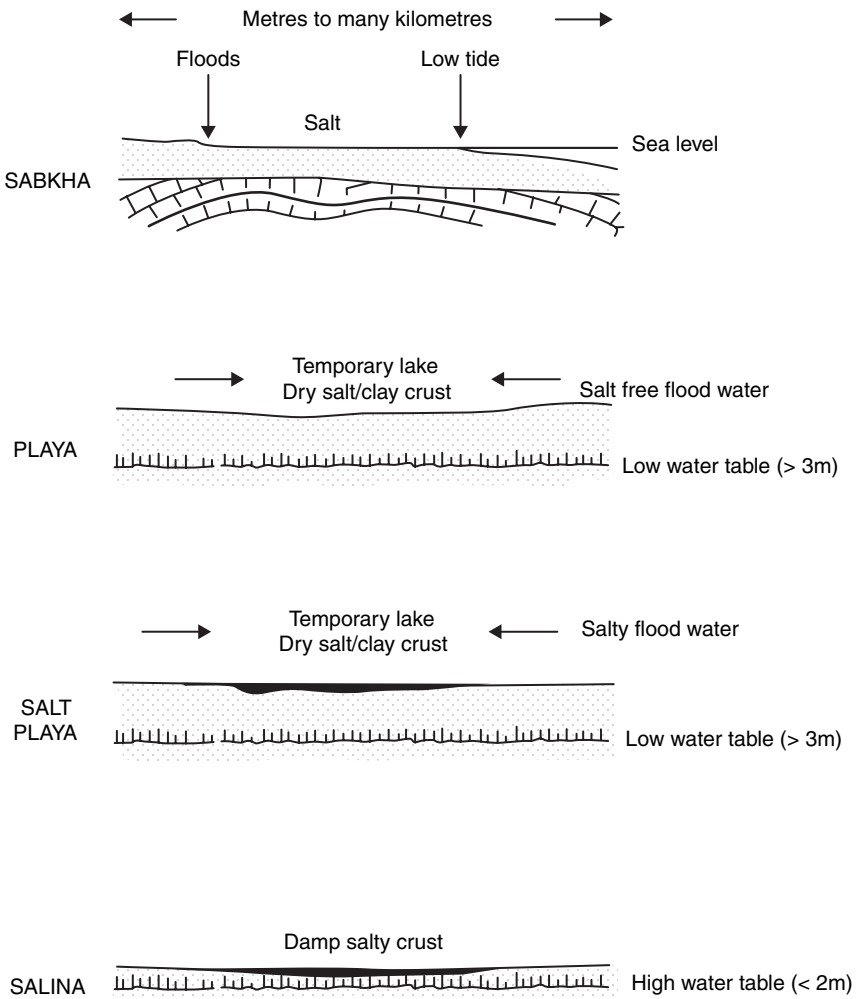


Figure 29.13 Idealised cross-sections through a sabkha, playa, salt playa and salina
Lee and Fookes (2005b), reproduced from Fookes (1976)

Ion (mg/l)	Open sea	Coastal seawater, Arabian Gulf	Sabkha water
Ca	420	420	1250
Mg	1320	1550	4000
Na	10700	20650	30000
K	380	650	1300
SO_4	2700	3300	9950
Cl	19300	35000	56600
HCO_3	75	170	150

Table 29.2 Typical compositions of seawater and sabkha water (in mg/l)
Data taken from Fookes *et al.* (1985)

The properties of sabkha soil can also change with the season; with once trafficable surfaces becoming impassable during the wet season as the cementing salts dissolve (Shehata and Amin, 1997). Sabkhas also formed during the Quaternary (Al-Amoudi and Abduljawad, 1995), leaving deposits along what formally was the coastline. However, the sea level has fluctuated since the Quaternary and many of the sabkhas created via this mechanism have been reworked, by action of the sea, to form carbonate sands and silts as well as bioclastic sands (Al-Amoudi and Abduljawad, 1995).

Care must be taken when constructing on these topographic features; the high concentration of salts within these crusts makes them problematic when coupled with the potential for flooding (Millington, 1994). Construction in such environments requires consideration of the stiffness of the underlying soil (particularly for salinas), as the density of these soils can be low (Al-Amoundi, 1994). Other issues include the collapse potential and the highly salty nature of the upper layers of the soil. Buried man-made structures within these landforms, e.g. concrete foundations or steel pipes, can experience accelerated weathering due to the high salt content within the soil (Shehata and Amin, 1997). Using materials from these areas for construction processes is not recommended (see section 29.3.7.5 for further details).

Silt or clay crusts formed by freshwater flooding are known as playas or salinas, with the difference in behaviour being dictated by the proximity of the water table to the ground surface. In base level plains the groundwater table can be relatively close to the surface, and suction within the vadose zone can result in a significant capillary rise, e.g. over 10 m in clays and between 1 to 10 m in silts (Bell 1998). If the playa is susceptible to the movement of water via capillarity then it is called a salina and salts are transported towards the surface (Lee and Fookes, 2005b). The increase in salt concentration within the salina, akin to sabkha and salt playa, can also make construction difficult. If the movement of groundwater intersects with the ground surface then the surface layers of the salina can be damp (**Figure 29.13**), with soft material directly below.

29.3.6 Aeolian erosion and the creation of challenging engineering environments including collapsible soils, hammadas and sand dunes

Arid soils are more susceptible to aeolian erosion than soils found in other regions due to climatic conditions and the resulting lack of vegetation. Aeolian erosion has two principal forms: firstly, the wind can transport soil particles from one location to another; the ability of the wind to transport soil particles is related to a number of factors including wind speed and particle type (e.g. size, shape, specific gravity), and secondly, soil transported by the wind can be used to scour existing topographic features (akin to the sea using transported sediment to erode the base of cliff faces). The scouring power of wind should not be underestimated and has negative connotations when working on site (for more information, see Parsons and Abrahams, 2009).

The mode of aeolian transport is dependent upon the size, shape and density of the soil particles (for given wind and topographic conditions). Very fine materials (less than 20 μm) will stay in suspension for a long time, their transport can last for several days and particles can be transported thousands of kilometres (Nickling and McKenna Neuman, 2009). Larger fine-grained materials (less than 70 μm but greater than 20 μm) are too heavy to stay in suspension for a long time, transport will last a few minutes to hours, and the particles can be transported tens of kilometres (Nickling and McKenna Neuman, 2009). Granular material is too heavy to transport via suspension and instead can be transported by saltation or rolling. Saltation describes the process where particles (up to 500 μm) move through the air in a cycle of collection, transportation and deposition via short elliptical paths. Larger particles will roll along the surface of the ground under the force of the wind; however, there is a point where the particles become too large to be transported by aeolian action.

The segregation of particles by size, shape and density, due to aeolian erosion, will result in a grading of particles within a basin relative to the dominant wind direction and source of material. The loss of transportable particles via aeolian action will result in a gradual lowering of the ground surface and at the same time an accumulation of particles too large to transport via aeolian action. Eventually the particles too large to be transported by the wind will cover the ground surface and prevent further erosion through aeolian action. The ground surface is then known as a hammada (see Stalker, 1999, for further details). These are resilient surfaces that can be beneficial to construction; the surface can be trafficable without the need for much preceding ground improvement, but once the surface is broken the affected area will once again be susceptible to aeolian erosion.

The movement of sand-sized particles – not necessarily quartz: the particles can be clay aggregations, salt crystals, non-quartz particles (Warren, 1994b) – via aeolian action can lead to the creation of dunes at the windward or leeward face of a topographic feature, whether naturally occurring or man-made, or where two or more winds converge (Lee and Fookes, 2005b). The type of dune formed (see Stalker, 1999, for details

of common dune types) is dependent upon a number of factors including the volume of sand transported and the consistency of the direction of the wind, with wind consistency being considered of primary importance (Lancaster, 2009). The particle diameter size range commonly found in a dune is 0.1–0.7 mm, with median grain diameters between 0.2 mm and 0.4 mm. A dune is considered to be poorly graded (Lee and Fookes, 2005b).

The processes that create dunes can cause them to migrate across the surface of an arid region, and they are then known as active dunes. Sands accumulate on the slip face, which causes the crest of the dune to move with time and, hence, the location of the slip face. The migratory rates of dunes (see Warren, 1994b, for details of the rates of movement with height) are inversely related to the dune height, and are a function of the cross-section and bulk density of the dune as well as the aeolian transport rate of the sand (Lee and Fookes, 2005b). Dunes can form across vast areas (known as dune fields or sand seas; sand seas being far larger than dune fields). For example, it is considered that about four-fifths of arid lands are barren rock or boulder fields (Bell, 1998) and that less than 25% of arid and hyper-arid regions are covered in active dunes (Warren, 1994b), yet Hills *et al*. (1966) cite an uninterrupted sand sea in the Sahara covering a surface area greater than France. In such places the dunes can achieve considerable heights; Lee and Fookes (2005b) describe extreme transverse dunes in the Sahara reaching 240 m.

The transition from arid to semi-arid conditions can result in a stabilisation of the dunes as increased precipitation levels results in increased coverage of vegetation. Dunes can also be stabilised with the cementation of the sand particles, via the development of salt crystals, or the weathering of the non-quartz particles within the dune (Warren, 1994b). Lancaster (2009) gives the geographic locations of dune fields and sand seas across the world and the activity of their dunes.

The aeolian erosion of arid soils is likely to occur during and after construction and this must be taken into account during the planning stage of any construction project. Aeolian deposits can affect structures (blocking doorways and in extreme cases burying houses; see Shehata and Amin, 1997), can affect construction processes (e.g. engine filters readily become blocked with fine materials, prevent setting out using optical devices) and can create logistical problems (e.g. see **Figure 29.14**) where loose material can be problematic to negotiate, particularly large vehicles not equipped with caterpillar tracks, such as articulated lorries. Measures can be established to protect a site against the build-up of wind-transported soils, such as using porous fences as sand traps, illustrated in **Figure 29.15**, although the land required to protect a site can be considerable.

Aeolian erosion denudes landscapes of the finer-grained particles, and creates transitory features such as dunes. However,



Figure 29.14 Car stuck on loose, aeolian deposits covering a road. The road was perfectly trafficable two weeks before
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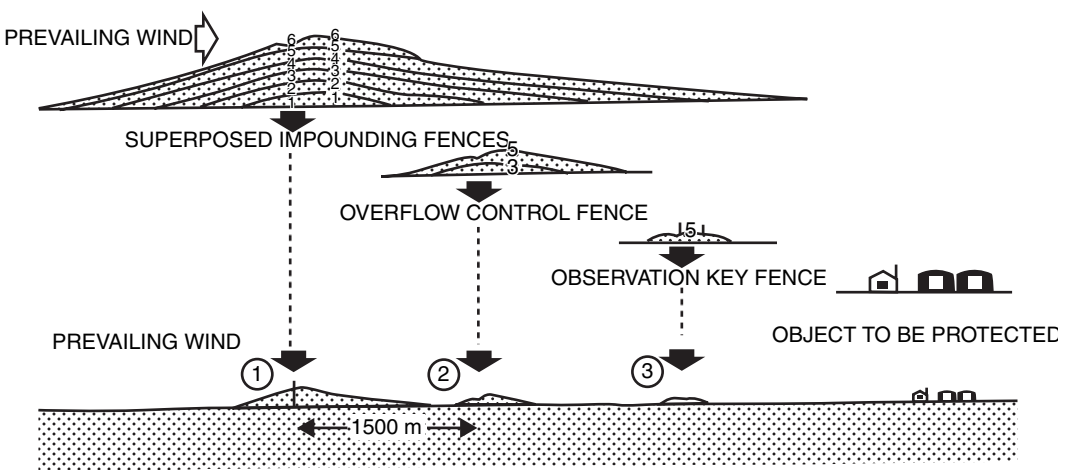


Figure 29.15 The use of porous fences as sand traps
Lee and Fookes (2005b), reproduced from Kerr and Nigra (1952); American Association of Petroleum Geologists

particle-laden winds (transporting particles with sizes in the range 60 μm to 2000 μm) also act as a geomorphologic process on rock formations, scouring them (akin to commercial sand-blasting, if not as intensive) and further shaping the topography of the uplands and foot-slopes; this process is slow and continuous. Laity (2009) quotes abrasion rates of the order of magnitude of 0.01 mm/year to 1.63 mm/year (abrasion can be far greater in places like Antarctica, where wind speeds can be considerable, although this is not considered here). A variety of different rock formations (Bell, 1998) are created depending on the type of rock, rock strength and the alignment of the layers of rock.

29.3.7 Undertaking site investigations in arid areas and classifying arid soils

Soil classification for geotechnical engineering applications should provide sufficient information so that the design and construction phases of a project are cost effective whilst minimising the risks of failure. However, to ignore the very nature of arid soils courts failure of the project (during or after construction) via potentially unexpected means (Smalley *et al.*, 1994). Whilst construction projects often do not have sufficient resources to fully classify the properties of a soil found on site (in terms of mineralogy, formation, etc.), it is recommended that the classification process is extended beyond the scope of a laboratory study to determine the geotechnical parameters of the soil and to consider textural (e.g. fabric and soil structure) aspects of the soil (Rogers *et al.*, 1994).

In arid conditions, soils are often unsaturated, can contain high salt concentrations and, due to the geomorphologic process that created them, can be loose, metastable and, therefore, collapsible, or expansive (see Chapters 32 *Collapsible soils* and 31 *Expansive soils* for further details). **Figure 29.16** can be used to help to identify soils that may experience changes in volume. Laboratory testing to identify physical parameters will not identify the risks posed to the foundation of a structure through chemical or physical weathering induced by high salt concentrations below the ground surface, or failure of the foundation via collapse (or expansion) of the soil skeleton upon wetting. Moreover, a perfectly serviceable structure could become expensive to maintain due to the continued accumulation of windblown deposits.

29.3.7.1 Guidance for site investigation in arid soils

The soil or rock formations encountered will depend on the location (as illustrated in section 29.3.1); soils found on uplands or foot-slopes will vary considerably from those in the plains and this must be considered when developing a site investigation strategy. Any deposits investigated are likely to be heterogeneous in nature due to the geomorphic processes that created them, so engineering parameters derived from a limited number of borehole samples or laboratory-derived data may be unreliable. Furthermore, any deposits encountered are likely to be unsaturated (unless within a base level plain, where the phreatic surface can be relatively close to the surface), could be dry, and are possibly metastable (e.g. talus slopes, aeolian deposits such as loess) making the collection of undisturbed samples effectively impossible. Therefore, any site investigation should be underpinned by a preliminary desk study to highlight the types of soil that may be encountered and identify the types of tests required to characterise the soils on site.

However, care is needed when using standards, e.g. British or European standards, as these may not be best suited to arid soil conditions (for tropical soil see Chapter 30 *Tropical*

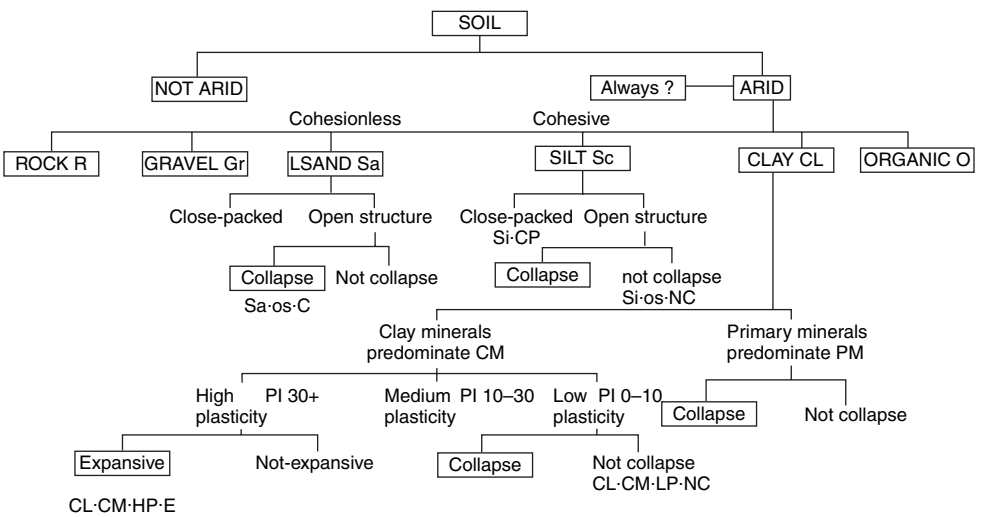


Figure 29.16 Proposed decision tree to indicate volume change problems . Si·CP, silt – close packed; Si·os·NC, silt – open structure – not collapse; Sa·os·C, sand – open structure – collapse; Sa·os·NC, sand – open structure – not collapse; Cl·CM·LP·NC, clay – clay minerals dominate – low plasticity – not collapse; Cl·CM·HP·E, clay – clay minerals dominate – high plasticity – expansive
Reproduced from Rogers *et al.* (1994)

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soils). For example Puttock *et al*. (2011) constructed earthworks in an arid environment and found success when combining French standards (*Technical Guideline on Embankment and Capping Layer Construction*; LCPC and SETRA, 1992), which they suggest are more suited to arid conditions than British standards, with practices taken from British standards (end performance specifications and placed soil investigation techniques).

29.3.7.2 *In situ* testing

In situ testing can determine geotechnical data or classify soil properties and can complement data obtained from laboratory tests (such as characterising and index testing, physical testing: including swelling and the collapse potential of reformed samples). *In situ* testing may disturb the soil but in loose, unsaturated, granular deposits (or those with collapse potential) this is likely to be less than the disturbance from extracting samples for laboratory analysis; British and European standards provide guidance on the suitability of various drilling methods in given soil conditions and describe the quality of the samples likely to be obtained (see section 29.7.2 British Standards). Guidelines for the application of various *in situ* geotechnical tests can be found in for example BS2247 (parts 1 to 12) (British Standards Institution, 2009).

An illustration of potential issues can be found in Livneh *et al*. (1998), who undertook a site investigation for a proposed runway in arid ground conditions containing gypsum. The ground conditions included sandy soils containing clays and gravels with silt and clay lenses and gypsum-rich lenses interspersed between the sandy soils; the authors suggest that the site could be characterised as a sabkha. The gypsum formed through precipitation of soluble minerals with changes in groundwater conditions and concern was raised over the performance of the ground if the gypsum was wetted during loading. Various *in situ* tests were undertaken including dynamic cone penetrometer testing, density determination and loss of weight with wetting of the gypsum-bearing deposits. It was found that low *in situ* density values, coupled with the potential for dissolution of the gypsum, were the parameters that dictated the pavement design for the runway. Livneh *et al*. (1998) carried out the following laboratory tests on samples taken from the site: plasticity index testing, dry density, California bearing ratio (CBR), unconfined compressive strength (UCS) and analysis of pore water chemistry, to supplement the information gathered from *in situ* testing.

Various *in situ* tests to evaluate the collapse and swell potential are discussed in Chapters 32 *Collapsible soils* and 33 *Expansive soils*, respectively, and details of the various control, interpretation and assessment issues are given in these chapters.

29.3.7.3 Laboratory testing

Care must be taken when investigating some arid soils in the laboratory, notably those that are metastable or those that contain high solutes such as sabkhas. Laboratory tests for collapsible soils and expansive soils can be found in Chapters 32 *Collapsible soils* and 33 *Expansive soils*, respectively.

An example of an arid soil that requires careful handling in the laboratory is described in a study by Aiban *et al*. (2006). The sabkha soil investigated was found to exhibit different behaviour depending on the water used in the laboratory tests. For this soil, **Figure 29.17** shows the differences in the particle-size distribution and **Table 29.3** gives the plasticity index for this sabkha soil when prepared with distilled water or sabkha brine.

These differences are believed to be related to the difference in the salt concentration between the soil and the water used; distilled water only contains minute concentrations of the solutes in the soil so a proportion of the crystalline salts cementing the soil would precipitate into solution. This change in the salt concentration within the soil changed the physical properties of the soil. Thus, it is important that any liquid used in a soil investigation is similar to the liquid that will be used on site.

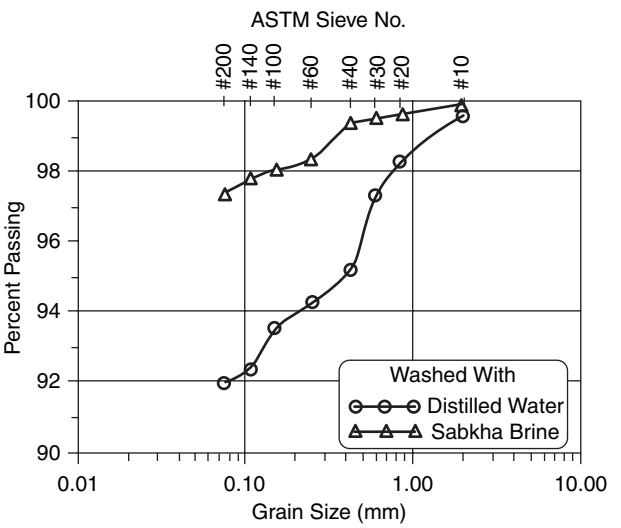


Figure 29.17 Grain-size distributions of a sabkha soil using either distilled water or sabkha brine to wash the soil
Reproduced from Aiban *et al*. (2006), with permission from Elsevier

Property particle-size range (passing ASTM sieve 200)		Value more than 50%
Liquid limit (LL)	Distilled water	42.5
	Sabkha brine	33.0
Plastic limit (PL)	Distilled water	26.3
	Sabkha brine	20.4
Plasticity index (PI)	Distilled water	16.2
	Sabkha brine	12.6

Table 29.3 Difference in plasticity index with the solution used to investigate a sabkha soil (Aiban *et al*., 2006)

29.3.7.4 Geophysical investigation of arid soils

Geophysical testing can be successfully used in arid ground conditions, it can be non-disruptive and it can cover significant tracts of land. Geophysical testing augments information derived from traditional borehole or trial-pit surveys. Guidance on the use of geophysical techniques in site investigations can be found in the *Geological Society's Engineering Geology Special Publication 19* (McDowell *et al*., 2002); although not written specifically for arid ground conditions this guide does introduce the geophysical techniques commonly associated with site investigations.

Shao *et al*. (2009) investigated the use of polarimetric synthetic aperture radar (SAR) to detect subsurface hyper-saline deposits in arid conditions and found promising results. Ray and Murray (1994) report on research into the use of airborne SAR, AVIRIS (Airborne Visible/infra-red Imaging Spectrometer) and thermal imaging to investigate the nature of surface sand deposits in an arid region within the USA and found that mobile sand deposits could be distinguished from fixed deposits using these techniques. The ability to detect mobile deposits stems from the difference in particle-size distribution (active soils tend to comprise a greater proportion of particles in the saltation range and also tend to be poorly graded) and the relative density of active and fixed sand deposits (Ray and Murray, 1994). The ability to detect certain problematic soil deposits geophysically provides an opportunity to optimise any additional investigation and provides the means of identifying geotechnical processes that will have to be undertaken before construction can continue.

29.3.7.5 Material resources for construction projects in arid environments

Various landforms within uplands, foot-slopes and plains (see section 29.3.1) are potentially sources of aggregates. **Table 29.4** indicates potential locations for sourcing aggregates and suggests potential applications within construction processes for these materials. Lee and Fookes (2005b) recommend alluvial deposits (fans and wadi channels) when sourcing aggregates; it is suggested that dune deposits can be used but with caution as the particle grading often encountered in dunes can be too fine to be readily applicable. It is recommended that deposits containing high concentrations of salts should be avoided (or treated to remove the salts). Lee and Fookes (2005b) also suggest that care should be taken when sourcing aggregates

Feature	Aggregate type	Nature of material	Engineering properties of fills	Potential volume
Bedrock mountains	Crushed rock suitable for all types of aggregate	Angular, clean and rough texture	Depends on rock type and processing. Good fills	Very extensive
Duricrust	Road base and sub-base	Often contains salts. Needs crushing and processing	May be self-cementing with time. Quite good rockfill	Often only small deposits of good quality
Upper alluvial fan deposits	Concrete and can be crushed for road base	When crushed and screened angular and clean. Otherwise sometimes dirty and often rounded	Good compaction as fill	Often very extensive but good quality material may be found only in small deposits
Middle alluvial fan deposits	May make road base	Often high fines content	Often good compaction as fill. Good bearing	Small to extensive
Lower alluvial fan deposits	Generally not useful	High fines content. Needs processing	Poor bearing and as fill	Small to extensive
Other piedmont plain alluvium	Variable, locally good concrete aggregate	Dirty, rounded and well graded. May contain salts	Good bearing capacity (dense sediments) and as fill. Locally poor due to clay and silt layers	Very extensive but good quality material in small deposits
Old river deposits	Concrete	Variable	Difficult to locate in field	Often deposits patchy and thin
Dunes	Generally not good	Usually too fine and rounded	Fills of poor compaction	Locally extensive
Interdunes	Fine aggregate	Coarse to fine, angular. Needs processing	Fills of poor compaction	Very localised
Salt playas and sabkhas	Not suitable	Very salty and aggressive	Poor. Special random fill.	Locally extensive
Coastal dunes	Generally not suitable	Generally too fine and rounded	Special fills often poor compaction	Sometimes extensive
Storm beach	Fine aggregate may be sharp or salty	Sufficiently coarse for concrete sand. Clean after processing	Random fill	Sometimes extensive
Foreshore	Generally not suitable	Fine, rounded sand	Salt contaminated, but might make random fill	Locally extensive

Table 29.4 Landforms and aggregate potential
Reproduced from Lee and Fookes (2005b); data taken from Fookes and Higginbottom (1980) and Cooke *et al*. (1982)

as cemented deposits are likely to require processing before being suitable.

The availability of water for construction projects in arid regions must be considered, as this can severely affect the processes being undertaken. Arid regions are often located in developing countries (Wallen and Stockholm, 1966), which have, potentially, limited infrastructure away from urban centres, and the climatic and geomorphologic processes affecting the site must be understood. Thus, water may have to be brought to a site, or groundwater must be extracted (if the phreatic surface is sufficiently close to the surface to make this option economically viable); groundwater can contain significant concentrations of dissolved minerals, which can have a detrimental effect on construction (casting of concrete, etc.) and these factors should be considered by a site investigation.

29.4 Aspects of the geotechnical behaviour of arid soils

When engineering a site in an arid environment it is vital to understand how the aridity will affect the work and what geomorphologic processes will (more than likely) continue to act on the site once construction is complete. This section contains a description of generalised aspects of the geotechnical behaviour of arid soils, and, therefore, considers:

- generalised aspects of the geotechnical behaviour of arid soils;
- various aspects that should be considered when dealing with arid soils including: the generalised behaviour of unsaturated soils; the development of suction within the soil; the relationship between total stress, effective stress, pore water pressure and pore air pressure; and how the shear and volume change behaviour can be evaluated;
- the behaviour of expansive and collapsible soils in arid regions and the effects of cemented soils, together with how such soils can be treated.

29.4.1 Generalised aspects of the geotechnical behaviour of arid soils

Arid soils, by definition, have a moisture deficit, which is likely to result in unsaturated conditions being prevalent (in the surface layers of the soil at least, as is the case in base level plains). This is not to suggest that groundwater cannot exist in arid locations, but it may be at a significant depth below the ground surface. The resulting suction may lead to an increase in effective stress of the unsaturated soil and can induce groundwater flow vertically upwards from the phreatic surface to the vadose zone. Both phenomena can have a marked impact on the behaviour of arid soils.

The suction in an arid soil is a function of the interaction between water and air within the pores. In simple terms, suction forms due to the creation of menisci within the pores because of surface tension between the pore water and pore air (Fleureau and Taibi, 1994) as the pore water preferentially lines the surfaces of particles before filling the pore space. However, in soils containing a distribution of pore sizes, some of the pores may fill completely with fluid (known as bulk water) whilst others remain unsaturated (containing meniscus water), resulting in a heterogeneous distribution of suction within the soil (Wheeler *et al*., 2003). Thus, the effective stress of the unsaturated soil is a function of the suction within the soil (see section 29.4.2), which in turn is a function of the fabric of the soil. Therefore, a change to the structure of the soil (such as could occur when obtaining samples for laboratory investigation), for a constant water content, will result in a different effective stress of the soil (Wheeler *et al*., 2003).

Suction within a soil may be described as matrix suction (a function of the relationship between pore water and pore air pressure) or total suction (which incorporates suction due to osmotic pressure within the soil in addition to matrix suction), although at high suction (1500 kPa and greater) total suction and matrix suction are considered to be the same (Fredlund and Xing, 1994). The relationship between the suction experienced within a soil and the water content is illustrated by a soil–water characterisation curve (Fredlund and Rahardjo, 1993a, 1993b); see **Figure 29.18**. The nature of these curves is determined by soil parameters such as the pore size distribution, the minimum radius of the pores as well as the stress history and the plasticity of the soil (Fredlund and Xing, 1994).

In arid environments, soils can be uncemented (with particles free of any cementing bonding material) or cemented. Various aspects of their behaviour including suction must be considered. These will be considered in turn, highlighting key features pertinent to arid soils. However, only brief details

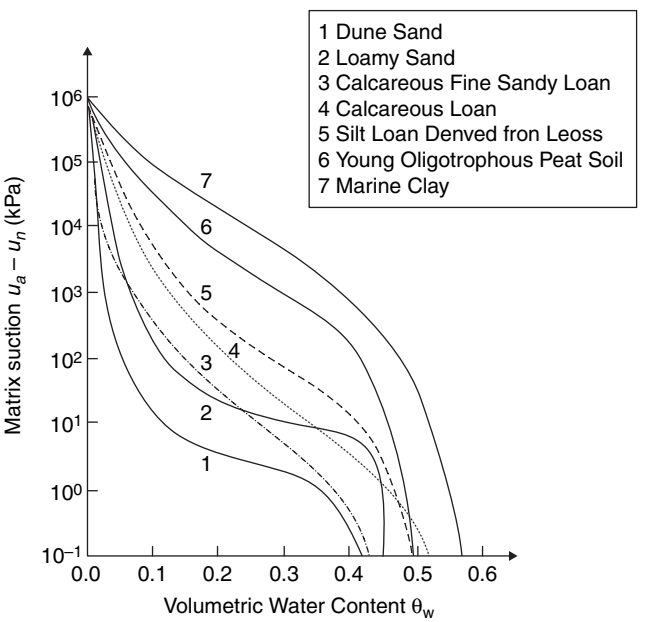


Figure 29.18 Typical soil–water characterisation curves for various soils Mitchell and Soga (2005)
Reproduced from Koorevaar *et al*. (1993), with permission from Elsevier

are provided and further details can be found in Fredlund and Rahardjo (1993a).

29.4.2 Engineering behaviour of unsaturated, uncemented arid soils

If a soil is uncemented and has reached the air entry point, the relationship between stress and pore water pressure must be modified to consider the effect of suction in the soil on the effective stress, and, hence, the shear strength, of the soil. At this point effective stress is a function of total stress, pore water pressure and pore air pressure (μ_a), which control the engineering response of an uncemented arid soil.

The shear strength of uncemented unsaturated arid soils may be determined using Equation (29.1) below (in which stress is σ, net stress is $\sigma - \mu_a$, effective stress is σ', shear strength is τ, angle of friction is φ', matrix suction is $\mu_a - \mu_w$ and cohesion is c'), although this may overestimate the shear strength, particularly at low suction. Instead of combining the net stress and the matrix suction to form the shear strength, which is affected by the angle of shearing resistance, an alternative relationship can be derived if the shear strength is considered to be a function of both the net stress and the matrix suction. This relationship for shear strength is given in Equation (29.2), where a and b are parameters for net stress and matrix suction, respectively. Using two angles of shearing resistance allows for the reduction in the effect of suction on shear strength, given by Equation (29.3), where ϕ^b is the angle of shearing resistance with respect to matrix suction. The angle of shearing resistance with respect to matrix suction is equal to the angle at high suction but can be reduced by an increase in saturation (Rassam and Williams, 1999).

$$\tau = c' + \left(\left(\sigma - \mu_a\right) + \left(\mu_a - \mu_w\right)\right)\tan\phi' \qquad (29.1)$$

$$\tau \propto a\left(\sigma - \mu_a\right) + b\left(\mu_a - \mu_w\right) \qquad (29.2)$$

$$\tau = c' + \left(\sigma - \mu_a\right)\tan\phi' + \left(\mu_a - \mu_w\right)\tan\phi^b . \qquad (29.3)$$

The compressibility and volume change behaviour for uncemented unsaturated soils have been studied using various models (e.g. the Barcelona model) developed by Alonso and Gens (1994). These allow net stress, deviatoric stress and volumetric strains to be assessed together for the soils. This suggests that matrix suction improves the ability of a soil to resist movement, even after the soil experiences plastic deformation. If suction within the soil dissipates then a new stress path will develop within the soil; this stress path results in the soils exceeding the pre-consolidation pressure and the soil will yield without an increase in external load (Alonso and Gens, 1994). The dissipation of suction results in a softening of the soil and an associated deformation.

Whilst an uncemented, unsaturated soil experiences elastic deformation with the dissipation of suction, i.e. the pre-consolidation pressure has not been reached, the soil will expand when water enters its pores (Alonso and Gens, 1994). Once the pre-consolidation pressure is reached the sample will yield and experience plastic (i.e. unrecoverable) deformation and the soil will compress. Even though suction continues to dissipate, the soil will not expand due to restructuring of the soil fabric on yielding (Alonso and Gens, 1994).

Restructuring of the soil fabric at the microscale in response to the dissipation of suction can result in disruption to the soil packing on the macroscale (Alonso and Gens, 1994). Thus, the overall effect may soften the soil and the actual pre-consolidation pressure may be less than predicted from the results of small-scale experiments. Softening may also occur if a soil experiences shearing. As the soil deforms under shear, the menisci within the pores may fail, and until the menisci reform the soil will experience a decrease in suction and as a result it will soften (Mitchell and Soga, 2005).

29.4.3 Relationship between net stress, volumetric strain and suction for collapsible and expansive arid soils

By studying the behaviour of soil samples taken from a site, Blight (1994) was able to describe the relationship between net stress, suction and volumetric strain for expansive and collapsing soils. In this section, drawing from the work of Blight (1994), a brief description of the relationship between net stress, volumetric strain and suction for collapsible and expansive soils is given. More details are given in Chapters 32 *Collapsible soils* and 31 *Expansive soils*, respectively.

An expansive soil with a low load will experience an increase in volumetric strain with the dissipation of suction and the associated increase in water content until the suction has fully dissipated and then the soil will experience additional swelling under saturated conditions (**Figure 29.19**). The dissipation of suction and the increase in volumetric strain may result in the expansive soil becoming unstable and at a critical level of suction the structure will become unable to support itself and will collapse. Once the soil structure has collapsed, additional expansion can occur with a continued reduction in suction. The load experienced by the soil will dictate the degree of volumetric strain experienced by the unsaturated expansive soil. Once suction has dissipated the soil may experience swelling or shrinking with a change in total stress.

Blight (1994) investigated the behaviour of a clayey silt sample using an oedometer at the natural water content of the soil and under saturated conditions. An oedometer was chosen because it allows a load to be applied to the soil sample, which was then inundated with water to simulate conditions likely to occur within the field, thus, allowing the degree of settlement for the soil to be predicted. The saturated soil sample in this investigation initially experienced heave under light consolidating pressure before yielding and suffering plastic deformation (see **Figure 29.20**). The unsaturated sample did not experience heave on loading and yielded at a greater pre-consolidation pressure

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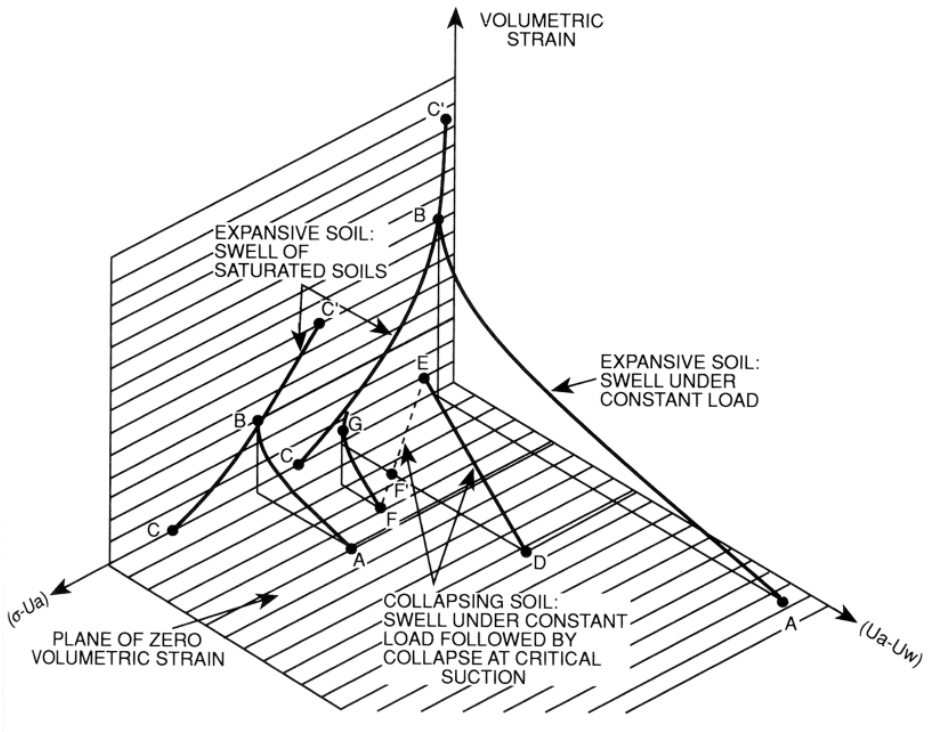


Figure 29.19 Figure 29.19 Three-dimensional stress--strain diagrams for partially saturated soil for the swell of a partially saturated soil under constant isotropic load
Reproduced from Blight (1994)

and, having yielded, was initially stiffer than the saturated sample (**Figure 29.20**). This is not a constant relationship, the soil particles within both the saturated and unsaturated samples will be forced together as the void ratio is reduced with increasing consolidating pressure, resulting in the samples becoming stiffer.

Thus, it is clear that actual deformations experienced by an unsaturated soil on site due to changes in loading can be very difficult to predict and it is recommended that laboratory or field trials are undertaken to determine the behaviour of the soil. Blight (1994) states that arid soils can be particularly problematic, due to the geomorphologic processes that formed them, as they can be prone to both expansion and collapse (echoed by Wheeler, 1994). Thus, some form of experimental investigation is considered fundamental if the behaviour of a soil is to be understood; oedometer testing is a popular laboratory method and plate loading has successfully been used in the field to describe the behaviour of arid soils (as illustrated in section 29.5).

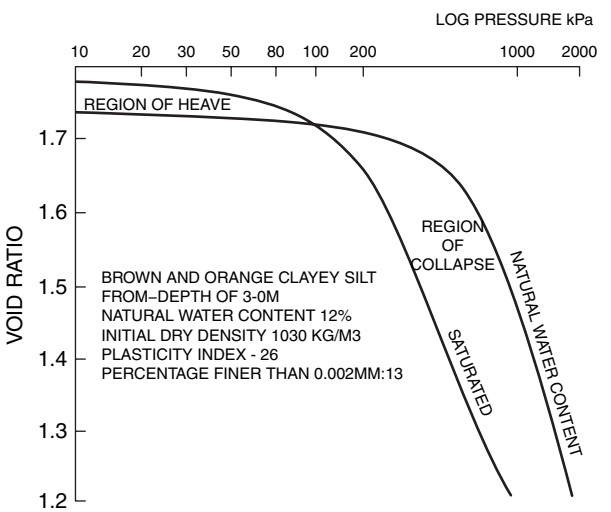


Figure 29.20 Behaviour of clayey silt at natural water content and after saturation
Reproduced from Blight (1994)

29.4.4 Cemented soils in arid conditions

Light cementation of soils (as opposed to the degree of cementation required to create a duricrust) is common in arid environments (see section 29.3.4 above) and will affect how a soil behaves on loading. Cemented soils may have increased tensile strength and may remain stable even after excavation; **Figure 29.21** shows aeolian sand deposits in a semi-arid environment during the wet season. The soil is stable after excavation due to cementation of the soil grains.

Care should be taken when assuming that the walls of an excavation will remain stable due to cementation; localised loading may still cause the sides of an excavation to deform

and ultimately fail. Cementation of the particles will result in an increase in the shear strength, increased elastic deformation during loading (**Figure 29.22**) and an increased yield stress. However, once the cemented soil yields, the residual strength of the uncemented soil and lightly cemented soils (assuming all other parameters are the same) are similar. This is because the crystals bonding the particles within the cemented soil together are relatively brittle and will shear at low strains (Mitchell and Soga, 2005). Once the cementation is disrupted the shear strength reduces and becomes a function of the behaviour of the soil particles, following the behaviour of uncemented soils.



Figure 29.21 Excavation with sides of stable cemented sand (aeolian deposits) during the wet season in semi-arid conditions
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29.4.5 Sabkha soils

Sabkha soils are heterogeneous in nature, may be cemented, may be metastable and may contain high salt concentrations. Much like other metastable deposits, changes in water content can induce collapse in sabkha soils. The dramatic effect of water on sabkha soil was demonstrated by Aiban *et al.* (2006), who showed how the CBR for unsoaked specimens was around 50% at the maximum dry density, compared to around 5% when soaked at the same density. This difference in behaviour is due to dissolution of the cementing salts.

Sabkha soils are considered to be metastable; once the strength of the salts cementing the particles is exceeded (or the influence of the cementing salts is reduced as dissolution occurs with changes in the pore water) then there can be rapid rearrangements of the particles within the soil resulting in compression. This behaviour is not universally encountered in laboratory investigations of the collapse potential of metastable sabkha samples; soils loaded and inundated with water do not always collapse. Al-Amoudi and Abduljawad (1995) suggest that this is a function of the volume of water introduced into the sample, with an equilibrium forming between the concentration of salts in the water and the cemented soil. They used a modified oedometer that applied a constant head of water across the sample and allowed fluid to flow through the soil whilst it experienced loading. The modified oedometer test exhibited collapse compression and lower resulting void ratios under a load, whereas a sample in a traditional oedometer test did not collapse.

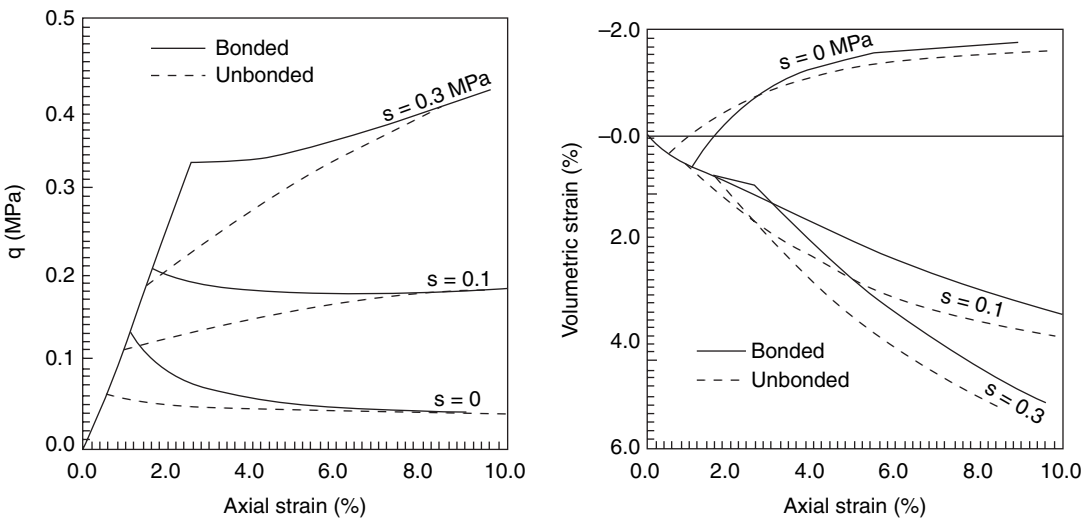


Figure 29.22 Simulated triaxial tests on unsaturated cemented and uncemented soils; (left) shear stress shear strain relationship and (right) volumetric response
Reproduced from Alonso and Gens (1994)

Al-Amoudi and Abduljawad (1995) suggested two simple equations that can be used to predict if the magnitude of the collapse in the oedometer signals problematic soil conditions on site. In Equations (29.4) and (29.5), i_{c1} and i_{c2} are the collapse potentials, Δe_c is the reduction in the void ratio on wetting, e_0 is the initial void ratio and e_1 is the void ratio at the beginning of saturation:

$$i_{c1} = \left(\frac{\Delta e_c}{1+e_1}\right) \times 100 \tag{29.4}$$

$$i_{c2} = \left(\frac{\Delta e_c}{1+e_0}\right) \times 100. \tag{29.5}$$

Equation (29.4) gives the change at the commencement of flooding and Equation (29.5) the collapse potential at the commencement of the consolidation test. Al-Amoudi and Abduljawad (1995) cite findings by Jennings and Knight (1975) that relates the collapse potential to the severity of the problem (see Chapter 32 *Collapsible soils* for further details).

29.4.6 Hydraulic conductivity and groundwater flow in unsaturated soils

Darcy's Law can be applied to unsaturated soils with modification. Mitchell and Soga (2005) give a relationship for the vertical seepage velocity in unsaturated soils, Equation (29.6), where $k(S)$ is the saturation-dependent hydraulic conductivity, ψ is the matrix suction and $\delta z/\delta x$ a gravitational vector measured upwards in the direction of z, for flow in direction i. This equation can be incorporated into a mass conservation equation to give Equation (29.7), which predicts changes in flow for changes in saturation and suction. Here, η is the porosity and $k(\psi)$ is the matrix suction-dependent hydraulic conductivity, derived from $k(S)$ using the soil–water characterisation curve.

$$v_i = -k(s)\left(\frac{\delta\psi}{\delta x_i} + \frac{\delta z}{\delta x_i}\right) \tag{29.6}$$

$$\eta = \left(\frac{\delta S}{\delta\psi}\frac{\delta\psi}{\delta t}\right) = \frac{\delta}{\delta x_i}\left[k(\psi)\left(\frac{\delta\psi}{\delta x_i} + \frac{\delta z}{\delta x_i}\right)\right], \tag{29.7}$$

in which vertical seepage velocity is v_i, matrix suction equivalent head is ψ, saturation is S, time is t, hydraulic conductivity is k, acceleration due to gravity is g.

This solution is valid provided that there are no changes in the soil structure or loss of water from the system (i.e. no loss of water due to biological activity, etc.). The transitional nature of the groundwater flow in unsaturated media, with changing permeability, degree of saturation and suction with flow and time, lends itself to numerical solutions (Mitchell and Soga, 2005).

Measurement of the hydraulic conductivity in unsaturated soils can be problematic due to the dependent relationship between the hydraulic conductivity, saturation and the matrix suction. The soil–water characterisation curve of a soil may be used to derive the hydraulic conductivity; an alternative approach is to use the apparent permeability (see Mitchell and Soga, 2005, for further details). The apparent permeability (k_r) and the saturated hydraulic conductivity (k_s), or the intrinsic permeability (K), may be used to estimate the hydraulic conductivity for an unsaturated soil, Equation (29.8), where ρ_{pf} and η_{pf} are the density and dynamic viscosity of the permeating fluid, respectively. The intrinsic permeability is defined by Equation (29.9), where e is the void ratio, k_0 is a pore shape factor, T is a tortuosity factor and S_0 is the wetted surface area per unit volume of particles.

$$k = k_r K \frac{\rho_{pf} g}{\eta_{pf}} = k_r k_s \tag{29.8}$$

$$K = \frac{1}{k_0 T^2 S_0^2}\left(\frac{e^3}{1+e}\right). \tag{29.9}$$

Lee and Fookes (2005b) suggest that in arid conditions the fines content can reach 30% without having a significant effect on the ability of the soil to drain.

29.5 Engineering in problematic arid soil conditions

The creation of metastable, expansive and chemically aggressive (from the viewpoint of the durability of man-made structures) soil deposits within arid regions results in a number of geohazards, which must be addressed by the geotechnical engineer. Therefore, this section introduces case studies of various ground conditions encountered in arid regions, including:

- foundations in expansive, desiccated or collapsible soils;
- foundations in chemically stabilised soil or rock formations;
- compaction of placed fill;
- stabilisation of sabkha soils.

Chapters 32 *Collapsible soils* and 33 *Expansive soils* describe the behaviour of collapsible soils and expansive soils, with more detailed explanations of structure–ground interactions. This section provides an overview of problems encountered when dealing with soil deposits in arid environments with the aim of highlighting areas that may necessitate the attention of the geotechnical engineer during design and construction.

29.5.1 Geohazards encountered in desiccated, expansive and collapsible soils

29.5.1.1 Examples of foundation behaviour in expansive soils

A detailed review of foundations suitable for arid soils that exhibit expansive behaviour can be found in Chapter 33 *Expansive soils*. That chapter deals with assessment and the foundation or remediation options available to deal with such

soils. The key aspect is the implication of the unsaturated state due to the arid environment, which dictates the nature of the problem and the responses needed.

For example, Kropp (2010) reviewed changes in the foundation design for domestic dwellings on expansive soils in the San Francisco Bay Area. Here, the preference in the local community is for grassed or planted gardens and watering is common. It was suggested that homeowners regularly overwatered the ground, to the point where the mass of water penetrating the ground exceeded that exiting the ground via evapotranspiration, and this change in localised groundwater conditions results in considerable, and potentially differential, ground movement; movement which strip footings were ill-suited to resist.

Nusier and Alawneh (2002) report on the distress caused to a single-storey concrete building with a mat foundation (a reinforced concrete slab) constructed on expansive clay in Jordan. The shallow foundation experienced differential ground movement as the ground under most of the house heaved, yet one corner of the building settled (due to the desiccating action of a tree); the mat was not sufficiently rigid to withstand these movements, the concrete foundation walls were not reinforced and the building suffered significant cracking. It was suggested that the soil under the building heaved due to a thermal gradient existing across the soil below the building and the soil surrounding the building directly exposed to sunlight, resulting in the migration of water from the surrounding area towards the ground below the house. Nusier and Alawneh (2002) concluded that had the foundation been deeper, to a depth where the pressure from the building acting on the soil equalled, or exceeded, that of the pressure from the expanding soil, then the situation could have been avoided. Instead, the building was repaired by installing a reinforced concrete skeleton to the outside of the building and a ring beam to the mat. This intervention cost approximately 25% of the original cost of constructing the building, highlighting the need to understand the probable behaviour of the ground before undertaking construction.

When a desiccated clayey soil experiences infiltration of water during, and directly after, a storm (or because of a leaking utility) the soil fabric will undergo restructuring as it swells. This phenomenon can be pronounced if the soil comprises a significant fraction of clays, or an active clay mineral such as smectite. The seasonal cycle of desiccation/wetting may also cause problems with deep foundations. Blight (1994) reports that pile tests in soil conditions in southern Africa indicate that soil shrinks away from the surface of a pile as it dries in the dry season; clearly this has significant implications when designing a deep foundation if there is a considerable reduction in the skin friction acting on a pile.

Various improvement approaches can be used to treat expansive arid soils (see Chapter 33 *Expansive soils*). For example Rao *et al*. (2001) investigated the performance of lime-stabilised expansive-clay compacted samples exposed to repeated cycles of wetting and drying. Samples were created with lesser and greater amounts of lime than the initial consumption of lime (ICL) value of the soil and were then exposed to several wetting and drying cycles. Both the wetting and drying phases each took 48 hours to complete. It was noted that repeated cycles of wetting and drying had a detrimental effect upon the lime-stabilised soil, in particular for samples with lime contents slightly above or below the ICL. Increasing the number of wetting and drying cycles resulted in:

- more clay detected in the stabilised samples, which Rao *et al*. (2001) suggest is illustrative of the decaying stabilisation of the clay soil;
- a higher liquid limit;
- a lower plastic limit;
- a lower shrinkage limit.

29.5.1.2 An example of foundation behaviour in a collapsible soil

A detailed review of how to deal with foundations in arid soils that exhibit collapsible behaviour can be found in Chapter 32 *Collapsible soils*. That chapter deals with assessment and the foundation or remediation options available to deal with such soils. The key aspect, as with expansive arid soils, is the implication of the unsaturated state due to the arid environment, which dictates the nature of the problem and the responses required.

Loess, a collapsible arid soil, is often considered to be a problematic soil readily collapsing after inundation by water or an increase in load. However, loess often initially experiences a small degree of settlement before experiencing heave, particular at low applied pressures (see section 29.4.3 and **Figure 29.20**). For example, during a plate-loading test, Komornik (1994) applied cycles of loading and unloading to a plate in a loess soil, at the natural water content and after wetting the soil. Wetting resulted in collapse, leading to large unrecoverable deformations. The difference between the behaviour of the soil at the natural water content and the soil after wetting illustrates the necessity of designing a foundation carefully if swelling or collapse potential is considered a possibility, and also the necessity of protecting the location from possible inundation during storms in order to prevent movement of the foundation after completion of construction.

A range of treatment and improvement approaches can be employed to mitigate the behaviour of collapsible arid soils (e.g. dynamic compaction, cement stabilisation) and further discussion can be found in Chapter 32 *Collapsible soils*.

29.5.2 Compaction of fill

In an arid environment, where water is not generally plentiful, applying too little water to the soil (or, in a hot arid environment, applying the desired amount and losing a proportion to evaporation) may result in the development of suction within the soil as it is compacted. The suction will resist the compactive effort and the net effect is a low-density fill, which may experience undesirable levels of settlement. It has been suggested that in

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such circumstances compacting the soil when it is dry might be more appropriate (Newill and O'Connell, 1994). Of course, dry compaction will produce fills with a high volume of air voids and measures must be taken to ensure that subsequent ingress of water is prevented or the fill may undergo restructuring of its fabric, causing the fill to deform or collapse.

In large construction projects, the volume of water required can be significant, which can create logistical problems when working in arid conditions. For example, Newill and O'Connell (1994) investigated the construction of a major road in an arid environment and estimated that the water required to create one kilometre of the road could be as much as 2800 m^3 and equated to approximately 20% of the total construction cost in sourcing and transporting the water to site. This value, by far, exceeds the cost of water for construction in tropical or temperate climates. It must also be considered that arid conditions are often located in developing countries (White, 1966), and the infrastructure can be, at best, extremely limited outside urban areas. Due to the potential lack of infrastructure in rural areas, the source of water may be from boreholes and it is unlikely that any treatment will be applied to the extracted water before it is used for construction. Therefore, the extracted water can potentially contain significant concentrations of salts, salts that will remain within the fill and can have a detrimental effect on the man-made structures placed upon it.

In addition, not all construction projects in developing countries boast the resources necessary to surmount the logistical problems of sourcing sufficient water for a project. For example, whilst working on a site in arid conditions a sister site (approximately 100 km away) faced significant problems when it was announced that the local village borehole was at risk of running dry and a moratorium was placed on the use of water for anything other than critical functions (which did not include construction). In order to continue construction, and ensure that the workforce had water for drinking and washing, 5 m^3 of water was transported by bowser daily. This placed a strain on the efficiencies of both sites and it was difficult to justify the use of water when compacting fills, when other more critical functions (drinking, washing, casting concrete, etc.) took precedence. Eventually it was decided that in order to meet the specifications (end-product specifications were being used) water would have to be used during compaction and the programmes for both sites overran.

When compacting soils in arid environments, particularly for significant geotechnical structures such as roads or embankments, the processes applied are likely to be governed by a framework or standard. Truslove (2010) and Puttock *et al*. (2011), by considering suitable earthwork specifications in arid environments, reviewed a number of standards used in various countries with arid lands. Truslove (2010) concluded that currently there are standards that permit the use of compaction of dry fill in rural areas (such as Kenya), whereas other standards refer to an end-product specification (minimum acceptable dry density, etc.) and do not necessarily comment on the amount of water used. Kropp *et al*. (1994) investigated a compacted fill that had suffered excessive deformation; the soil was a granular material that contained clay and had been compacted to a density greater than the minimum required in the end-product specification, although with a water content less than the optimum (up to 7% less). It was concluded that wetting had induced collapse and that, perhaps, the standard should also impose conditions on the water content of the compacted fill to try to avoid this scenario. Truslove (2010) cites studies that suggest that consideration must be given to the nature of the soil underlying the compacted fill and that care should be taken when selecting the water content of the fill as there is the potential for water to migrate from the fill into the underlying soil, which could induce structural changes within the underlying soil, jeopardising the performance of the fill above.

Sahu (2001) investigated the performance of six regionally sourced soils, from locations in Botswana, when mixed with fly ash with the aim of producing a range of materials that could be used as base courses in highway construction. The six soils ranged from Kalahari sand, calcrete, silts (of various plasticities) and silty sands as well as an expansive clay soil. Various proportions of fly ash were mixed with the soils, and the mixtures were hydrated and left to cure. With the exception of the expansive clay, the inclusion of fly ash lowered the maximum dry densities. However, the inclusion of the fly ash did increase the CBR for the soils containing silts and the calcite; therefore it was suggested that this made these soils suitable for use in highway construction. The inclusion of fly ash in the Kalahari sands and the expansive clays did not have a noticeable impact on the CBR and these were deemed unsuitable for highway construction.

29.5.3 Stabilisation of sabkha soils

Al-Amoudi *et al*. (1995) cites instances where geotechnical improvement techniques such as dynamic compaction, stone columns, densification piles and the creation of drainage have all successfully been used to improve the physical properties of these notoriously weak materials. Al-Amoudi *et al*. (1995) quotes unconfined strengths of around 20 kPa and SPT N ranges of 0 to 8 for these types of formations of sabkha soils, although a value of 30 kPa can be achieved at depth (Shehata and Amin, 1997).

Alternative ground-improvement materials, such as the use of filler particles (limestone dust or marl), to reduce the void ratio of the soil and potentially increase the UCS, have been trialled with sabkha soils (Al-Amoudi *et al*., 1995). The limestone dust (a by-product of an industry local to the study) and marl increased the maximum dry density of the sabkha soil, with the limestone dust reducing the optimum water content and the marl increasing the optimum water content. However, varying the content of limestone dust or marl did not noticeably change the UCS of the sabkha soil, suggesting that the incorporation of these materials can reduce the volume of voids of the compacted sabkha soil but not have a noticeable increase on its strength.

Cement and lime have also been investigated and found to be beneficial as stabilising agents for sabkhas soils (**Figure 29.23**), although there are concerns with the long-term durability of the stabilised materials (Al-Amoudi *et al*., 1995).

Damage to the base and sub-base courses of highways due to movement in the underlying sabkha sub-grades has resulted in investigations into the use of geotextiles between the road foundation and the sub-grade (Abduljawad *et al*., 1994; Aiban *et al*., 2006). Static and dynamic testing of dry and wet sabkha soils, underlying various geotextiles and sub-base materials in a laboratory, illustrated that geotextiles could increase the load required to induce deformation within the soil. **Table 29.5** illustrates the force required to induce a 30 mm deformation at the load plate on the soil for soaked and 'as moulded' conditions with and without a geotextile.

29.6 Concluding comments

Arid soils form in environments where evapotranspiration rates exceed rainfall. Arid soils cover about a third of the Earth's land mass and occur in hot and cold arid regions. However,

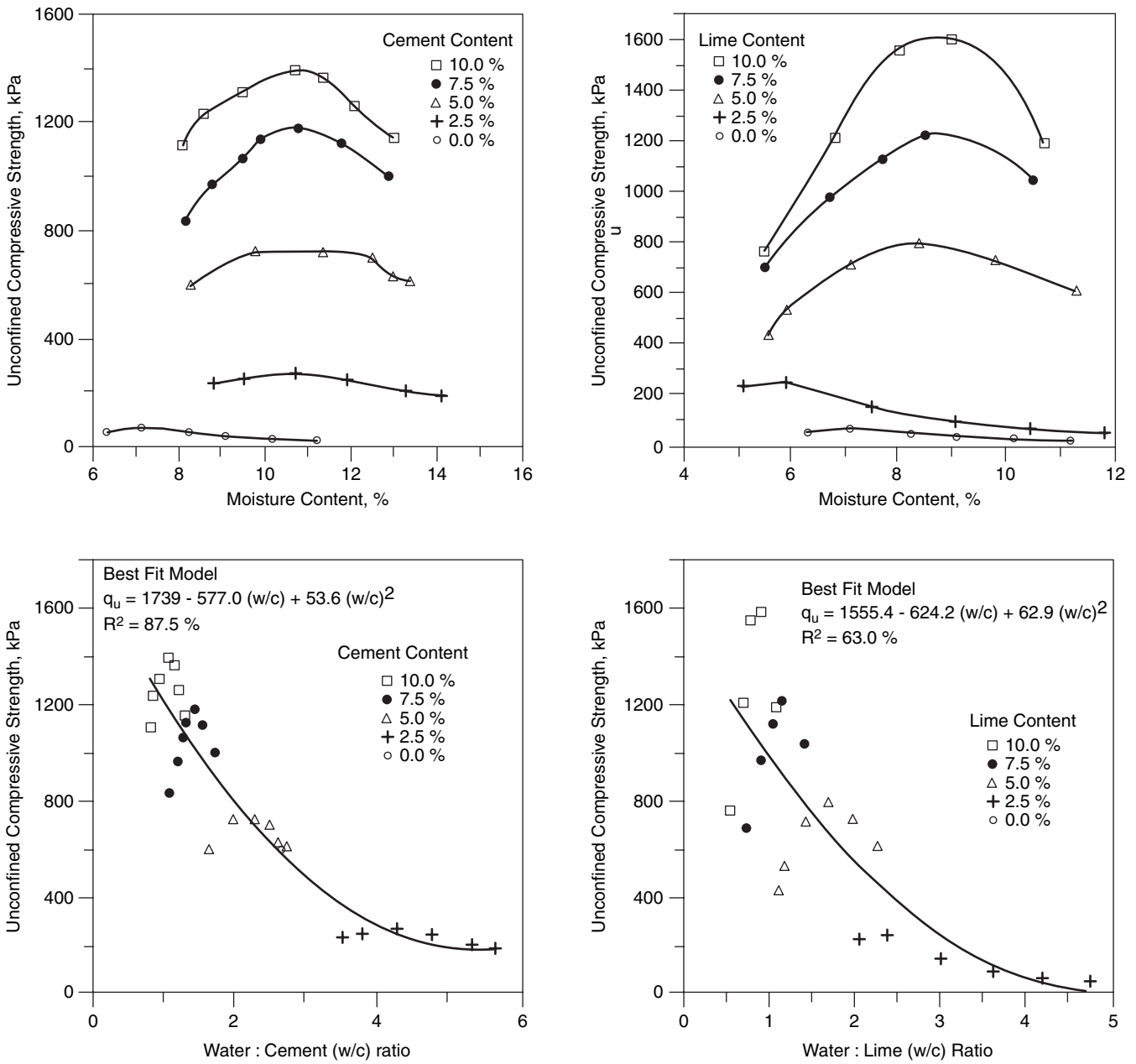


Figure 29.23 Stabilisation of a sabkha soil with cement (top left) and lime (top right). Illustration of the relationship between the UCS and the water/cement ratio (bottom left) or the water/lime ratio (bottom right)

Reproduced from Amoudi *et al.* (1995) © The Geological Society

Sample code	Load at 30 mm deformation (kN)	Test condition
Without geotextile		
SG0H65W	1.8	Soaked
SG0H65D	14.3	As moulded
With geotextile		
SG140H65W	2.8	Soaked
SG300H65W	6.1	Soaked
SG400H65W	6.7	Soaked
SG300H65D	23.5	As moulded

Table 29.5 Loads required to achieve a deformation of 30 mm at the load plate on a sub-base overlying a sabkha sub-grade, with or without a geotextile, in soaked and 'as moulded' conditions (Aiban *et al.*, 2006)

soils formed under arid climates can also be found in temperate regions due to ancient geological processes.

Arid soils present a number of challenges to engineers owing to:

- the highly variable nature of their engineering properties associated with active and ancient geomorphic processes;
- the considerable diurnal temperature ranges experienced, potentially resulting in accelerated disintegration of soil and rock formations as well as man-made structures;
- lithification processes, which can result in cementation of the soil particles, making excavation problematic;
- various problematic soils, which occur in arid conditions, such as those with significant shrink–swell or collapsing potentials, or aggressive salt environments.

Key to understanding how arid soils behave and how these soils can be engineered is achieved through an appreciation of how geomorphology controls their formation. Associated with these processes are soils that can be cemented or uncemented, whilst being unsaturated. Once the geomorphologic effects are understood, the appropriate engineering assessments and mitigation can be applied. A number of logistical challenges associated with water also occur in arid environments: limited availability (resulting in the use of groundwater with high salt concentrations in construction processes, or having to bowse water from external locations to the site); or poor management (for example through lack of appreciation of the significant impact of flash flooding on the erosion of natural, or man-made, soil formations, or the impact of water containing high salt concentrations on construction processes such as the casting of concrete).

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29.7.3 Useful websites

British Geological Survey (BGS); www.bgs.ac.uk
British Standards Institution (BSI); http://shop.bsigroup.com/
US Geological Survey (USGS); www.usgs.gov

It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 40 *The ground as a hazard*
- Chapter 76 *Issues for pavement design*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

Chapter 30
Tropical soils

David G. Toll Durham University, UK

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Tropical soils are formed primarily by *in situ* weathering processes, and hence are residual soils. Terminology for tropical soils is confused and many classification schemes exist, based on either pedological, geochemical or engineering criteria. For classification schemes to be useful they need to include the effects of disintegration due to weathering, mineralogy (particularly the 'unusual' clay minerals that are particular to tropical soils), cementation and structure. Tropical residual soils are highly structured materials, both at macro and micro levels. The micro-structure is produced by leaching out of minerals during weathering, leaving an open structure. Tropical soils are also likely to be cemented soils due to deposition of minerals either during or after weathering. The highly structured nature of tropical soils, combined with the fact, they often exist in an unsaturated state, makes them difficult to deal with as engineering materials. However, they often have good engineering properties. Nevertheless, some tropical soils can be problematic, demonstrating collapse or shrink–swell movements. Ground investigation for tropical soils poses some difficulties due to their heterogeneous nature. Sampling of tropical soils so that the original structure is maintained can be a major challenge, and hence there is a strong emphasis on *in situ* testing methods.

30.1 Introduction

Tropical soils are formed primarily by *in situ* weathering processes, and hence are *residual soils*. The climatic conditions in tropical regions, with high temperatures and high levels of precipitation, lead to stronger chemical weathering of primary minerals and a greater penetration of weathering than occurs in other regions of the Earth (**Figure 30.1**).

The weathering process propagates from the Earth's surface and is controlled primarily by water movement within the joints of the parent rock. Weathering starts on the joint surfaces and progressively penetrates into the rock mass. The direction of propagation of a weathered profile will be dictated by the direction of the joints, so while there is likely to be a vertical component, weathering can also progress horizontally or at other inclinations.

Weathering is usually defined by the degree of decomposition/disintegration of the rock to form soil. This is rated as Grade I (*fresh rock*) through to Grade VI (*residual soil*), where the rock has fully decomposed to form soil. This weathering scale will be discussed in more detail below. The upper three grades, VI (*residual soil*), V (*completely weathered*) and IV (*highly weathered*), represent a material where more than 50% of the rock has decomposed to form soil; the term *tropical residual soil* is used to describe these upper three weathering grades that are dominated by soil material (note the distinction between ***tropical*** *residual soil*, which incorporates all three weathering grades IV, V and VI, and *residual soil,* which only describes Grade VI). The term *saprolite* is used to describe completely and highly weathered material, i.e. a soil which

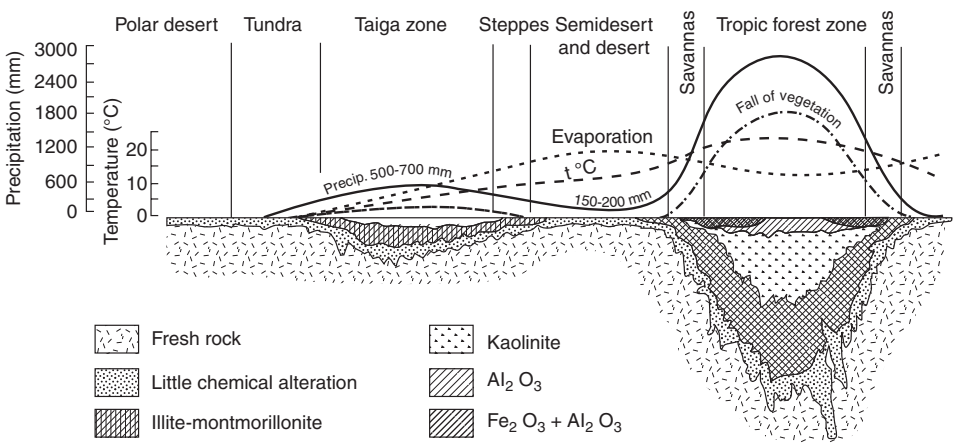


Figure 30.1 Depth of residual soil development related to climatic factors. Weathering may penetrate to greater than 30 m in tropical regions
Reproduced from Strakhov (1967)

still contains some unweathered rock, but does not incorporate the fully decomposed Grade VI.

The particle size distribution (grading) of tropical soils can be highly variable. Early stages of weathering are likely to produce coarser-grained soils. Large boulder- or cobble-sized fragments of unweathered material may still survive as corestones. Stronger chemical weathering will cause minerals such as feldspar and mica to decompose to form clay minerals, so the resulting residual soil may be more fine-grained. The resulting grain size distribution after weathering will depend on the parent rock. For instance, acidic igneous rocks (e.g. granite) contain a high proportion of silica that will resist weathering, so the resulting soil will contain sand-sized silica particles. **Table 30.1** shows the types of residual soil resulting from different parent rocks.

A common feature of many tropical soils is the presence of iron or aluminium oxides, often referred to as sesquioxides (Fe_2O_3 or Al_2O_3). This produces the reddish coloration common to many tropical soils. Iron and aluminium oxides released by weathering are not dissolved as in more acidic environments and remain *in situ*. Iron oxide is crystallised as haematite when the soil is seasonally desiccated, or as goethite in a constantly humid environment; haematite gives the soil a red colour, goethite a brown or ochreous colour. Gibbsite is the main aluminium oxide formed. During the weathering process, silica and bases (K, Na, Ca, Mg) are lost in solution or incorporated into clay minerals.

The term *laterite* has been used to describe a wide range of red soils, and the term has become almost meaningless in an engineering sense. It was used originally to describe a red clay which hardened irrecoverably on exposure to air, but has subsequently been used to describe almost any soil with reddish coloration. The term *laterite* will be used here to refer to a soil with a high degree of iron cementing and the term *red tropical soil* will be used for a soil that has red coloration but without significant cementing.

The weathering process can be divided into three stages based on the mineralogical and geochemical changes (Duchaufour, 1982). These stages are based on the dominant clay minerals that are produced by weathering:

(1) fersiallitisation (smectite clays dominant);

(2) ferrugination (kaolinite and smectites);

(3) ferrallitisation (kaolinite and gibbsite).

As weathering proceeds (ferrugination–ferrallitisation) there is a decrease in concentration of silica and bases and an increase in concentration of iron and aluminium oxides. The resulting soil at the final stage of weathering (ferrallitic soil) is clay-rich and also rich in sesquioxides.

The iron, in the presence of negatively charged clay particles, may be present in the reduced form as ferrous oxide (FeO). Ferrous (Fe^{2+}) iron is soluble and highly mobile, whereas ferric (Fe^{3+}) iron is virtually insoluble. If oxidising conditions become present (such as exposure to air in an excavation or by lowering of the groundwater table) the soluble ferrous iron is precipitated as ferric oxides (Fe_2O_3) such as goethite or limonite. This results in an indurated soil, i.e. a soil that becomes harder through cementation. Alternating reduction and oxidation due to fluctuating groundwater levels in the geological past leads to the development of concretionary or nodular laterites.

Indurated, rock-like *duricrusts* can be formed by cementing of tropical soils by a number of chemicals. Iron and aluminium oxides or silica released during weathering, or calcium carbonate, can be moved laterally or vertically through the soil profile by groundwater flow and may accumulate sufficiently in some horizons for crystallisation to occur, resulting in duricrust formation.

Terminology for tropical soils is confused and many classification schemes exist, based on either pedological, geochemical or engineering criteria. The classification given by Duchaufour (1982), based on weathering stages, has been adopted as the basis of a Working Party Report by the Geological Society (Anon., 1990), later published by Fookes (1997). However, it will be argued that there are other factors, such as weathering grade, degree of cementation and mass structure, that are equally important.

Tropical residual soils are often highly structured materials, both at macro and micro levels. The micro-structure is produced by leaching out of minerals during weathering, leaving an open structure, combined with the effects of secondary cementing by minerals deposited during or after weathering has taken place. The cementing (bonding) can maintain the fabric of the soil in a metastable state, i.e. such a loose structure could not exist if the bonding was not present to support it. **Figure 30.2** shows a typical example of the micro-structure of a residual soil from Singapore. The open fabric can be clearly seen as well as a variation in the degree of cementing within the soil.

Parent rock	Residual soil type	Relative susceptibility to tropical weathering
Calcareous rock (limestone, dolomite)	Gravel in clayey or silty matrix	1 (most vulnerable)
Basic igneous rock (gabbro, dolerite, basalt)	Clay (often grading into sandy clay with depth)	2
Acid crystalline rock (granite, gneiss)	Clayey sand or sandy clay (often micaceous)	3
Argillaceous sedimentary rock (mudstone, shale)	Silt or silty clay	4
Arenaceous sedimentary or metamorphic rock (sandstone, quartzite)	Sand (clayey sand in the case of residual arkose or feldspathic sandstone)	5 (least vulnerable)

Table 30.1 Types of residual soil from different parent rocks
Data taken from Brink *et al.* (1982)

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Tropical soils have a reputation for being 'problematic'. This is because they do not conform to the widely used classification systems that have been developed for temperate sedimentary soils to identify likely engineering behaviour. They are also difficult to investigate, as attempts to sample or test them can destroy the cementing and structure that supports them. This chapter will identify the problems in classifying and investigating tropical soils, and identify schemes and methods that are appropriate for tropical soils.

Of course, some tropical soils can be highly problematic. Residual soils can exist in a loose, structured state and can collapse on loading, or due to wetting, leading to sudden settlements. Other problematic types of tropical soil contain highly expansive smectite clays, resulting in significant heave or shrinkage as wetting or drying occurs. These aspects of behaviour of tropical soils will be considered in a range of applications: foundations, slopes and highways.

30.2 Controls on the development of tropical soils

30.2.1 Weathering processes

One of the most important factors in the engineering behaviour of tropical soils is the degree of weathering. Little (1969) first proposed a six-grade classification for tropical residual soils. This has become the well-established scheme for identifying the degree of rock weathering used in the Geological Society Working Party Report on Core Logging (Anon., 1970) and the Working Party Report on Rock Mass Description (Anon.,

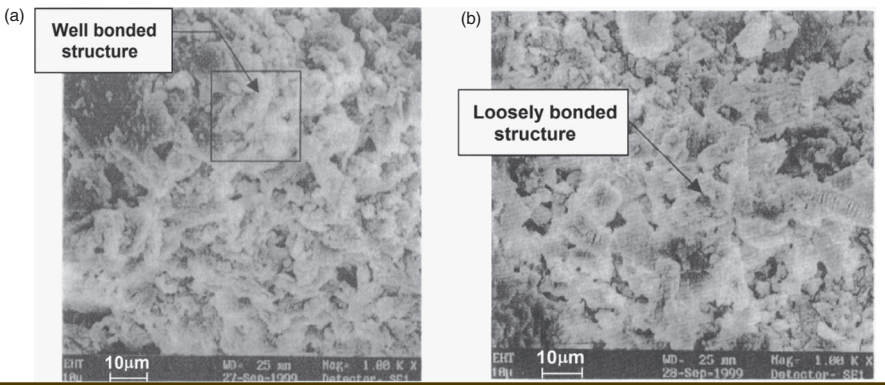


Figure 30.2 Scanning electron microscope images showing the micro-structure of a tropical residual soil from Singapore: (a) well bonded with strong cementing; (b) looser structure
Reproduced from Aung *et al.* (2000)

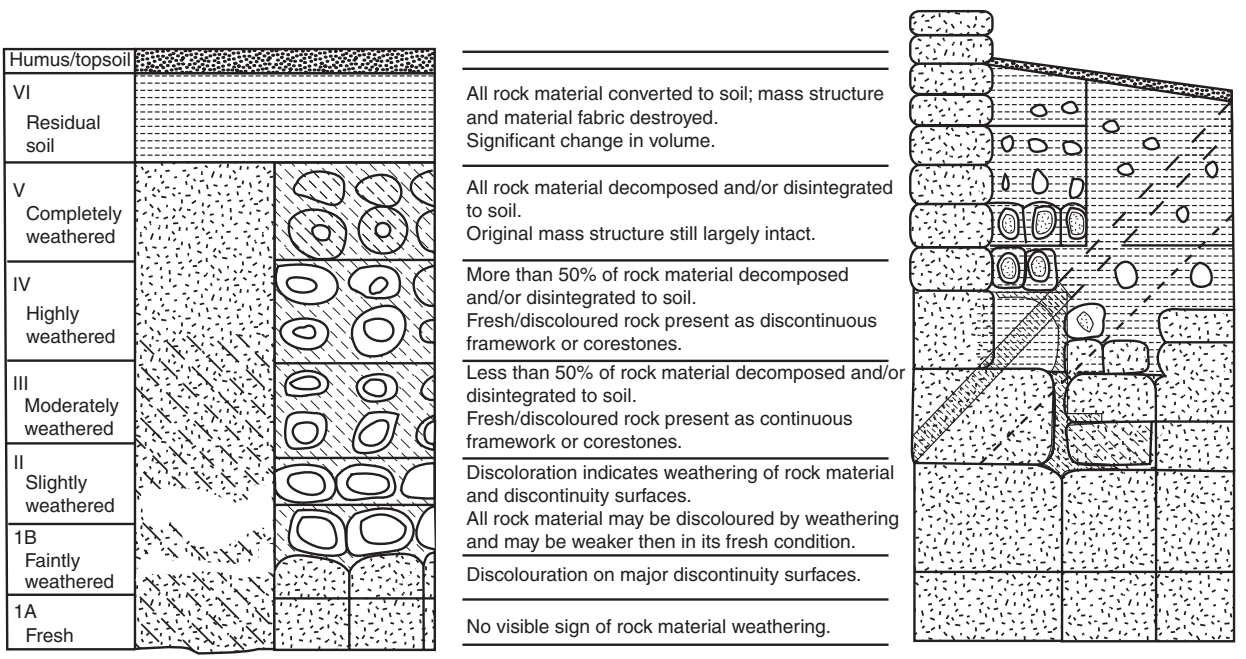


Figure 30.3 Schematic diagrams of typical weathering profiles
Reproduced from Anon. (1990) © The Geological Society

1977). This was the scheme used in ISRM (1978), whereby the relative percentages of 'rock' that has decomposed/disintegrated to form 'soil' are used to define the weathering grade. **Figure 30.3** shows a useful diagrammatic representation of the scheme.

British Standard BS 5930 (British Standard Institution, 1999) provided an alternative method to characterise the degree of weathering by identifying weathering grades based on three approaches, as shown in **Figure 30.4**. However, the British Standard was superseded by Eurocode documents for soil

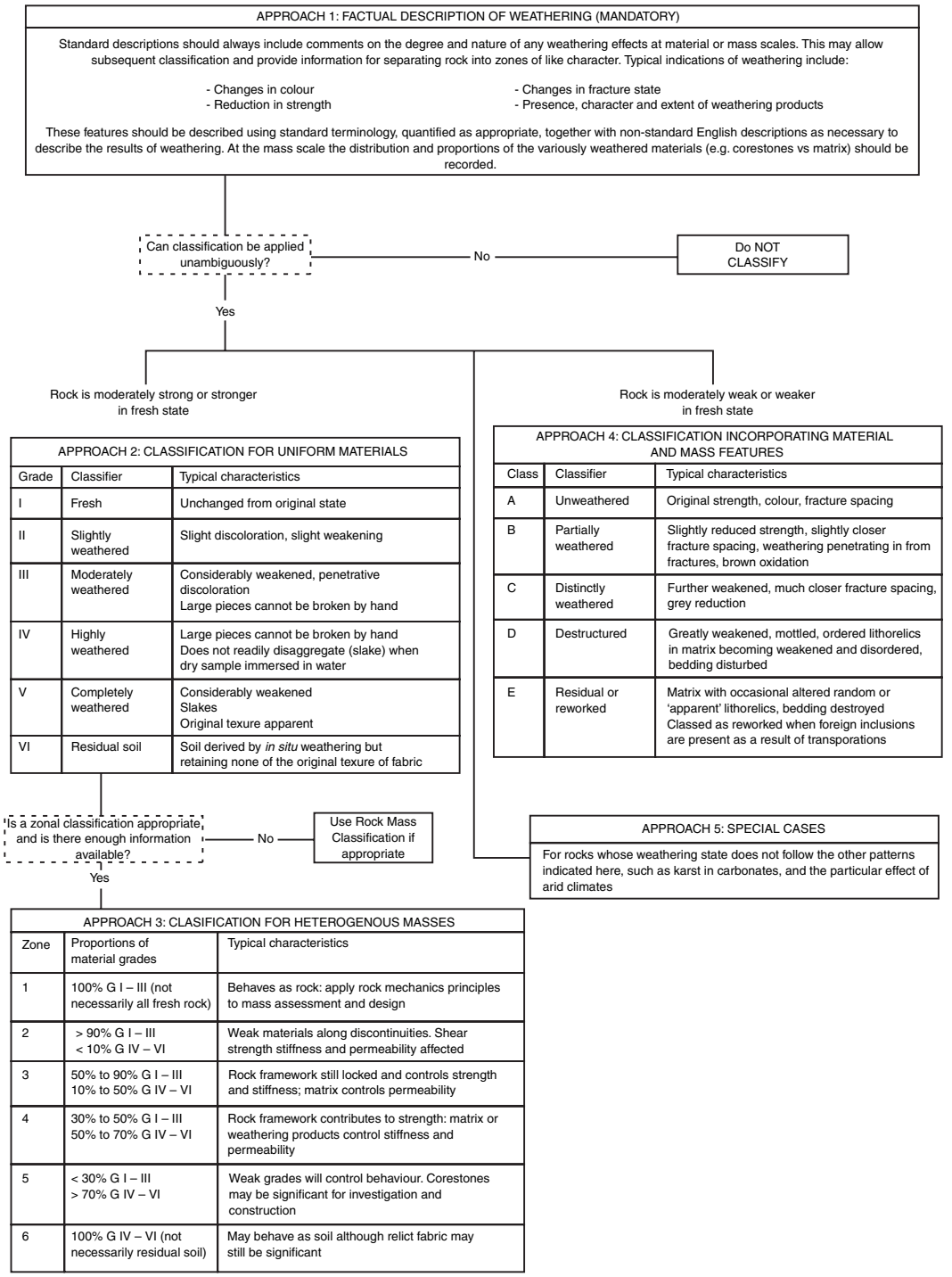


Figure 30.4 Description and classification of weathered rock for engineering purposes
Reproduced with permission from BS 5930 © British Standards Institution 1999

and rock description in April 2010. EN 14689-1:2003 (British Standard Institution, 2003) for rock description returned to the scheme illustrated in **Figure 30.3**. Hencher (2008) saw this transition from the BS approach to the EN definitions as a retrograde step. As will be discussed later, in section 30.3.2, a full classification of tropical soils must include more than just the weathering grade, and should also incorporate mineralogy, secondary cementation and structure.

As has been noted, Duchaufour (1982) identified three phases of development in the weathering process, based in changes in mineralogy and geochemistry, as opposed to the degree of decomposition/disintegration:

(1) fersiallitisation;

(2) ferrugination;

(3) ferrallitisation.

As weathering proceeds (fersiallitisation–ferrugination–ferrallitisation) there are changes in mineralogical composition, in particular the formation of clay minerals. This is associated with a decrease in concentration of silica and bases (K, Na, Ca, Mg) and an increase in concentration of iron and aluminium oxides. Fersiallitic soils are dominated by 2:1 clay minerals (smectites). The main clay mineral present in ferruginous soils is kaolinite (1:1) although some smectite may be present. Ferrallitic soils are dominated by kaolinite and gibbsite (aluminium oxide).

Clay minerals are alumina-silicates that are made up of sheets comprising either silica tetrahedrons (tetrahedral sheets) or alumina octahedrons (octahedral sheets). 1:1 clay minerals are made up of alternating tetrahedral and octahedral sheets. The adjacent layers are closely bonded together, preventing water molecules from penetrating between the sheets. This makes them relatively low-activity minerals (i.e. low shrinkage/swelling). 2:1 clay minerals have one octahedral sheet sandwiched between two tetrahedral sheets. Adjacent layers are not held together strongly, thus allowing water molecules to penetrate between sheets, resulting in high-activity minerals (i.e. high shrinkage/swelling).

Duchaufour's stages of weathering are shown in **Table 30.2**, together with comparisons with other pedological schemes and commonly used geotechnical descriptive terms. Duchaufour's scheme was adopted by a Geological Society Working Party Report on Tropical Soils (Anon., 1990; Fookes, 1997) for its classification of tropical soils, as will be discussed in section 30.3.2.

30.2.2 Parent rock

The type of residual soil will depend on the parent rock. Some typical examples of residual soil types are given in **Table 30.1**. Also shown are the relative susceptibilities to weathering in a tropical environment (Brink *et al.*, 1982).

30.2.3 Climate

The stage of weathering which is achieved is controlled by the climate. The effect of climate on the weathering products is shown in **Table 30.3**.

Reference	Rainfall (mm per annum)	Clay mineral type
Pedro (1968)	< 500	Montmorillonite
	500–1200/1500	Kaolinite dominant
	> 1500	Gibbsite and kaolinite
Sanches Furtado (1968)	800–1000	Kaolinite and montmorillonite
	1000–1200	Kaolinite dominant
	1200–1500	Kaolinite and gibbsite

Table 30.3 Climate and weathering products
Data taken from McFarlane (1976)

Pedological classifications			Common geotechnical terminology	Colour	Mineralogy
Duchaufour	USA	FAO/ UNESCO			
VERTISOL (fersiallitic)	Vertisol	Vertisol	Black Cotton soil	Black, brown, grey	Smectites (montmorillonite), kaolinite
ANDOSOL (fersiallitic)	Inceptisol	Andosol	Halloysite/ Allophane soil	Red, yellow, purple	Kaolinite (halloysite), allophane
FERRUGINOUS	Alfisol	Nitosol, alfisol, lixisol	Red tropical soil	Red, yellow, purple	Kaolinite, hydrated iron oxide (haematite, goethite), hydrated aluminium oxide (gibbsite)
FERRISOL (transitional)	Ultisol	Ferralsol	Lateritic soil, latosol	Red, yellow, purple	Kaolinite, Hydrated iron oxide (haematite, hoethite), hydrated aluminium oxide (gibbsite)
FERRALLITIC	Oxisol	Plinthisol	Plinthite, laterite	Red, yellow, purple	Kaolinite, Hydrated iron oxide (haematite, goethite), hydrated aluminium oxide (gibbsite)

Table 30.2 Terminology for tropical soils

The phases of weathering defined by Duchaufour (1982) can also be related to climate. In Mediterranean or sub-tropical climates with a marked dry season, stage 1 (fersiallitisation) is rarely exceeded. In a dry tropical climate development stops at stage 2 (ferrugination). Only in humid equatorial climates is stage 3 (ferrallitisation) reached (see **Table 30.4**).

The world distributions of the major types of tropical residual soils are shown in **Figure 30.5**. Fookes (1997) notes that these broad classes of soils extend beyond the tropics in favourable conditions. Examples are ferrallitic soils on high-rainfall sub-tropical continental east coasts, and fersiallitic soils in west coast/Mediterranean and continental interiors in mid-latitudes.

The development of duricrusts (rock-like cemented/indurated soils) is also highly dependent on climatic conditions. Ackroyd (1967) presents possible conditions under which the different stages of concretionary material develop in ferricrete (or laterite) (**Table 30.5**). The climate is categorised using the Thornthwaite moisture index (Thornthwaite and Mather, 1954). This provides a way of defining climatic conditions based on the difference between the precipitation and evapotranspiration expressed as a ratio to the potential evapotranspiration. A negative value indicates a dry environment and positive values indicate more humid environments.

Phase	Soil type	Zone	Mean annual temperature (°C)	Annual rainfall (m)	Dry season
1	Fersiallitic	Mediteranean, subtropical	13–20	0.5–1.0	Yes
2	Ferruginous Ferrisols (transitional)	Subtropical	20–25	1.0–1.5	Sometimes
3	Ferrallitic	Tropical	> 25	> 1.5	No

Table 30.4 Summary of Duchaufour's residual soil phases in relation to climate factors
Data taken from Anon (1990)

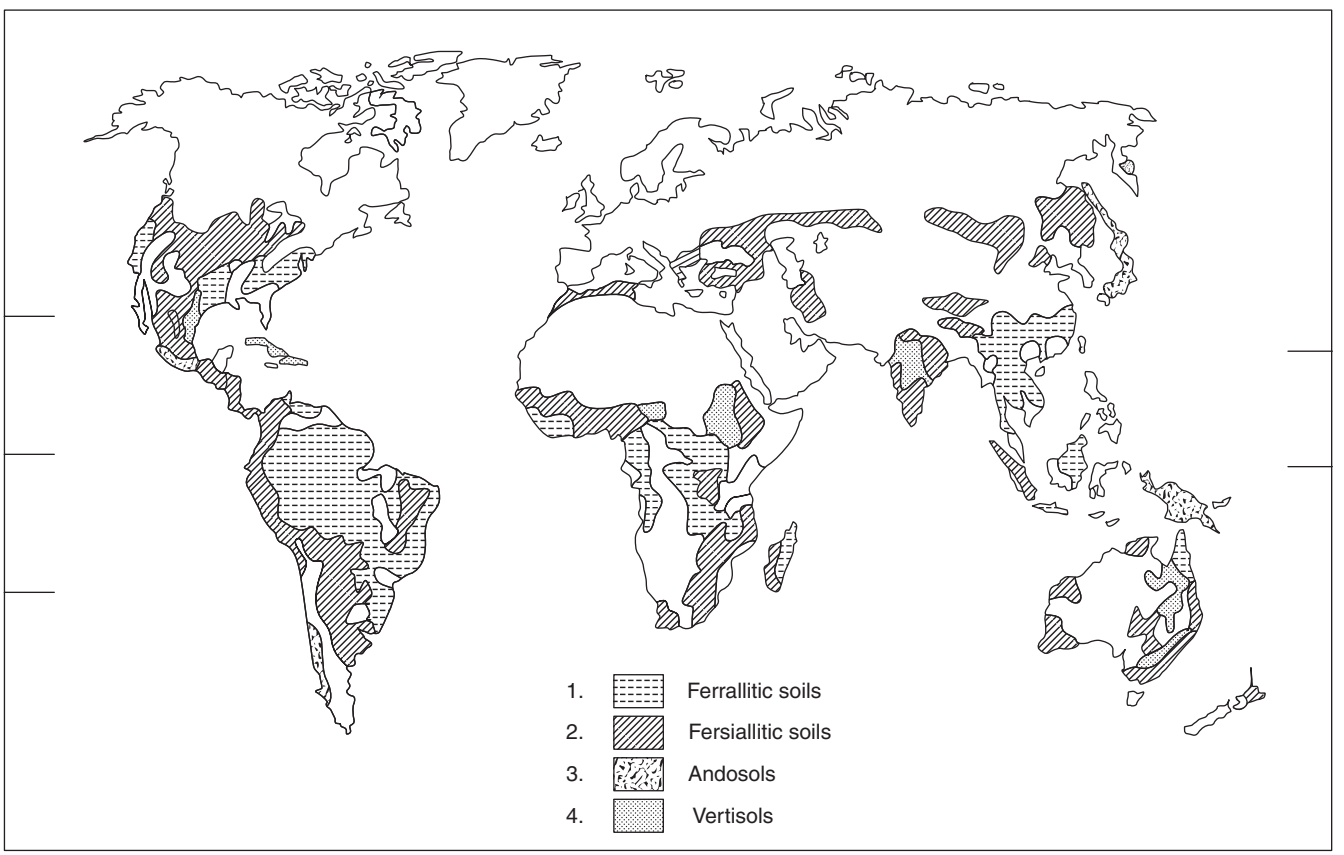


Figure 30.5 Simplified world distribution of tropical residual soils
Based on FAO World Soil Map (Fookes, 1997)

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However, McFarlane (1976) points out that the conditions for development of concretions are not the conditions under which the laterite minerals initially form. McFarlane identified a model of laterite formation based on a fluctuating groundwater table in an overall downward flow environment. A cycle of development may result, where the original lateritic deposit is weathered, producing iron and aluminium oxides that are mobilised and recrystallised elsewhere to form a new laterite deposit.

30.2.4 Relief and drainage

Relief and drainage have major effects on the stages of weathering. Idealised soil profiles (catenas) are shown for three rock types in **Figure 30.6**. However, these simple catenary sequences will be modified by the drainage conditions. If the base of the slope is poorly drained, then fersiallitic soils will develop even on acid crystalline rocks. Equally if drainage conditions are good then ferrallitic soils may develop on basic rocks.

30.2.5 Secondary cementation

Many tropical soils are bonded due to the presence of chemical cementing agents. These chemicals may develop by pedogenic processes induced by the accumulation of iron or aluminium oxides (as they are released during weathering) or the movement of leached silica, aluminium and iron oxides, gypsum or carbonates by groundwater flow. These minerals may accumulate sufficiently in some horizons for crystallisation to occur.

The resulting harder varieties of pedogenic materials such as calcrete, silcrete, ferricrete and alucrete are known as duricrusts or pedocretes. The cementing agents for each type of duricrust are identified in **Table 30.6**.

Netterberg (1971) categorised calcretes into calcified material, powder calcrete, nodular calcrete and hardpan calcrete, and showed that the engineering properties were highly dependent on the type of calcrete. Charman (1988) adopted a similar classification scheme for laterite (or ferricrete) (**Table 30.7**).

Different stages of laterisation are reflected in the silica/alumina ratio (SiO_2/Al_2O_3) (Desai, 1985). Typical values are given in **Table 30.8**.

	Annual rainfall (mm)		
	750–1000	1000–1500	1500–2000
Thornthwaite moisture index	–40 to –20	–20 to 0	0 to +30
Length of dry season (months)	7	6	5
Type of product	Rock laterite or cuirasse	Hard concretionary gravels	Minimum requirements for concretions to develop

Table 30.5 Possible conditions for development of concretionary laterite

30.3 Engineering issues

30.3.1 Investigation

Ground investigation for tropical soils poses some difficulties due to the heterogeneous nature of tropical residual soils. The soil properties will be dependent on the degree of weathering, which varies within the weathered mass; there can often be 'corestones' of relatively unweathered rock (which can be of cobble or boulder size) contained within a matrix of more

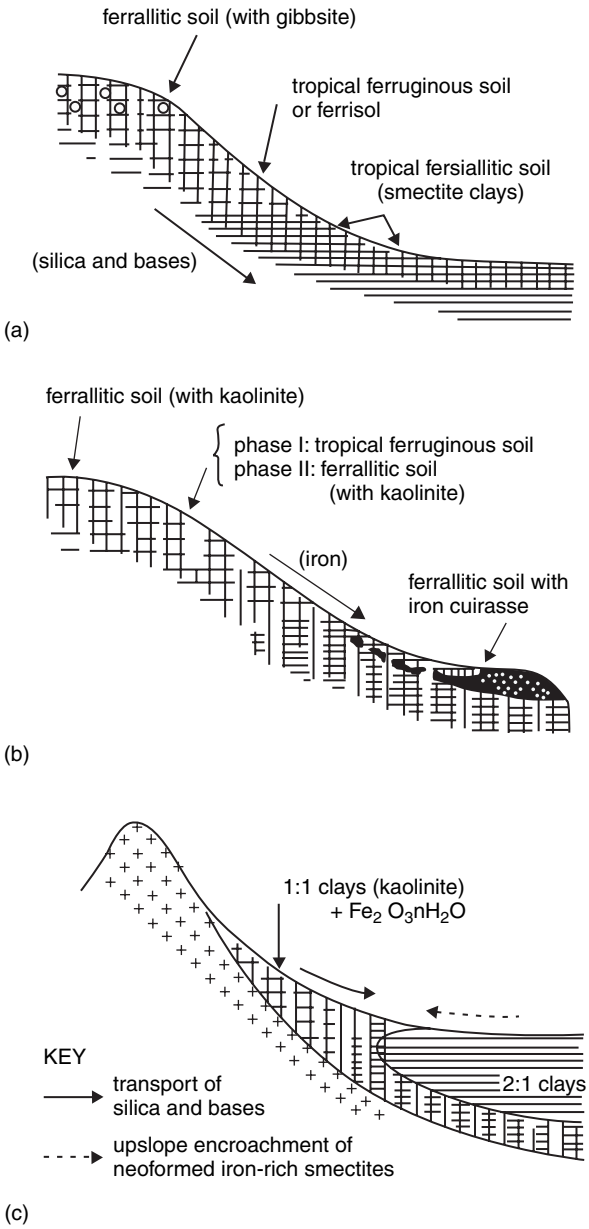


Figure 30.6 Simple soil catenas for different rock types: (a) basic volcanic rocks in a humid tropical climate; (b) acid crystalline rocks in a humid tropical climate; (c) around inselbergs in the tropics (Inselbergs are prominent steep-sided hills of resistant igneous rock rising out of a flat plain)
Reproduced from Fookes (1997) (from Duchaufour, 1982)

weathered material. The original rock mass structure, such as joint sets, will be represented in relict form in the tropical residual soil and can represent planes of weakness (Irfan and Woods, 1988; Au, 1996).

The overall degree of weathering will vary from the highest degree of weathering near the ground surface to less weathered and possibly fresh rock deeper within the ground mass. However, the changes between grades of weathering will be progressive; there are unlikely to be sharp distinctions between layers of different weathered materials, so identification of boundaries between weathering grades can be highly subjective.

In addition to this, the degree of secondary cementing can be highly variable, both in terms of the amount of the cementing minerals (e.g. iron oxides) and the strength of the cemented bonds. The 'structured' nature of tropical soils (Vaughan, 1985a) makes them particularly sensitive to disturbance during sampling and testing.

Duricrust	Cementing mineral
Silcrete	Silica
Calcrete	Calcium or magnesium carbonate
Gypcrete	Calcium sulphate dihydrate
Alucrete (bauxite)	Hydrated aluminium oxides
Ferricrete (laterite)	Hydrated iron oxides

Table 30.6 Duricrusts and their cementing minerals

Sampling of residual soils so that the original structure is maintained can be a major challenge. Driven or pushed samplers are likely to cause significant breakdown of the structure, resulting in samples that no longer represent the *in situ* conditions. Large block samples, trimmed by hand, may be the only way to get satisfactory samples. Details of sampling procedures are outlined in Fookes (1997).

There has been success with using rotary coring techniques, using triple-tube core barrels and air foam flush to recover good quality samples (Phillipson and Brand, 1985; Phillipson and Chipp, 1982). Mazier core barrels (73 mm diameter) are commonly used, as are Treifus triple-tube barrels (63 mm diameter). Plastic lining tubes should be used to protect the core on extrusion. Water drilling flush should not be used, as the flushing medium can cause erosion of the core or result in changes in water content. Even compressed air flush can potentially change the suctions in samples (Richards, 1985).

Due to the difficulties in recovering high-quality, undisturbed samples, there has been a strong emphasis on *in situ* testing to determine the engineering properties of tropical soils. Pressuremeter and plate load tests are suitable tests for assessing *in situ* properties. Standard penetration testing (SPT) is also widely used. Cone penetration testing (CPT) and vane testing are unlikely to be suitable for weathered profiles containing significant amounts of rock material, as penetration of

Age	Recommended name	Characteristic	Equivalent terms in the literature
Immature (young)	PLINTHITE	Soil fabric containing significant amount of lateritic material. Hydrated oxides present at expense of some soil material. Unhardened, no nodules present but may be slight evidence of concretionary development	Plinthite Laterite Lateritic clay
	NODULAR LATERITE	Distinct hard concretionary nodules present as separate particles	Lateritic gravel Ironstone Pisolitic gravel Concretionary gravel
	HONEYCOMB LATERITE	Concretions have coalesced to form a porous structure which may be filled with soil material	Vesicular laterite Pisolitic ironstone Vermicular ironstone Cellular ironstone Spaced pisolitic laterite
Mature (old)	HARDPAN LATERITE	Indurated laterite layer, massive and tough	Ferricrete Ironstone Laterite crust Vermiform laterite Packed pisolitic laterite
	SECONDARY LATERITE	May be nodular, honeycomb or hardpan, but is the result of erosion of pre-existing layer and may display brecciated appearance	

Table 30.7 Classification of laterite
Data taken from Charman (1988)

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the testing device will be restricted. However, these techniques may be suitable for Grade VI residual soils.

The pressuremeter has been used for the investigation of properties of tropical soils (e.g. Schnaid and Mantaras, 2003). However, as Schnaid and Huat (2012) note, the pressuremeter response curve will be dependent on a combination of *in situ* horizontal stress, soil stiffness and strength parameters, and these parameters will reduce with destructuration at high shear strains. Schnaid and Huat suggest that, in residual soils, the pressuremeter should be viewed as a 'trial' boundary value problem against which a theoretical pressure-expansion curve predicted using a set of independently measured parameters can be compared to field pressuremeter tests. A good comparison between a number of observed and predicted curves can give confidence that the selected parameters used in the prediction are sensible, and can therefore be adopted in design. An example of a pressuremeter test carried out in a Brazilian residual soil reported by Schnaid and Mantaras (2003) is compared against a numerical simulation in **Figure 30.7** and shows that good agreement can be achieved.

Stage of laterisation	SiO_2/Al_2O_3
Unlaterised soil	> 2
Lateritic soil	1.3–2
Laterite	< 1.3

Table 30.8 Typical values for silica/alumina ratio
Data taken from Desai (1985)

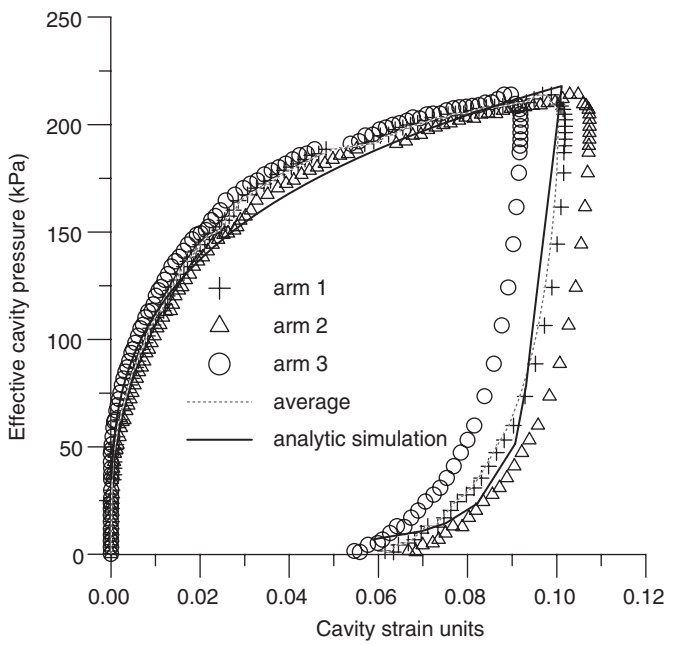


Figure 30.7 An example of a pressuremeter test carried out in saprolite from Brazil compared to an analytical simulation
Data taken from Schnaid and Mantaras (2003)

Plate load testing is a popular option for assessing tropical soils as it has the advantage of testing a larger volume of material, thereby giving a measure of the mass behaviour of a heterogeneous soil. Procedures for performing such tests are described by Barksdale and Blight (1997). The results can be used primarily to estimate the stiffness or compressibility of the soil. The interpretation of the results may be complicated by the structured nature of the soils and unsaturated state, as both factors will influence the initial soil stiffness and the yield stress observed. Schnaid and Huat (2012) suggest that interpretation of test data may require sophisticated numerical analysis with appropriate constitutive models, rather than the conventional interpretation methods commonly used to estimate the elastic modulus.

Standard penetration tests are widely used around the world for assessing the relative density and hence the angle of shear resistance of soils, based on empirical correlations. It has to be recognised that such correlations have usually been established from databases of tests on sedimentary soils. They are unlikely to be appropriate for tropical residual soils as they take no account of any cementing/bonding that influences the strength of the soil. Local relationships may need to be determined for a particular tropical soil that take account of the cementing and local variability.

Schnaid *et al*. (2004) have suggested that cementation of residual soils can be observed by considering the ratio of the elastic stiffness to ultimate strength, G_0/N_{60}, where G_0 is the shear modulus at very small strains and N_{60} is the SPT test value standardised to a reference value of 60% of the potential energy of the SPT hammer. They plotted this ratio against $(N_1)_{60}$, where the N_{60} value is normalised to take account of the vertical effective stress and hence should give a closer indication of relative density of the deposit. They found that the bonded structure has a marked effect for residual soils, producing values of normalised stiffness (G_0/N_{60}) that are considerably higher than those observed in fresh uncemented materials.

30.3.2 Classification

The search for an appropriate scheme for classifying tropical soils has occupied engineering geologists, geotechnical engineers, pedologists and soil scientists for many years. A proliferation of such schemes exist: the Geological Society Working Party Report on Tropical Soils (Anon., 1990; Fookes, 1997) tabulates over 20 different schemes developed between 1951 and 1986, each with a different end use in mind. Although a number of well-developed pedological schemes exist, they are not always relevant for classifying tropical soils for engineering use. Leong and Rahardjo (1998) conclude that, in spite of the efforts to develop classification schemes by geologists, pedologists and engineers, no suitable classification system exists for the study of residual soils.

The Geological Society Working Party Report on Tropical Soils (Anon., 1990; Fookes, 1997) represents the most complete attempt to produce a useful classification scheme for tropical soils. The Working Party opted for a purely pedogenic

classification scheme based on the work of Duchaufour (1982) (**Table 30.2**).

Another major work on residual soils (Blight, 1997) proposes a rather different classification scheme (Wesley and Irfan, 1997). They identify the factors influencing residual soil behaviour as:

- physical composition (e.g. percentage of unweathered rock, particle size distribution, etc.);
- mineralogical composition;
- macro-structure (layering, discontinuities, fissures, pores, etc. discernible to the naked eye);
- micro-structure (fabric, inter-particle bonding or cementation, aggregation, etc.).

To take these factors into account they suggest grouping soils into three types (**Table 30.9**).

According to Wesley and Irfan, Group A (which is not strongly influenced by particular clay minerals) is typical of many tropical soil profiles. The group is further sub-divided by structure components (macro-structure dominated, micro-structure dominated or soils not significantly influenced by either). They suggest that the engineering properties of Group B (which includes vertisols) will be very similar to transported soils with the same clay mineralogy. Group C, which is dominated by minerals only found in tropical soils (halloysite, allophane and aluminium and iron sesquioxides) is sub-divided according to the minerals present.

A way to incorporate these different aspects is proposed by the author of this chapter, where the four factors of disintegration, mineralogy, cementation and structure (DMCS) are encoded on a six-point scale for each factor. It is suggested that the four factors are depicted as shown in **Figure 30.8** using the scales listed in **Table 30.10**. A higher number in each category indicates a more problematic material.

Major division		Sub-group
Group A	Soils without a strong mineralogical influence	(a) Strong macro-structure influence
		(b) Strong micro-structure influence
		(c) Little or no structure influence
Group B	Soils with a strong mineralogical influence derived from clay minerals also commonly found in transported soils	(a) Montmorillonite (smectite group)
		(b) Other minerals
Group C	Soils with a strong mineralogical influence deriving from clay minerals only found in residual soils	(a) Allophane sub-group
		(b) Halloysite sub-group
		(c) Sesquioxide sub-group (gibbsite, goethite, haematite)

Table 30.9 Classification of residual soils
Data taken from Wesley and Irfan (1997)

It has to be recognised that there will be cross-linkages between the different categories; for example a material that is fresh rock (D = 1) but has no secondary cementing (C = 6) will not be a problematic material, even though the cementing category has a high score. Similarly, a residual soil (D = 6) that has hardpan cementing (D = 1) will not be problematic, as the cementing will overcome the decomposition due to weathering and result in a strong rock-like soil.

30.3.3 Characteristics and typical engineering properties

The stages of weathering result in different clay mineralogies, and these are reflected in the cation exchange capacity (CEC) of the clay fraction (Anon., 1990). Typical values are given in **Table 30.11**.

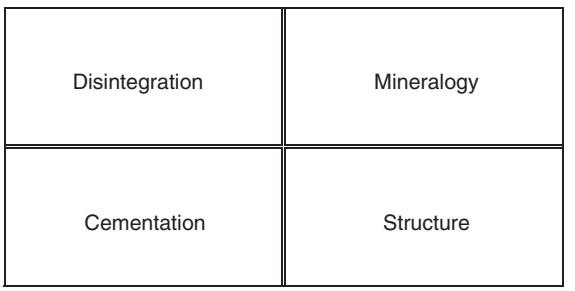


Figure 30.8 DMCS classification scheme

Grade	Disintegration	Grade	Mineralogy[1]
6	Residual soil	6	Smectite (vertisol)
5	Completely weathered	5	Smectite/kaolin (ferruginous)
4	Highly weathered	4	Allophane/halloysite (andosol)
3	Moderately weathered	3	Kaolinite (siallitic)
2	Slightly weathered	2	Kaolinite (ferrisol)
1	Fresh/faintly weathered	1	Kaolinite/gibbsite (ferrallitic)
Grade	**Cementation[2]**	**Grade**	**Structure spacing**
6	No cementing agents present	6	Very small (less than 60 mm)
5	No evident cementing effect	5	Small (60 mm to 200 mm)
4	Weakly cemented	4	Medium (200 mm to 600 mm)
3	Nodular	3	Large (600 mm to 2 m)
2	Honeycomb	2	Very large (greater than 2 m)
1	Hardpan	1	No evident macro-structure

[1] These are ranked in order of engineering behaviour rather than stages of weathering.

[2] This is a measure of secondary cementing of the weathered material, not the initial cementation of the parent rock.

Table 30.10 Proposed classification of tropical residual soils

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Fersiallitic soils can comprise vertisols or andosols that form at the early stage of weathering. Vertisols commonly occur in areas of impeded drainage such as on valley floors. They are often black or dark brown in colour and contain smectite clay minerals. They often exhibit excessive shrinkage and swelling properties and present major engineering problems.

Andosols are fersiallitic soils that develop from volcanic parent rocks and contain amorphous allophane or halloysite. They frequently exist in an extremely loose state, and can have water contents of around 200%. Nevertheless, these high water contents do not reflect in their engineering behaviour since they generally have low compressibility and high angles of shearing resistance (Wesley 1973, 1977).

The latter stages of weathering (ferruginous, ferrisols, ferralitic) usually result in soils with red coloration, reflecting higher iron and aluminium sesquioxide contents. They generally contain low-activity kaolinite minerals, and do not usually present major engineering problems. However, they may perform quite differently from temperate sedimentary soils, and hence the application of standard classification systems can lead to difficulties.

Stage of weathering	Cation exchange capacity (mEq/100 g)
Fersiallitic soil	> 25 (typically 50)
Ferruginous soil	16–25
Ferrallitic soil	< 16

Table 30.11 Typical values for cation exchange capacity of the clay fraction
Data taken from Anon. (1990)

Ferruginous soils (red tropical soils) have red coloration but do not contain high iron oxide contents. Ferrisols (lateritic soils) have higher iron contents, and contain granular nodules (pisoliths) of iron cemented material. Ferricrete (laterite cuirasse or carapace) is an indurated deposit, heavily cemented with iron oxides, and can behave like weak rock.

The effect of secondary cementation is to improve the engineering properties. **Figure 30.9** shows the variation of void ratio, compressibility and strength properties (c' and ϕ') in a weathering profile. In the Grade VI residual soil, the void ratio will be high due to leaching out of minerals and the resulting structured form of the soil. This open structure results in high compressibility and low strength properties.

Figure 30.9 shows the effect that secondary cementing by sesquioxides (laterisation) can have on the residual soil near the ground surface. This results in an upper profile with variations in the degree of cementing, from a concretionary or partly cemented layer to a fully cemented layer at the surface. The presence of the sesquioxides partially fills voids and reduces

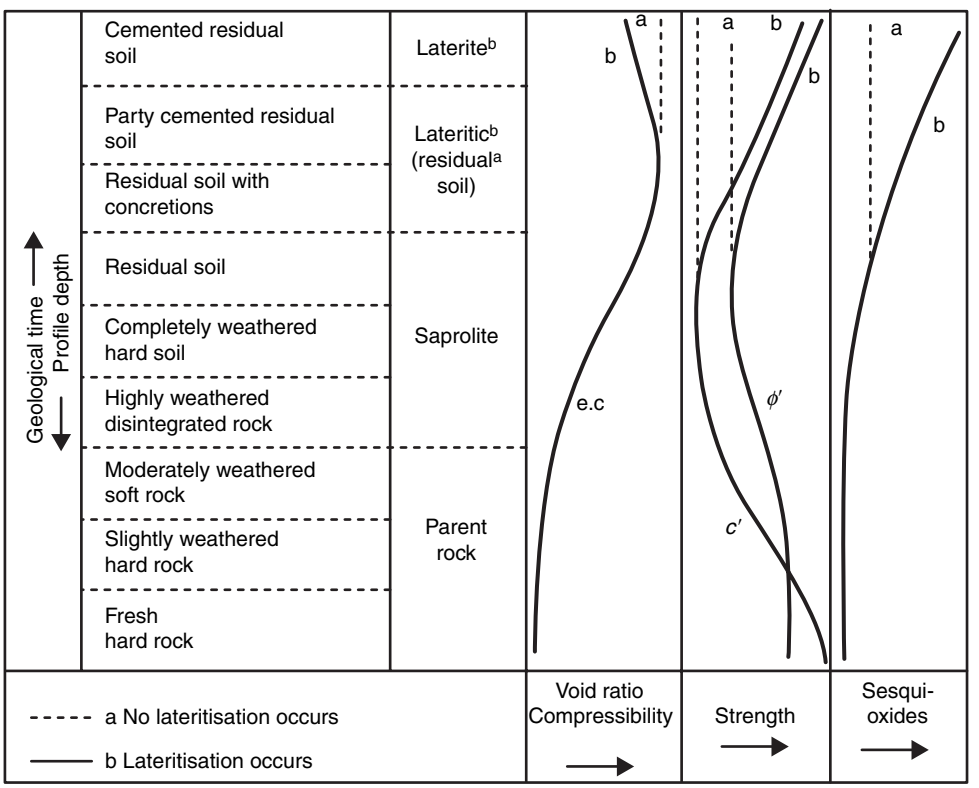


Figure 30.9 Changes in compressibility and strength in a weathering profile
Reproduced from Blight (1997), Taylor & Francis Group (adapted from Tuncer and Lohnes, 1977 and Sueoka, 1988)

the compressibility. Secondary cementing also produces major improvements in strength properties.

The permeability of tropical residual soils is controlled to a large extent by the macro-structure provided by relict joints etc. The more mature residual soils (Grade VI) may have lower permeability (**Table 30.12**), as the weathering will modify the macro-fabric and reduce the dominance of relict joints, as well as producing more clay minerals. However, materials that have been laterised can have higher permeability due to the open micro-structure and vesicular nature of the more cemented materials.

30.3.4 Problematic behaviour

Geotechnical classification schemes such as the Unified Soil Classification Scheme (USCS), which are widely used for temperate sedimentary soils, have severe limitations when applied to tropical soils. This gives tropical soils a reputation for being 'problematic' as they do not conform to these simple classification systems. However, many tropical soils, particularly the 'red' soils, can be good engineering materials and are often not problematic.

That is not to say that all tropical soils are problem-free. Some residual soils can exist in a loose state. Some ferruginous soils (Red Coffee soils) may exist with densities as low as 0.6 Mg/m^3, i.e. less than the density of water. This loose structure may be sustained by an unsaturated state, where suctions give strength to clay 'bridges', supporting the coarser particles and maintaining a low density. If the soil is wetted, so that the strength of the clay bridges is lost, the soil may collapse. Cementing agents may also maintain a loose metastable structure. If the soil is loaded beyond the yield strength of the bonding material, this can also lead to collapse. More information is provided in Chapter 32 *Collapsible soils*.

A further factor to be aware of is that the particles themselves may be crushable; this may be due to a loss of intrinsic strength as a result of chemical attack during weathering, or it may be that the coarse-grained size fraction is in fact made up of finer particles that are held together by secondary cementing or by a clay matrix. When subjected to high stresses the particles may start to crush, resulting in additional volumetric compressions (Lee and Coop, 1995).

Zone	Relative permeability
Organic topsoils	Medium to high
Mature residual soil and/or colluvium	Low (generally medium or high in lateritic soils if pores or cavities present)
Young residual or saprolitic soil	Medium
Saprolite	High
Weathered rock	Medium to High
Sound rock	Low to medium

Table 30.12 Permeability of weathering profiles in igneous and metamorphic rocks
Data taken from Deere and Patton (1971)

Another major problematic type of tropical soil is vertisols, as they contain smectite clays and can be highly expansive. Information on dealing with such swelling/shrinking soils is discussed in Chapter 33 *Expansive soils*.

Other problematic aspects of tropical soils are identified below. In many cases, the soils are not problematic *per se*, but problems result from inappropriate use of classification systems that were designed for temperate soils and are not applicable to tropical soils.

30.3.4.1 Presence of 'unusual' clay minerals

Some clay minerals found in tropical soils (halloysite and allophane) exist in non-platey forms. These are unlike the common clay minerals found in temperate soils (kaolinite, illite, montmorillonite), which are generally platey in nature. The engineering behaviour of allophanous or halloysitic soils is often very different from what would be predicted by schemes like the USCS (Wesley, 1973, 1977).

Allophane is amorphous (or is poorly structured) and can hold significant amounts of water within the amorphous mineral. On drying it appears that allophane forms a more ordered structure, completely changing the nature of the soil. An allophanous clay can change in behaviour to appear like a sand after drying.

Halloysite is a member of the kaolinite family but has a tubular habit. Water can be held within the 'tubes', where it does not contribute to the engineering behaviour. Halloysite exists in two forms: hydrated halloysite and metahalloysite containing no water of crystallisation. In metahalloysite the 'tubes' may split or become partially unrolled. The transition takes place if the moisture content reduces below about 10% or the relative humidity drops below about 40% (Newill, 1961). The change is irreversible.

Because of the ability of allophane and halloysite to hold water that does not contribute to the engineering behaviour, they are often classified as troublesome soils (since they have high natural water contents and liquid limits) (Wesley, 1973). However, they generally have very good engineering properties (Wesley, 1977). They show high angles of shearing resistance (compared to kaolinite or montmorillonite). Also, since they are not platey in form, they do not show a significant reduction in angle of shearing resistance due to clay particle alignment. Therefore, residual angles of shearing resistance are also high.

30.3.4.2 Presence of cementing agents

Many tropical soils are structured due to the presence of cementing agents that produce a physical bonding between soil particles. Schemes like the USCS are based on measurements on remoulded (or destructured) soil (Atterberg limits and particle size determination) and therefore cannot take account of soil structure. This is a severe limitation even for many temperate soils, but can be particularly limiting for tropical soils.

Under tropical weathering conditions iron and aluminium oxides are released and are not dissolved (as would occur in more acidic environments), so remain *in situ*. The presence

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of iron and aluminium oxides significantly affects the behaviour of tropical soils. Newill (1961) demonstrated that these oxides can suppress the plasticity of tropical clay soils, since on removal of the iron oxides the liquid limit was found to increase. This is the case if the oxides have an aggregating effect on the clay minerals. However, it is also possible for the oxides to contribute to plasticity, as was found by Townsend *et al.* (1971). If the oxides are present as amorphous colloids they can have a large water retention capability due to their large specific surface, and will then contribute to plasticity.

30.3.4.3 Difficulties in determining Atterberg limits

Atterberg limit determinations on tropical soils are sensitive to the methods of preparation (e.g. Moh and Mazhar, 1969). Different degrees of pre-test drying (e.g. oven dried, air dried or tested from natural moisture content) can produce very significant differences in the Atterberg limits (Anon., 1990; Fookes, 1997). In addition the amount of mixing of the soil during test preparation can also change the index properties significantly (Newill, 1961).

A comparison of the effects of different degrees of pre-test drying on the Atterberg limits is shown in **Table 30.13**. It should be noted that these changes are irreversible, and a permanent change in plasticity is produced by drying. To overcome problems relating to pre-test preparation, Charman (1988) suggests a procedure for testing the susceptibility to the method of preparation. This involves testing at different drying temperatures and different periods of mixing. If sufficient time is not available for such a detailed test programme, the best solution is to test the material without drying below the natural moisture content with a standard mixing time of five minutes.

30.3.4.4 Difficulties in determining particle size distributions

Like Atterberg limits, the measurement of clay content can also be affected by pre-test drying, since the drying process causes the clay particles to aggregate (Newill, 1961). These aggregations are only partially disaggregated by standard dispersion techniques, and clay fractions are often underestimated. For example, a red clay from Sasumua, Kenya showed a clay fraction of 79% when testing at natural moisture content, but this reduced to an apparent value of 47% after oven drying (Terzaghi, 1958). Another problem is that the coarse fraction of red soils often consists of weakly cemented particles which readily break down and change grading during sieving or compaction (Gidigasu, 1972; Omotosho and Akinmusuru, 1992).

30.3.4.5 Unsaturated state

Many tropical soils exist in an unsaturated state, since evapotranspiration is greater than precipitation. Water tables are often greater than 5 m deep, in many cases considerably deeper. The strength of these soils will be very dependent on moisture conditions.

Soil suction is made up of two components: *matric suction* and *osmotic suction* (also called *solute suction*). The sum is known as the *total suction*. Matric suction is due to surface tension forces at the interfaces (menisci) between the water and the gas (usually air) phases present in unsaturated soils (the surface tension effect is sometimes referred to as capillarity). Osmotic suctions are due to the presence of dissolved salts within the pore water.

In much of the soil science literature, suction is expressed in pF units, i.e. the logarithm (to base 10) of the suction expressed in centimetres of water (Schofield, 1935). For engineering applications, it is generally more convenient to use conventional stress units. The relationship to convert from pF units to kPa is given by:

$$\text{suction (kPa)} = 9.81 \times 10^{\text{pF}-2}. \qquad (30.1)$$

The suction scale (showing both kPa and pF units) with some points of reference and indications of the moisture condition of a soil are shown in **Figure 30.10**.

The maximum suction that can be sustained within the pores of a soil will depend on the pore size. In a clean sandy soil, where pore sizes will be of the order of 0.1 mm or larger, the maximum suctions will be very small (typically < 5 kPa). In clean silty materials, where pore sizes might be of the order of 0.01 mm, the maximum suctions are likely to be less than 100 kPa. However, in clayey soils, where pore sizes can be less than 0.001 mm, high suctions greater than 1000 kPa can be sustained. This explains why clean sandy soils have no strength when they dry out (they lose the suction 'bonds' that hold them together as they cannot sustain high suctions). However, clayey soils can become very strong when they dry out, due to the high suctions that are maintained in the fine pores of the soil. The suctions pull the soil particles together and give the soil considerable strength in a dry state.

	At natural moisture content			Air dried			Oven dried (105°C)		
Location	LL	PL	PI	LL	PL	PI	LL	PL	PI
Costa Rica	81	29	52				56	19	37
Dominica	93	56	37	71	43	28			
Kenya (red clay)	101	70	31	77	61	16	65	47	18
Kenya (lateritic gravel)	56	26	30	46	26	20	39	25	14

Table 30.13 Effect of drying on classification tests on red soils

Suction (kPa)	Suction (pF)	Reference points	Moisture condition
1 000 000	7	Oven dry	Dry
100 000	6		
10 000	5		
1 000	4	Wilting point for plants Plastic limit	Moist
100	3		
10	2		Wet
1	1		
0.1	0	Liquid limit saturated	

Figure 30.10 The suction scale

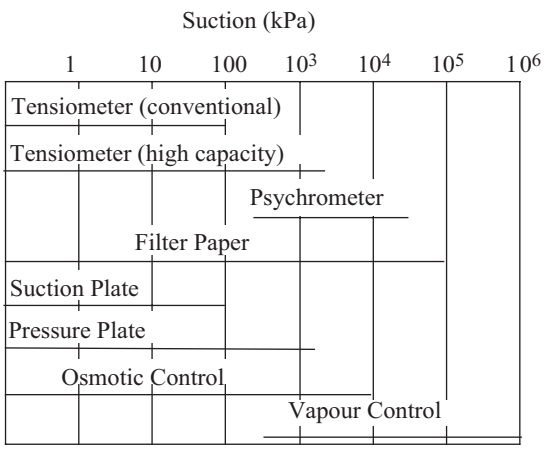


Figure 30.11 Ranges for which suction measuring/controlling devices are suitable

There are a number of different techniques for suction measurement and control. Their suitability varies according to the range of suctions operating. An indication of appropriate ranges is shown in **Figure 30.11**. It is generally necessary to use a variety of techniques in order to cover the entire suction scale.

As a soil dries out (or wets up) the suction within the soil will change. The relationship between water content and suction is known as the soil water retention curve (SWRC) (also called the soil water characteristic curve, SWCC). Although water contents

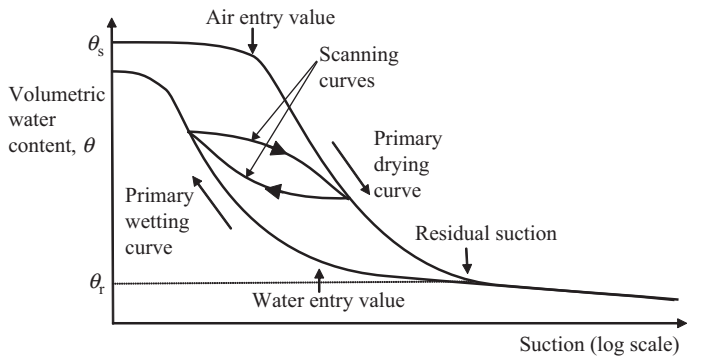


Figure 30.12 Typical soil water retention curve

are usually defined gravimetrically (i.e. by weight) in geotechnical engineering, soil water retention curves are often expressed in terms of volumetric water content, θ, or degree of saturation, S_r versus suction. **Figure 30.12** shows a typical SWRC.

If the soil starts from a saturated state and is then subject to drying, it will follow the *primary drying curve*. At a value of suction known as the *residual suction* (with a corresponding residual water content, θ_r) the SWRC may flatten, and much smaller changes in volumetric water content result from an increase in suction. To achieve zero water content (equivalent to an oven-dried condition) requires a suction of the order of 1 GPa (pF 7) (Fredlund and Xing, 1994). On wetting from an oven-dried state, the soil will follow the *primary wetting curve*. The primary drying and wetting curves define an envelope of possible states within which the soil can exist. If drying is halted part way down the primary drying curve and wetting is started, the soil will follow an intermediate *scanning curve*, which is flatter than the primary wetting curve, until the primary wetting curve is reached. Therefore, different suctions can exist at a given water content depending on the pathway followed.

The most commonly used approach to interpreting shear strength behaviour in unsaturated soils is to adopt an extended version of the traditional Mohr–Coulomb approach. This extension to unsaturated soils was put forward by Fredlund *et al.* (1978). It involves two separate angles of shearing resistance, to represent the contribution to strength from the net stress (total stress referenced to the pore air pressure) and matric suction (the pore water pressure referenced to the pore air pressure), giving the shear strength equation as

$$\tau = c' + (\sigma - u_a) \tan \phi^a + (u_a - u_w) \tan \phi^b, \qquad (30.2)$$

where

τ is shear strength;

c' is the effective cohesion intercept (many tropical soils demonstrate a true cohesion intercept at zero effective stress due to their bonded structure);

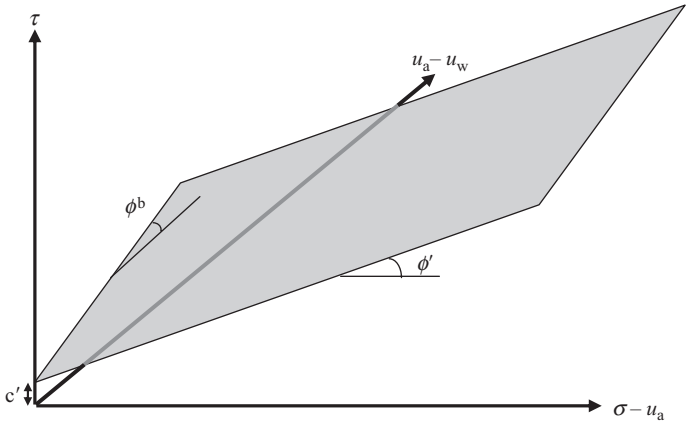


Figure 30.13 The extended Mohr–Coulomb failure envelope

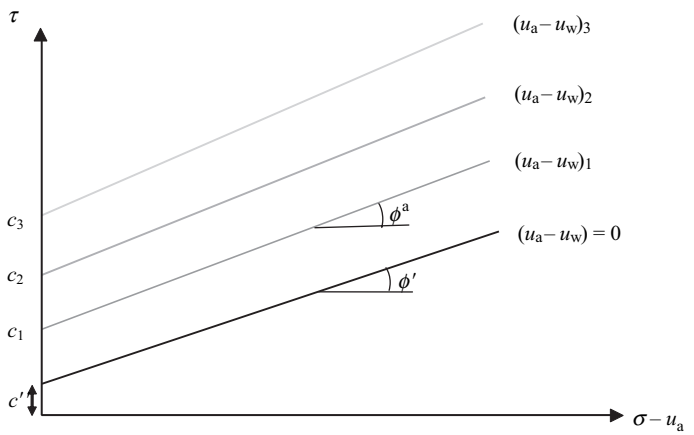


Figure 30.14 The extended Mohr–Coulomb failure envelope in net stress space

ϕ^a is the angle of shearing resistance for changes in net stress $(\sigma - u_a)$;

ϕ^b is the angle of shearing resistance for changes in matrix suction $(u_a - u_w)$.

This separates the effects of net stress $(\sigma - u_a)$ and suction $(u_a - u_w)$ and treats them differently by having two angles of shearing resistance relating to the two components of stress. The extended Mohr–Coulomb failure surface is shown in three dimensions in **Figure 30.13**. The surface is shown by views in the net stress plane in **Figure 30.14** and in the suction plane in **Figure 30.15**.

Figure 30.14 shows that the strength envelope increases as the suction increases. This can be represented as an increase in the total cohesion, c where:

$$c = c' + (u_a - u_w) \tan\phi^b \qquad (30.3)$$

Figure 30.15 shows the increase in total cohesion, c, as the suction increases. The slope of the graph is defined by ϕ^b. The

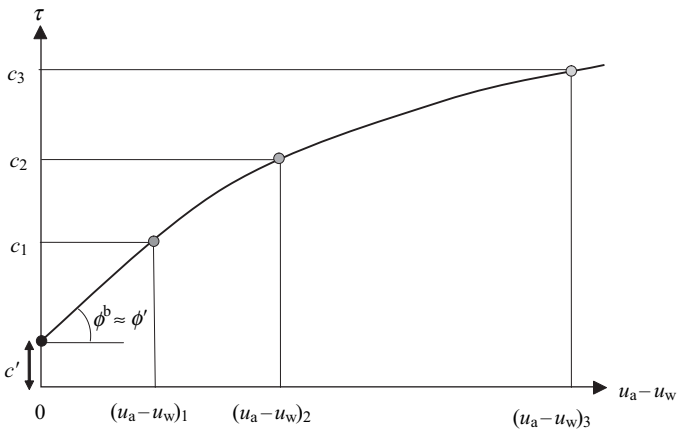


Figure 30.15 The extended Mohr–Coulomb failure envelope in matric suction space

relationship between τ and $(u_a - u_w)$ has been found to be non-linear by Escario and Saez (1986) and Fredlund *et al.* (1987). Below the air entry value (when the soil remains saturated) ϕ^b is equal to ϕ', but at higher suctions the value of ϕ^b reduces (**Figure 30.15**). The tangent value may fall to zero at high suctions, implying no further increase in strength at higher suctions.

A more complete model of unsaturated soil behaviour is that proposed by Alonso *et al.* (1990) and now known as the Barcelona Basic Model. This extends the Modified Cam Clay model to the unsaturated state, and provides the coupling between volumetric and deviatoric behaviour that is essential for a complete understanding of soil behaviour. It introduces the concept of a loading-collapse (LC) surface that defines yielding due either to external loading (total stresses) or wetting (loss of suction).

If the role of suction in an unsaturated soil is not recognised, then test results can be incorrectly interpreted. For instance, the compressibility of an unsaturated soil may be observed to be low at the water content at which the specimen is tested, as a result of the presence of a significant suction. However, the same soil could have much higher compressibility if the soil is wetted and loses the suction. Similarly, an apparent cohesion intercept observed in strength tests may be largely due to suction rather than being a true 'cohesive' contribution to strength from bonding. Again, this component of strength will reduce (and may even be lost entirely) if the soil is wetted.

30.3.5 Foundations

Problems with shallow foundations on tropical residual soils are usually associated either with collapse problems on loose, metastable soils or shrink–swell movements on expansive vertisols.

Collapse settlements can result from overstressing the cemented micro-structure of tropical soils. The compressibility may be low if the stresses applied do not exceed the yield strength of the cementing material. However, if the foundation is loaded beyond this stress, large and rapid settlements can take place.

Collapse settlements of foundations can also occur due to wetting. Some ferruginous soils (Red Coffee soils) may exist with densities as low as 0.6 Mg/m^3, i.e. less than the density of water. This loose structure is often maintained by bridges of clay minerals which support the sand- or silt-sized particles. The strength of these bridges is controlled by suction, and if the soil wets up or becomes flooded, there is a rapid collapse of the loose structure, resulting in large surface settlements.

A build-up in moisture under a pad or raft foundation can occur due to cutting off evaporation and changes in the temperature regime due to construction of a covered area (e.g. a concrete foundation). However, wetting up can also be caused by simpler means, such as the construction of soakaways for buildings, or leaking services, resulting in loss of strength and failure of the foundation.

If shallow foundations are constructed on vertisols, the likely problem will be seasonal movements as water contents change beneath the foundation due to wetting and drying. Foundation heave will be observed in wet season conditions, and settlements will be induced by shrinkage in dry season conditions. The zone of variation of water content will affect the edges of the foundation, while the central area of a large raft may not be affected. This results in differential movements that can be severely deleterious to the foundation and the overlying structure.

Methods to deal with foundation construction on expansive soils are:

30.3.5.1 Removal and replacement

Remove expansive material and replace with non-expansive soils. Generally the expansive layer extends to depths too great to economically allow complete removal and replacement. It must then be determined what depth of excavation and fill will be necessary to prevent excessive heave.

30.3.5.2 Remoulding and compaction

The swell potential of expansive soils can be reduced by decreasing the dry density. Compaction at low densities and at water contents wet of optimum will reduce the swell potential. However, the bearing capacity of the soil at the lower density may not be adequate. Some soils have such a high potential for volume change that compaction control does not significantly reduce swell potential.

30.3.5.3 Surcharge loading

If the surcharge load applied is greater than the swelling pressure then heave can be prevented. For example, a swell pressure of 25 kPa can be controlled by 1.5 m of fill and a concrete foundation. However, swelling pressures are often too high (~400 kPa) for this to be a realistic option.

30.3.5.4 Pre-wetting

This is based on the assumption that increasing the water content will cause heave prior to construction. If the high water content is maintained, there will be no appreciable volume change to damage the structure. However, the procedure has many drawbacks. Expansive soils are normally clays with low permeability, and the time required for adequate wetting may be years. Also, the increase in water content will reduce the strength of the soil and cause reductions in bearing capacity.

30.3.5.5 Moisture control by horizontal and vertical barriers

Soil expansion problems are primarily the result of fluctuations in water content. Non-uniform heave is the major cause of damage, as opposed to total heave. If changes in water content can be made to occur slowly and if the water content distribution can be made uniform, differential heave can be minimised. Moisture barriers do not prevent the heave taking place but have the effect of slowing the rate of heave and providing a more uniform moisture distribution.

Horizontal barriers installed around a building can limit the migration of moisture into the covered area. Concrete aprons, or paved areas for car parking can achieve this. The width of the barrier should be sufficient to extend the 'edge distance', i.e. the distance measured inward from the slab edge over which the soil moisture varies enough to cause soil movement (Post-Tensioning Institute, 1980).

30.3.6 Slopes

Many of the landslides which occur in the saprolitic zone of tropical residual soils are directly or indirectly controlled by relict discontinuities (Brand, 1985; Nieble *et al.*, 1985; Dobie, 1987; Irfan *et al.*, 1987; Irfan and Woods, 1988). Many types of mineral infillings and coatings may be present along relict discontinuities as a result of weathering processes, including clay minerals. Some discontinuities may be polished or slickensided as a result of internal deformation in the slopes (Irfan, 1998). These infilled or polished surfaces may have low residual angles of shearing resistance, providing a plane of weakness, so that failure is constrained to occur on these relict surfaces.

Landslides are often triggered by rainfall, particularly in tropical climatic regions, where rain storms can be very intense. Major landslides occur all too often, but minor landslides occur even more frequently. Although minor landslides may not lead to loss of human life, they still have economic and social impact.

A clear linkage has been established between landslide occurrence and high rainfall in tropical regions of the world. This has been confirmed by studies in Brazil (Wolle and Hachich, 1989), Puerto Rico (Sowers, 1971), Fiji (Vaughan, 1985b), Hong Kong (Brand, 1984; Au, 1998), Japan (Yoshida *et al.*, 1991), Nigeria (Adegoke-Anthony and Agada, 1982), Papua New Guinea (Murray and Olsen, 1988), Singapore (Pitts, 1985; Tan *et al.*, 1987; Chatterjea, 1994; Rahardjo *et al.*, 1998; Toll, 2001), South Africa (van Schalkwyk and Thomas, 1991) and Thailand (Jotisankasa *et al.*, 2008).

Soil slopes in tropical regions are normally unsaturated during the dry season, and the groundwater table may often be at

depths of more than 10 m for most of the year. When the soil is unsaturated, suction or negative pore water pressure provides additional strength to the soil, hence stabilising the slope. This additional strength may disappear during an intense rainstorm when the soil becomes saturated and pore water pressure becomes zero.

Figure 30.16 shows rainfall data for a large number of landslides in Singapore (Toll, 2001). It shows the rainfall on the day of the landslide (*triggering* rainfall) plotted against the rainfall in the five-day period preceding it (*antecedent* rainfall). It can be seen that it is usually not a single rain storm that produces failure; rather it is a build-up of pore water pressure over a number of days due to the antecedent rainfall followed by a storm that finally triggers the landslide event. However, there are occasions where a single storm is big enough to produce a failure even when there has been no significant antecedent rainfall in the preceding days.

It is important when studying climate effects on slopes that we do not always assume that rainfall will produce a rise in water table level. Infiltration of rainfall at the surface can produce significant changes in pore water pressure without a change in water table (although a perched water table may be induced at the surface) (Toll, 2006).

30.3.7 Highways

Traditionally, materials used in the construction of road bases have been clean graded aggregates, generally obtained from crushed rock. However, in tropical climatic zones, good quality rock for crushing is often unavailable because of the extensive weathering that occurs in the tropics. Even when it is available, the costs of processing and transporting the material can make this an uneconomical option. Therefore naturally occurring materials such as lateritic gravels or calcretes are widely used for construction of roads with low traffic volumes. These natural materials generally contain a greater amount of fines, and the fines have higher plasticity, than is accepted by many existing specifications for construction materials. Details of materials which have been successfully used as road base construction materials in the tropics are given by Lionjanga *et al*. (1987), Grace and Toll (1989), Gidigasu (1991), Metcalf (1991), Netterberg (1994) and Gourley and Greening (1997).

Because naturally occurring gravels have greater quantities of fines it means that soil suction and fabric are major factors controlling their behaviour. The presence of around 10% clay in a lateritic gravel was sufficient to provide a matrix with small pore sizes which could sustain significant suctions (Toll, 1991). Yong *et al.* (1982) similarly report that a small clay fraction had an important effect in influencing the suction characteristics of a weathered granite. Therefore, the effects of suction should not be overlooked in granular materials if they contain small amounts of clay. Provided the fines are well distributed, it will be possible to develop high suction throughout the soil. The matrix of fines will then act as a binder which can hold the granular material together, thus imparting overall high strength and stiffness.

The ability to maintain soil suction, and hence maintain good performance, is highly dependent on the avoidance of wetting up of the road base material, particularly in unsurfaced roads. In cases where the water table is close to the surface, the benefits of suction cannot be relied upon, and conventional specifications using good quality aggregates must be adopted. For cases where the water table is more than 5 m below the

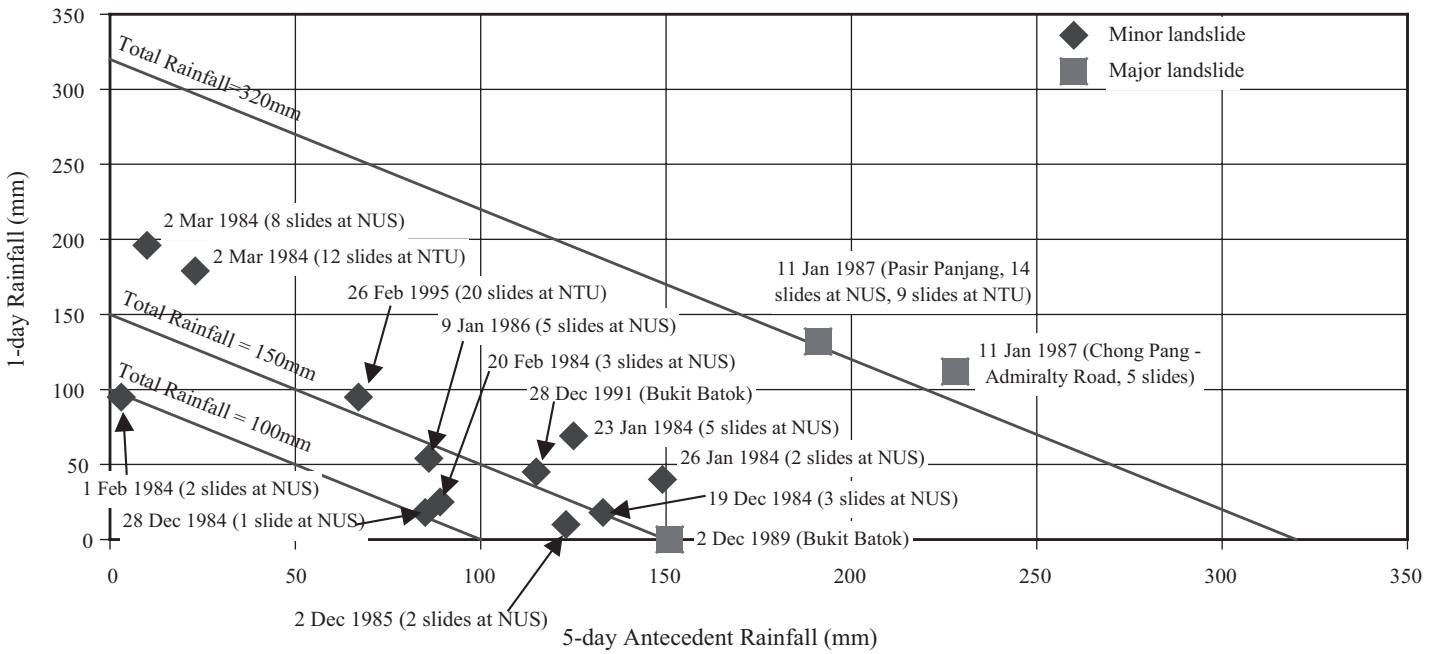


Figure 30.16 Rainfall events leading to landslides in Singapore

ground surface, reliance can be placed on suction, provided drainage measures ensure that water will not pond on the road surface. A bituminous surface is beneficial in preventing direct infiltration into the road base material.

Trial constructions in Kenya of low-volume roads surfaced with a sprayed bituminous seal coat compared lateritic gravel with conventional crushed stone for the road base (Grace and Toll, 1989). It was found that the lateritic gravel sections showed better performance. This was because failures of the bituminous surface coat allowed water to penetrate and spread within the high-permeability crushed stone base, softening the sub-grade and causing large areas of cracking. On the laterite sections, a small pothole formed where the bituminous surface coat failed, but it did not spread, due to the low permeability of the road base material. Any water accumulating in the pothole during a rain storm evaporated during drying periods. Therefore, the laterite sections performed better than the 'higher quality' construction methods.

Toll (1991) argues that a good material for a road base in a sub-tropical or tropical climate will have sufficient fines to allow significant suctions to develop and also produce low permeability. A small amount of clay can be beneficial in this. However, the fines content should not be so great as to suppress the dilatent tendency of the granular fraction or to significantly reduce the angle of shearing resistance. Also, any clay present should have low activity in order to restrict shrinkage and swelling. The fines should be well distributed throughout the fabric if they are to support high suctions and provide a strong binder, holding the granular fraction together. This also produces low permeability.

Charman (1988) provides comparisons between specifications for natural gravels for road bases used around the world. More recently, Paige-Green (2007) gives recommended material specifications for unsealed rural roads (**Table 30.14**) based on experience in southern Africa. This is based on distinguishing between materials that will become slippery, erode, ravel or form corrugations (**Figure 30.17**).

Paige-Green suggests a minimum value of soaked CBR (at 95% modified AASHTO compaction) of 15%. This is even lower than the *minimum* value of 20% suggested by Grace (1991) for bituminous sealed roads, combined with an *average* value of soaked CBR of 40%. Gourley and Greening (1997) suggest a minimum soaked CBR of 45% (at 100% modified AASHTO compaction) for sealed roads carrying less than 0.01 million equivalent standard axles (ESA), but a higher requirement of 55–80% for more highly trafficked roads carrying 0.5 million ESAs (the lower limit for road base CBR of 55% is for a weak sub-grade with CBR = 3–4% and the higher limit of 80% for a strong sub-grade with CBR > 30%).

30.4 Concluding remarks

Tropical soils pose many challenges for geotechnical engineers. They are highly structured at both micro and macro levels. They are often cemented due to deposition of minerals either during or after weathering has taken place. They can be highly heterogeneous. A major difficulty is that traditional classification systems that have been developed for temperate sedimentary soils cannot be used to infer likely engineering behaviour for tropical soils. Simple classification tests, such as Atterberg limits, cannot be easily determined, due to the presence of unusual clay minerals, such as halloysite and allophone, or due to the effects of iron or aluminium sesquioxides.

Tropical soils are often thought of as problematic soils, largely due to the difficulty in classifying them. However, many tropical soils, particularly the 'red' soils, often have good engineering properties, such as low compressibility and high strength. The cemented structure can enhance their strength.

Nevertheless, some tropical soils do demonstrate problematic behaviour. Examples are those that exist in a loose, metastable state that can collapse when loaded, or when subject to wetting. Other problematic tropical soils are vertisols that contain active smectite clay minerals that demonstrate excessive shrink–swell behaviour when subject to drying and wetting.

Property	Value
Maximum size (mm)	37.5
Maximum oversize index (I_o)[1]	5%
Shrinkage product (S_p)[2]	100–365 (maximum of 240 preferable)
Grading coefficient (G_c)[3]	16–34
Soaked CBR (at 95% modified AASHTO compaction)	> 15%
Treton impact value (%)	20–65

[1] I_o, the oversize index, is the percentage retained on 37.5 mm sieve
[2] S_p = linear shrinkage × (% passing 0.425 mm sieve)
[3] G_c = ((% passing 26.5 mm – % passing 2.0 mm) × (% passing 4.75 mm))/100

Table 30.14 Recommended material specifications for unsealed rural roads
Data taken from Paige-Green (2007) © The Geological Society

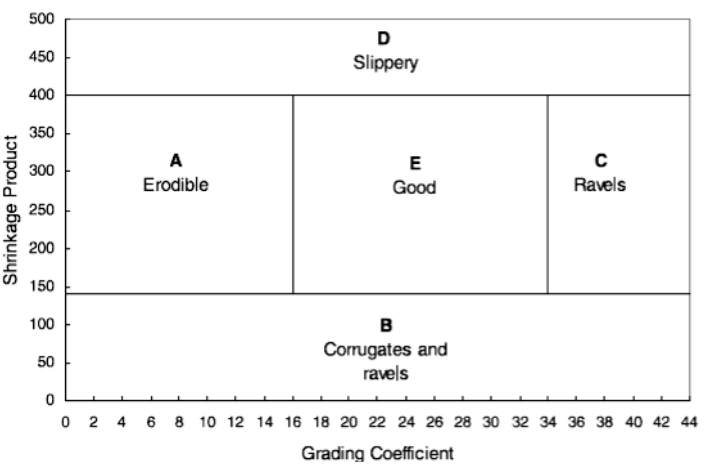


Figure 30.17 Categories of road performance for rural unsurfaced roads (See Table 30.14 for definitions of shrinkage product and grading coefficient)
Reproduced from Paige-Green (2007)

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It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 40 *The ground as a hazard*
- Chapter 76 *Issues for pavement design*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 31
Glacial soils

Barry Clarke University of Leeds, UK

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A large volume of geological literature exists on glacial soils, which are common throughout the world's temperate zone. In the UK they account for some 60% of all soils and globally 10%. There is little published information on geotechnical characteristics despite the number of ground investigations. Glacial soils can vary from deformed basal layers that retain many of the original features of those layers to unsorted mixtures of gravel, sands, silts and clays to laminated clays. They can be deposited through a process of pressure and shear beneath a glacier as it advances, or be deposited when the ice melts. This creates a spatially variable soil which contains features that impact on the mass behaviour and lead to wide variation in results from a single source of glacial soil, making the selection of design parameters difficult. Routine sampling may not pick up these features. Tills are deposited in such a way that their characteristics do not necessarily conform to soil mechanics theory and empirical relationships created from studies of gravitationally compacted soils. Hence developing the ground model depends on knowledge of the genetic classification, creating a regional database to enhance data from new investigations, tests on reconstituted tills and a consistent framework to evaluate the mechanical characteristics.

31.1 Introduction

Glacial soils are a hazard. The modes of deposition result in spatially variable soils, which are difficult to classify and characterise. Inadequate ground investigation and lack of understanding of the impact of processes on the geotechnical behaviour leads to claims and even failures. Yet a large volume of literature exists on Quaternary (glacial) geology (Hughes *et al.*, 1998) and glacial soils are common throughout the world's temperate zone. In the UK they account for some 60% of the soils (**Figure 31.1**) and globally 10%. A more detailed map and GIS database related to features formed during the last glacial event in the UK (the Devensian) are provided by Clark *et al.* (2004). The majority of this literature, which is often contradictory, focuses on geological aspects of glacial till (or till). Although a large number of ground investigations have been carried out in glacial terrains in the UK, only a fraction of the information they revealed is available to the geotechnical profession (Trenter, 1999). Hence the issues highlighted in **Table 31.1**.

Glacial soils include those that are deposited under water (glaciolacustrine and glaciofluvial soils) resulting in consolidation processes that fit with soil mechanics theory developed for gravitationally compacted or sedimented soils. However, the deposition of the majority of glacial soils (tills) is a complex process as a result of periods of glaciation, pore water regimes and source materials leading to spatially variable, unsorted mixes of boulders, gravel, sands, silt and clay which can include lenses and layers of sand and gravel and laminated clay. This clay can include rock flour and clay particles. There is evidence that these tills do not conform to classic theories of soil mechanics (Clarke *et al.*, 1997; Hughes *et al.*, 1998) which means the application of empirical and theoretical rules based on observations of sedimented soils is inappropriate. They can be difficult to sample and the inclusion of gravel and sand lenses can produce a range of stiffness, strength and permeability with depth. Quality sampling is usually restricted to clay matrix-dominant till which is free of cobbles. Any gravel or fissures, which is usual, may affect the sampling, and lead to a wide range of test results for a given property, making it difficult to select design parameters unless there is a clear set of rules to take into account the natural variability and the impact the fabric, composition and the sampling process have upon the derived parameters.

In conclusion, till may have been gravitationally compacted, sheared, possibly reworked and weathered. Till can contain a range of particle sizes from clays to boulders and can vary from clay matrix-dominant till that contains discrete granular particles to clast-dominated tills, which contain some fines. Tills can be fissured and laminated. The implication for the construction industry is that till is a challenging material as it is difficult to predict the characteristics of a particular till. This chapter focuses on tills, though reference is made to other glacial soils to help understand the formation of tills.

31.2 Geological processes

The movement of a glacier or ice sheet across the underlying soil or rock either deforms that material creating a deformation till, or erodes it, moves it to a new location and deposits it either as a lodgement or melt-out till or as a water-lain deposit (**Figure 31.2**). These processes create a range of different debris formations (see **Table 31.2**), which leads to a genetic classification based on those processes (**Table 31.3**). The erosion and transportation processes, for example, result in a breakdown of the material due to crushing, fracturing and abrasion, which may be assisted by freezing of the ground at the base of the

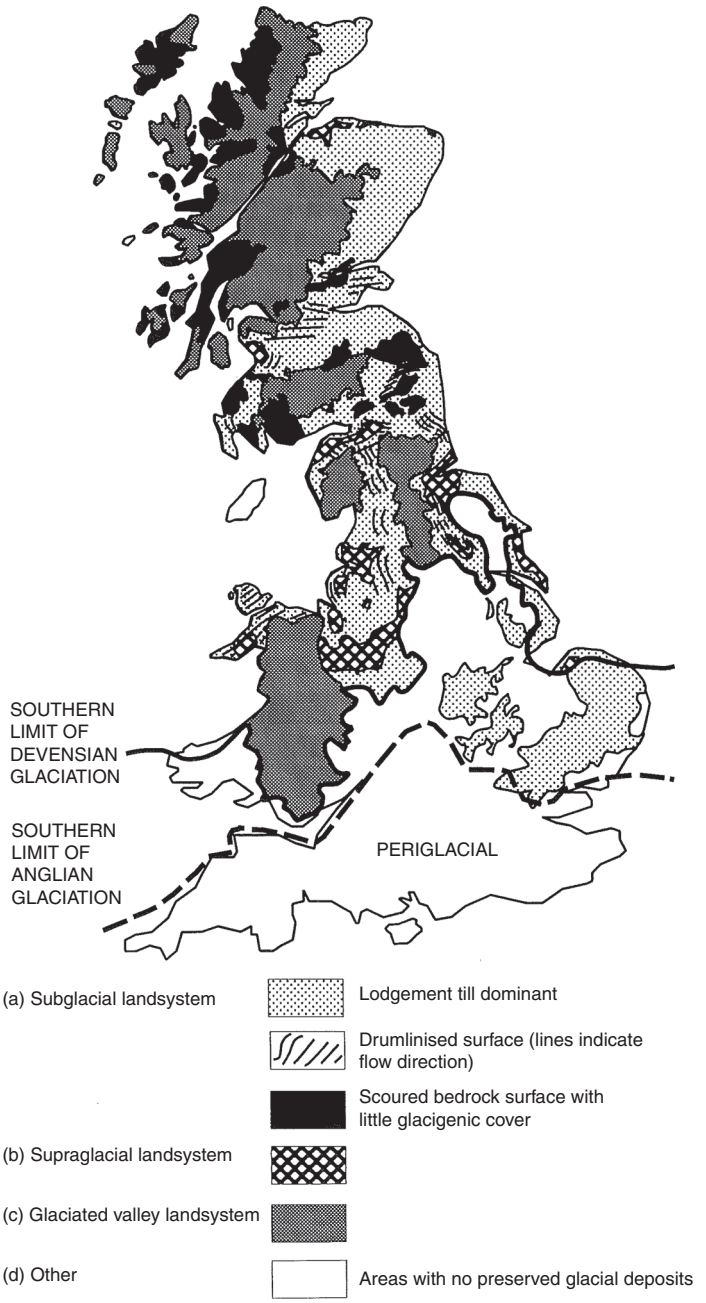


Figure 31.1 Distribution of glacial soils in the UK
Reproduced from Eyles and Dearman (1981), with kind permission from Springer Science+Business Media

glacier. This leads to a homogenised diamicton, which is the geological term used to describe a poorly sorted sediment. If a glacier is moving across soil and that soil is not frozen, then sub-glacial deformation can occur. Hence it is possible to find glacio-tectonic features, such as faults and folds (**Figure 31.3**), in glacial soils. Further deformation results in the folded material breaking up, leading to pockets of original layers known as boudins within the till matrix. Glacial soils produced from ice sheets are all derived from the base of the ice sheet. Glacial soils produced from valley glaciers can include supraglacial debris that falls from the valley sides. Glacial debris is classed as sub-glacial (base of the glacier), supraglacial (surface of the glacier) or englacial (within the glacier). Supraglacial and englacial materials may not be modified by the movement of the ice; basal materials are modified because of abrasion and crushing. Debris can move between these three zones depending on the physical processes that take place within the glacier as it moves and as the temperature changes. The debris (**Figure 31.4**) can be spread into place beneath a glacier (e.g. lodgement and deformation tills); deposited beneath a glacier as the glacier melts (e.g. melt-out till); deposited within a melt-water lake (e.g. laminated clays); or deposited from melt-water streams (e.g. sands and gravels). Glacial deposits are divided into those that are typically sorted or structured deposits from melt water; and directly deposited non-sorted deposits. This simple classification can be misleading because directly deposited glacial soils can contain layers, lenses or pockets of stratified water-lain deposits if the glacier has passed over a deformable bed composed of water-lain deposits, which may or may not be a glacial deposit.

It must be noted that glaciers can move considerable distances and advance and re-advance during cold periods and retreat (or melt) during warm periods. It is estimated that the UK experienced up to 16 periods of glaciation during the Pleistocene era (approximately the last two million years) (DoE, 1994). The consequence of this is that glacial soils may have moved several times and been deposited in different forms. This contributes to the complex deposits of spatially variable glacial soils. It also implies that a simple classification of glacial soils may not be possible.

Consider a glacier moving across a rock surface (**Figure 31.5**). The rock is eroded creating a mixture of boulders, gravels, sands and rock flour. This material may be moved a short distance and be deposited at the base of the glacier. As the glacier continues to advance abrasion of the sub-glacial layer continues, leading to an unsorted mix which, when deposited, is known as lodgement till. Some of the sub-glacial material will migrate into the glacier. This can either be deposited as melt-out till or melt-water deposits as the glacier retreats. The next time the glacier advances it advances over till. Initially the till will deform, the amount of deformation depending on the distance moved, the pressures and the temperature profile. This till now becomes deformation till, using the definition that it is a sub-glacial material that has not moved very far before being deposited. Therefore a deformation till could contain traces of previous glacial deposits (**Figures 31.3** and **31.5**). Further movement can increase the amount of abrasion and crushing, leading to a lodgement till. This is why it is not unexpected to find lenses of water-lain deposits of laminated clays and sands and gravels within what appears to be a till. It explains why rock particles can be found many miles from their original source and still be intact and why rock particles from one source can be widely distributed. It also shows why

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Construction process	Hazard	Description
Investigation	Strength	Some tills are so strong that it is difficult to obtain undisturbed samples. The fabric of till can have a significant effect on the strength determined from routine tests. Empirical correlations developed for stiff clays may not apply to basal tills. Boreholes may not be positioned to pass through zones of weaker materials.
	Permeability	*In situ* permeability is often greater than that measured in the laboratory because of fissures and laminations.
	Boulders	This could lead to false identification of rock head from borehole records.
	Laminated clay	It will be difficult to identify whether this is a pocket or layer of laminated clay from borehole records.
	Sand and gravel	It will be difficult to identify whether this is a pocket or layer of sand and gravel from borehole records.
Excavation	Strength	Some glacial soils, particularly basal tills, are very dense and hence difficult to excavate; explosives and road picks have been used. Basal tills can contain pockets of more ductile soil which can influence the stability of excavations.
	Boulders	Some glacial soils may contain boulders which can vary in size up to rafts of rock. These could be an issue for excavations for piles, tunnels or diaphragm walls.
	Boulder beds	These can occur between different layers of tills or within melt-out tills and basal till. Their presence will impact on any form of excavation.
	Laminated clay	Deformation till formed from glaciolacustrine deposits can include pockets and lenses or layers of laminated clays. These may be softer than the surrounding till, leading to instability.
	Sand and gravel	Glaciolacustrine deposits and deformation till formed from glaciolacustrine deposits can include pockets and lenses or layers of sands and gravels. These could be water-bearing and, if connected to an aquifer or source of water, lead to flooding.
	Fissures	Many basal tills are fissured as a result of depositional processes. This can lead to block failure and the development of shear zones.
Design	Strength	The *in situ* undrained shear strength of some basal tills exceeds that measured in the laboratory. This leads to overdesigned foundations.
	Stiffness	The stiffness of some basal tills is such that settlement calculations overpredict the amount of settlement. The stiffness of some tills is such that the pore water response is lower than expected.
	Boulders	These are more of an issue for excavation, but the presence of a boulder could lead to an unsafe estimate of performance of a test pile.
	Boulder beds	This could lead to differential movements of a group of end-bearing piles.
	Laminated clay	Pockets of laminated clay could lead to differential movement of a group of piles. Laminated clay beneath the base of piles could lead to excessive settlement. Laminated clay over the pile length could reduce the capacity. Laminated clay layers could trigger slope failures in the longer term.
	Sand and gravel	Pockets of sand and gravel could lead to differential movement of a group of piles.

Table 31.1 Examples of construction problems associated with glacial soils

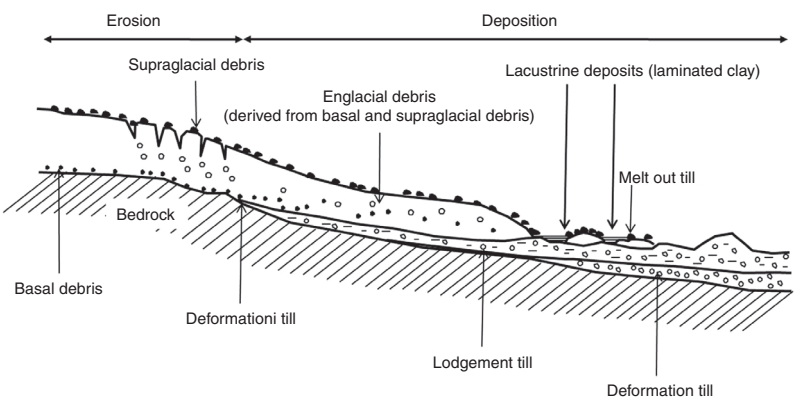


Figure 31.2 The formation of glacial soils
Reproduced from Hambrey (1994); UCL Press

Type of debris	Deposition Environment	Type of glacial soil	Description
Supraglacial			Debris that is carried on the surface of a glacier; derived from rock falls and, possibly, from the glacier bed
Englacial			Debris that is found within a glacier either derived from supraglacial or sub-glacial material
Sub-Glacial			Debris that is deposited from the base of a glacier
	Glacioterrestial		Basal deposits on land
		Melt-out till	Deposited by melting of stagnant or slowly moving debris-rich glacier ice without subsequent transport or deformation
		Flow till	Deposits that flow from the margin of a glacier
		Lodgement till	Deposited by spreading under pressure glacial debris from a sliding glacier bed
		Deformation till	Sediment which has been disaggregated and (possibly) homogenised by shearing in the subglacial deformed layer
	Glaciofluvial		Sand and gravel deposited from water from a melting glacier
		Ice marginal	Adjacent to the glacier
		Proglacial	Deposited in front of or just beyond the outer limits of a glacier or ice sheet
		Sub-glacial	Deposited beneath the glacier
	Glaciolacustrine		Laminated clays found in ice dammed lakes
		Ice contact	Deposited adjacent to the ice margin
		Distal	Deposited some distance from the glacier
	Glaciomarine		Glacial deposits found within a marine environment
		Fjord	Deposited within a fjord
		Continental	Deposited on the continental shelf
		Waterlain	Deposited by water
		Proximal	Deposited near to the glacier
		Distal	Deposited some distance from the glacier

Table 31.2 Key descriptions of glacial materials

it is difficult to be certain that an unsorted soil of glacial origin can be classified as a lodgement till.

There is a genetic classification of tills (**Figure 31.4** and **Table 31.3**) based on those produced after one cycle of advance and retreat of a glacier. This classification may not apply in practice because of the cycles of advance and retreat of glaciers leading to glacial deposits being transported and deposited several times. In addition to the glacial action, post-glacial processes can also alter the properties of glacial soils both between movements of a glacier and since the last glaciation. This adds a further complication when interpreting glacial soil profiles. For example, in northern England it is not uncommon to find three layers of glacial soil: an upper and lower red till separated by a thin layer of sand and gravel or laminated clay from a lower grey till (**Figure 31.6**). It is commonly stated that this succession was originally deposited as a grey lodgement till by a single ice sheet advance, and that the reddish colour of the upper till is solely due to post-glacial weathering (Eyles and Sladen, 1981; Lunn, 1995). There are, however, some aspects of the composition of the glacial succession that do not seem to be adequately explained by this single deposit and weathering concept. These include the red (upper) till generally having a much lower stone content (gravel, cobbles, boulders) than the grey (lower) till (Beaumont, 1968) and the frequency and extent of the sand/gravel laminated clay layers, which in some cases are well over a square kilometre in area, are very probably pro-glacial (glaciolacustrine/glaciofluvial) deposits and therefore indicate that the ice front was receding and releasing debris-laden melt water before the area was again overlain by later ice and ice-transported deposits (Hughes *et al.*, 1998). This demonstrates the difficulty of producing a simple geotechnical model from a complex deposition process which is not fully understood; although a number of geological and hydrogeological domains are distinguishable based on field characteristics (e.g. McMillan *et al.*, 2000 for further details).

The consequences of a failure to understand the geological model can be dramatic, as highlighted in **Table 31.1**. The sample descriptions leading to the soil classification do not

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Criterion	Lodgement till	Melt-out till	Flow till	Deformation till
Deposition	Deposited by plastering of glacial debris from the sliding base of a moving glacier, by pressure melting and/or other mechanical processes (Hambrey, 1994)	Deposited by a slow release of glacial debris from ice, neither sliding nor deforming internally (Dreimanis, 1988)	Deposition accomplished by gravitational slope processes and may occur supraglacially, sub-glacially or at the ice-margin (Dreimanis, 1988)	Comprises rock or unconsolidated sediment detached by the glacier from its source; primary sedimentary structures distorted or destroyed and some foreign material admixed (Elson, 1988)
Position and sequence	Lodged over older glacial sediments or on bedrock	Usually deposited during glacial retreat	Most commonly the uppermost glacigenic deposit	Formed and deposited sub-glacially, often where the glacier moves upslope
Basal contact	Lodgement and melt-out tills formed and deposited at glacial base. Contact with the substratum (bedrock or unconsolidated sediments) generally erosional and sharp Glacial erosion-marks and clast alignment have same orientation. Supraglacial melt-out tills may have variable basal contact		Variable basal contact but seldom conformable over longer distances. Tills may fill shallow channels or depressions	Variable basal contact
Landforms	Mainly ground moraines, drumlins, flutes and other sub-glacial landforms	Those ice-marginal landforms where glacier ice stagnated	Associated with most ice-marginal landforms	Landforms rarely diagnostic
Thickness	Typically one to a few metres thick but may attain substantial thickness in the English lowlands; relative lateral consistency	Single units usually a few centimetres to a few metres thick. Units may stack to much greater accumulated thickness	Very variable. Individual flows usually a few tens of centimetres to metres thick. Units may stack to accumulated thickness of many metres	Varies up to many metres depending upon nature of glacier bed
Structure	Usually massive but may contain various consistently oriented macro- and microstructures. Sub-horizontal jointing common and vertical and transverse joints may also be present. Orientation of deformation structures related to stress applied by moving glacier and may be laterally consistent	Either massive, or with faint structures partially preserved from debris stratification in basal debris-rich ice. Loss of volume with melting leads to draping of sorted sediments over large clasts	Either massive or displaying various flow structures depending on type of flow and water content	Primary structure may be preserved but usually deformed, especially in upper part of the sequence which may blend into other massive tills
Grain size composition	Abrasion in traction zone during lodgement produces silt-size particles typical of lodgement tills. Most have relatively consistent grain-size composition except for the basal part which may contain boulders of local glacier bed	Winnowing of silt and clay-size particles occurs during melt-out. Some particle size variability inherited from debris bands in ice. Supraglacial melt-out tills of valley glaciers contain characteristic coarse-grained debris	Usually diamicton with polymodal particle size distribution. Some particle size redistribution and sorting may occur during flow. Inverse or normal grading may develop	Deformation tills derived from weak rocks contain clasts separated by minor amounts of finer matrix. Clast size reflects bedding thickness of original material
Lithology of clasts and matrix	Lithological composition often more consistent than other tills. Composition of matrix particularly uniform. Materials of local derivation increase in abundance towards basal contact	Supraglacial melt-out till more variable in composition with increased possibility of exotic material	Lithological composition generally same as source material. May include incorporated glacier bed or exotic materials depending on debris source, transport and deposition	Deformation tills generally have same lithological composition as underlying sediments. Occasional erratics present particularly in upper part of the sequence

Table 31.3 (continued)

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Criterion	Lodgement till	Melt-out till	Flow till	Deformation till
Clast shapes and their surface marks	Sub-angular to sub-rounded clasts. Bullet-shaped, faceted, crushed, sheared and streaked-out clasts more common in lodgement than other tills. Lodged clasts striated parallel to direction of the lodging movement	Variable degree of roundness but angular clasts occur where supraglacial melt-out debris is englacially or supraglacially derived	If present, soft sediment clasts may be rounded or deformed by shear. More resistant rock clasts will retain their original shape	Clast shape and surface marks generally inherited from original material and not diagnostic. Clasts generally transported passively and not significantly modified
Fabric	Strong macro fabric with clast long axes parallel to local direction of movement. Transverse orientation possible, associated with folding and shearing	Fabric inherited from glacier transport. Melt-out process may weaken fabric, particularly micro-fabric	Fabric may be random or strongly developed and parallel or transverse to flow direction. Fabric may vary laterally over short distances	Preferred orientation rare and generally reflects shearing deformation
Consolidation	Consolidation process depends on pore pressure regime, temperature profile and permeability of glacier bed. Tills can be 'lightly overconsolidated' to 'heavily overconsolidated'	Lightly overconsolidated	Usually normally consolidated	Variably consolidated
Density	Very dense, often in excess of gravitationally compacted stiff clays due to combination of normal and shear stress during deposition	Bulk density lower and more variable than lodgement till	Density lower than lodgement tills and typical of normally consolidated deposits	Spatially variable densities due to presence of lower density pockets of material and dilatency during deformation
Strength	Very strong due to high densities. Strengths can approach those of weak rock in some cases	Strength lower and more variable than lodgement till	Strengths typical of normally consolidated deposits	Spatially variable strength due to presence of lower density pockets of material and dilatency during deformation
Permeability	Highly impermeable if a clay matrix-dominant till	Permeability variable due to mix of particle sizes	Relatively permeable compared to clay matrix-dominant lodgement tills	Spatially variable strength due to presence of lower density pockets of material and dilatency during deformation
Relevant references for summaries of diagnostic properties	Goldthwait (1971), Boulton (1976b), Dreimanis (1976), Boulton and Deynoux (1981), McGown and Derbyshire (1977), Eyles *et al.* (1982), Clarke and Chen (1997), Clarke *et al.* (1998), Clarke *et al.* (2008)	Boulton (1976b), Dreimanis (1976), McGown and Derbyshire (1977), Lawson (1979), Boulton and Deynoux (1981), Shaw (1985)	Boulton (1976b), Lawson (1979, 1982), Boulton and Deynoux (1981), Lutenegger *et al.* (1983), Gravenor *et al.* (1984), Rappol (1985), Drewry (1986)	Elson (1988), Boulton (1979), Clarke *et al.* (2008)

Table 31.3 Characteristics of genetic till types
Data taken from Trenter (1999) (CIRIA C504), Hambrey (1994), Dreimanis (1988) and Elson (1988)

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necessarily describe the macro and micro behaviour of the soil mass. Laminated lenses or layers may trigger failures in excavated slopes; lenses of sand and gravels can lead to local collapses in excavations and, importantly, in open hole excavations; layers of water-bearing sand and gravel can flood excavations; boulders can impede piling and tunnelling operations; failure to correctly identify rock head can affect the design and construction of end-bearing piles and tunnels along the rock head elevation.

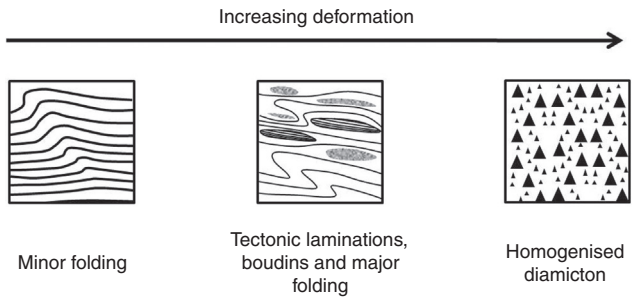


Figure 31.3 Glaciotectonic features found in deformation tills

31.3 Features of glacial soils

A genetic classification of glacial soils is based on the deposition process (**Figure 31.4**). The source of glacial material controls the lithology of the particles and, possibly, the grain size distribution and particle morphology (e.g. Benn and Evans, 1998). The mode of transport effects the abrasion process and hence the morphology, grain size distribution, fabric and structure. The deposition process affects the geological and geotechnical profile. Lawson (1981) defined till as a sediment deposited directly by glaciers which is a primary deposit including lodgement till, deformation till, communition

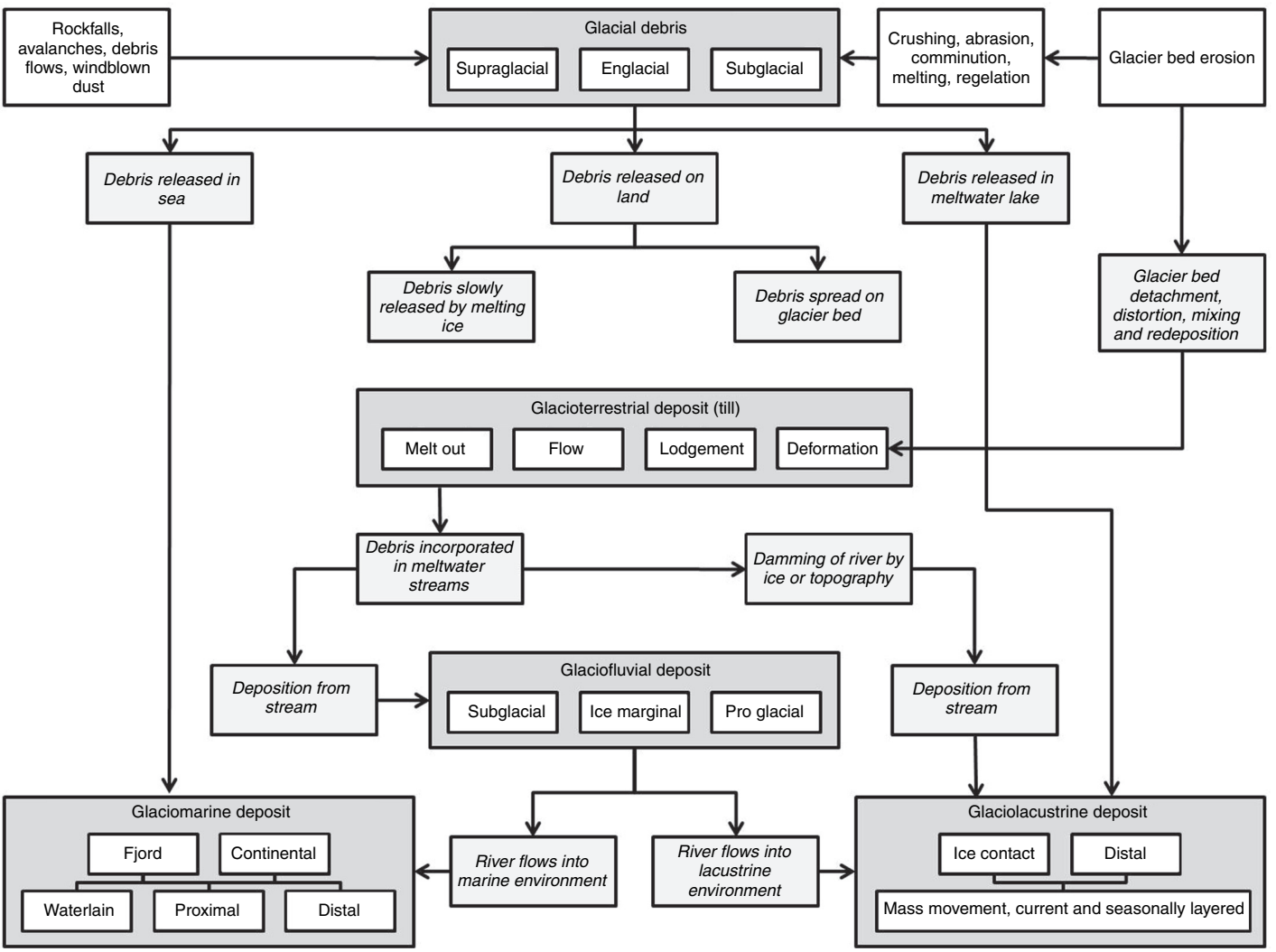


Figure 31.4 The transport and deposition processes for glacial soils leading to a genetic classification. Table 31.2 lists definitions of the key terms
Reproduced with permission from CIRIA C504, Trenter (1999), www.ciria.org

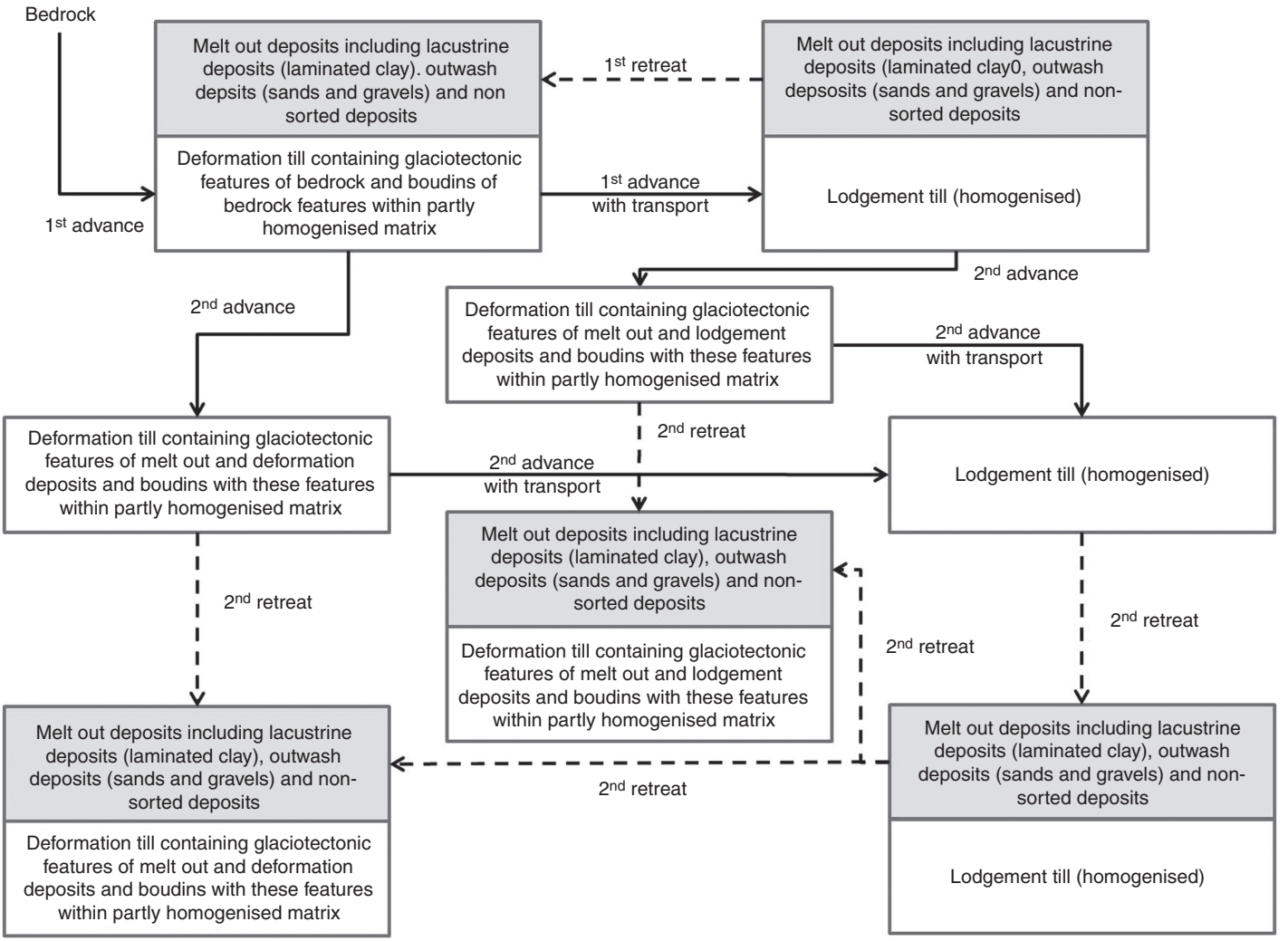


Figure 31.5 The processes at the base of a glacier showing how reworking of a glacial landscape can incorporate features of glacial soils in subsequent glacial deposits

till (formed by sub-glacial grinding or crushing) and melt-out till. There are a number of types of failures of tills which occur because of their fabric (**Table 31.1**). In this case fabric refers to depositional and post-depositional processes (Trenter, 1999) which create the clasts, layers, lenses, fissures, cracks and joints found in tills. The risks associated with these features can be substantially reduced by building up a ground model of local glacial geology from other investigations and observations of excavations. Fookes (1997) describes the total geological model, which is based on the principle that you start by creating a geological model of the site from known regional and local information, and that preliminary model drives the design of the ground investigation which is designed to confirm the model, fill in the gaps in the model and produce data to characterise the ground and produce design parameters.

31.3.1 Deformation till

Deformation till is formed of soil, which may or may not be glacial in origin. Ice moving across this soil will start to deform the soil, initially leading to glaciotectonic features of faults and folds, sometimes separately referred to as glaciotectonite. As the ice continues to move, these features will be further distorted. Folded material (**Figures 31.3** and **31.5**) will become detached to create pockets of material within the till. These, known as boudins, will become increasingly drawn out, forming tectonic laminations and eventually homogeneous till.

Deformation tills that have been deposited by a glacier passing over rock head will contain intact rock fragments, which can be extensive, leading to the possibility that rock head may be incorrectly identified. The lateral extent of rock encountered in a borehole is unknown, but observations from excavations indicate that the pieces of rock can vary in size from

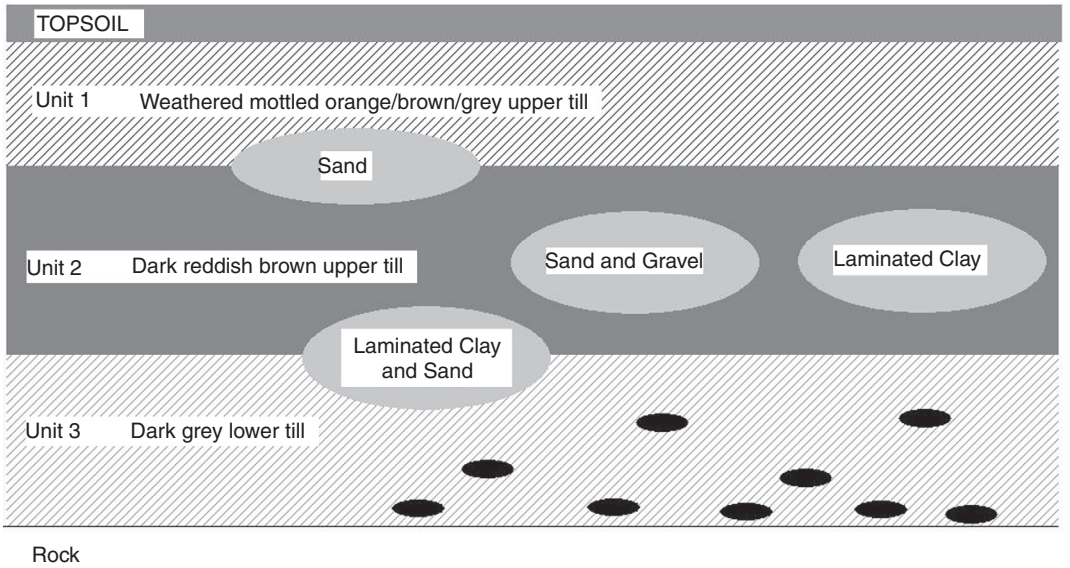


Figure 31.6 A tripartite succession found in the north of England showing three units of till separated by thin layers of laminated clays and sands and gravels
Reproduced from Clarke *et al.* (2008)

boulders to extensive rafts of rock. Therefore it is important to prove rock head. These boulders and rafts of rock can present problems during excavation. It is very difficult to indicate how many boulders will be present from borehole records. It is prudent to assume that till contains boulders even if none are encountered in the boreholes.

Deformation tills that have been deposited by a glacier passing over glacial soils will contain remnants of those soils often within a matrix of gravel, sands, silts clay and rock flour. These could include intact rock fragments, lodgement till, melt-water tills including laminated clays, and lenses of sands and gravels. It may be difficult to identify whether they are isolated pockets of these materials or layers of materials. It is prudent to assume that glacial soils can contain pockets and lenses of more ductile material. Laminated clay pockets could lead to zones of weakness and potential failure, especially in excavations (see **Table 31.1**). Sand and gravel lenses could be water-bearing and, if interconnected, lead to potential flooding of excavations. These features are common, suggesting many tills are deformation tills derived from melt-out, deformation and lodgement tills.

Many basal tills (or sub-glacial tills) are deformation tills (e.g. Bennett and Glasser, 1996). Indeed the deformation process could be responsible for many of the hazards listed in **Table 31.1**. The fact that these tills are created from underlying soil, which may have been deposited by a previous glaciation, means that the composition of the till is the same as the underlying soil. This may make it difficult to identify the base of a deformation till. The till could contain glaciotectonic features, rafts of intact soil or rock that have not been subject to abrasion and a range of particle sizes and pockets of intact, possibly distorted, material such as laminated clay as well as sands and gravels.

Thus samples may not be representative of the soil mass, leading to uncertainty over the 3D geological model. There could be unidentified zones of weakness (e.g. laminated clay lenses) or misinterpreted zones of weakness. For example, failures in excavations in till have been triggered by lenses of laminated clay – especially where they dipped into the excavation. There could be water-bearing layers. For example, a piling contractor took the view that the soil description referred to a clay and therefore open boreholes could be used. Unfortunately one borehole encountered a sand layer which proved to be continuous across the site and possibly connected to a nearby river. This led to all boreholes being fully cased. Proving rock head is critical, especially when considering end bearing piles or tunnelling operations.

31.3.2 Lodgement till

This is a sub-glacial or basal till, which is deposited as ice advances because the frictional resistance between the till and the bed is greater than the shear introduced by the moving ice. There are two theories of deposition: one related to effective stress at the bed of the glacier (Boulton, 1974) and the other (Hallett, 1979) in which the contact pressure between the glacial soil and bed is independent of the effective stress. In the effective-stress model, the stress is created by the weight of ice and the water pressure. The stress will depend on the temperature at the base of the glacier and the hydrological characteristics of the bed. The effective stress will increase as the permeability of the bed below a warm-based glacier increases. This means abrasion will increase if the debris continues to

move. However, the friction between the debris and the bed will also increase, which could lead to the deposition of the debris due to lodgement. If the bed is less permeable, then the effective stress will not be as great. If the bed of the glacier is frozen, the effective and total stress will be similar. The debris can be deposited at any time, depending on the friction.

The alternative view (Hallett, 1979) is that the debris is carried by the ice. Abrasion occurs as the debris moves across the bed. The debris will be deposited as the ice slows down or melts. Both of these theories are correct at some time as they depend on the depth of ice, the speed of ice movement and the temperature profile (**Figure 31.7**).

Lodgement tills are generally a very dense mixture of sands, silts and clays or rock flour and could be considered a relatively uniform deposit. Often they are a clay matrix till containing gravel, which typically has rounded edges, striated faces and is aligned with the direction of flow. The particle size distribution can be uniform or gap graded. Shear planes are evident.

Lodgement tills are difficult to sample because of their density and presence of gravel. Thick-walled sampling tubes are commonly use, which can lead to sample disturbance. For the same reasons it is difficult to sub-sample these tills, hence strength tests are usually based on 100 mm samples. *In situ* strength can exceed laboratory-measured strengths. This has affected tunnelling operations because the assumed strength was less than the *in situ* strength. These tills are described as clays and therefore there is an assumption that they have an undrained shear strength. Designs based on that strength are often conservative because the mobilised strength is much greater. These tills have been highly remoulded and can include shear planes, which means that their expected behaviour based on classic soil mechanics theories and the interpretation of standard laboratory tests is not experienced. The tills are denser, stronger and stiffer *in situ* than predicted from routine site investigations. This does lead to conservative designs, but given the uncertainties over the 3D model from a site investigation that is acceptable unless use can be made of regional knowledge.

31.3.3 Sub-glacial melt-out till

This till comprises debris deposited as a glacier melts primarily due to geothermal heat. There may also be sediment flow features. A key difference between this till and lodgement till is that this till is likely to be less dense as the till was lowered into place rather than spread into place. These tills are characterised by their variability leading to difficulty in selecting characteristic design parameters.

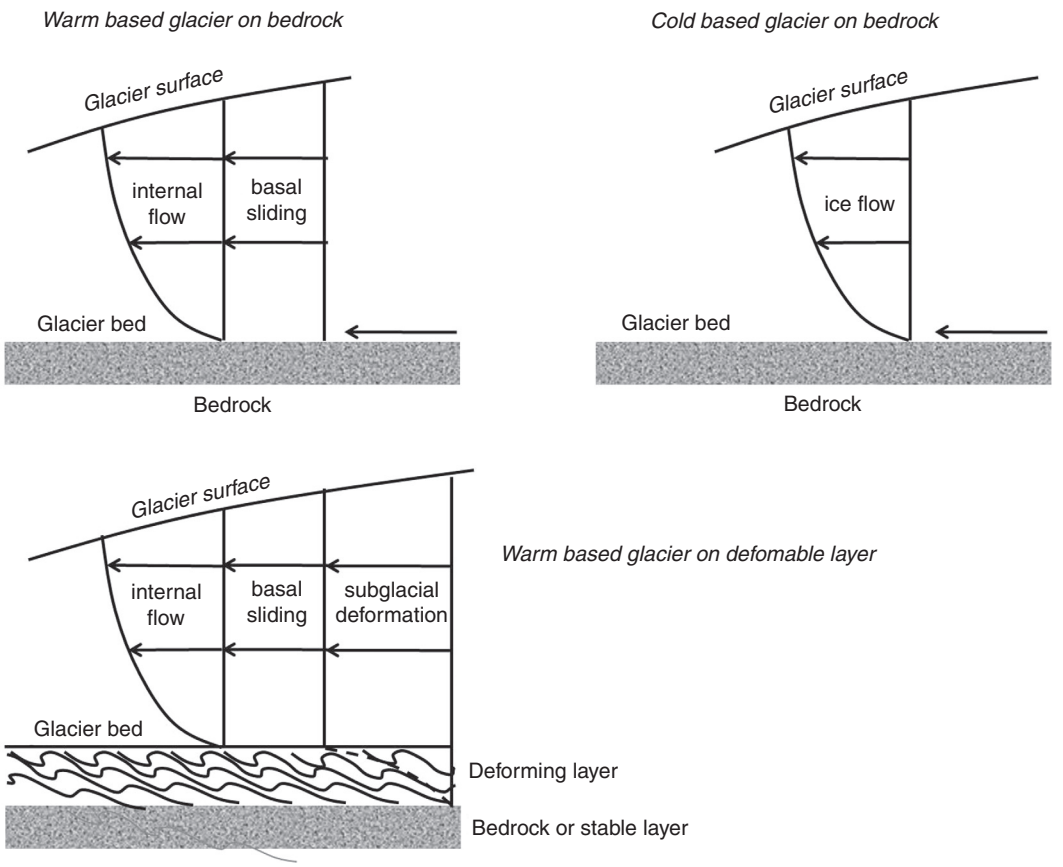


Figure 31.7 The relationship between the temperature profile and type of bed and the effect that they can have on the depositional process. The contours within the ice mass refer to the relative deformation of the ice
Reproduced from Boulton (1976); The Royal Society of Canada

31.3.4 Supraglacial melt-out till

This till is formed as the surface of a glacier melts. It is typically a loose, coarse, angular deposit that has been reworked because of movements of debris associated with flow and topography. As the ice melts releasing the supraglacial and englacial debris, the debris can insulate the underlying ice leading to an irregular topography. In this case, the debris will eventually be deposited as an ablation till creating a landform of moraines (mounds or ridges) of various types including lateral moraines formed at the margin of the ice, hummocky moraines deposited as the ice retreats and end moraines formed at the front of a glacier. If the debris layer is thin, then the ice melts uniformly leaving a thin layer of coarse material. These layers could include outwash sediments. The total extent of supraglacial till is significantly less than that of sub-glacial tills because it is deposited only when the ice melts.

31.3.5 Macro features of tills

The periods of glaciation sometimes lead to layers of different tills separated by a thin layer of sands and gravels or laminated clay. The upper tills are variously described as a distinctly different lodgement till to the lower layer or a weathered version of the lower layer. They are often of a different colour, supporting the weathering hypothesis, though the alternative is that the source of the till is different.

Deformation and lodgement tills often contain shear planes (e.g. Boulton, 1970) with associated conjugate shears because of the shearing that takes place as the tills are moved. These planes can lead to local failures and block failures in excavations. Both deformation and lodgement tills are very dense and brittle. It is difficult to recreate this density in soil mechanics laboratories due to the shearing process linked to the pressure of the weight of ice. The shearing process also means that these tills are highly remoulded implying there is little loss of strength on excavation. It is not unusual to find natural slopes in tills of 45º (Vaughan, 1994) even though the laboratory tests indicate lower strengths and hence lower slopes. The pockets and lenses within a till are likely to be less dense and more ductile.

Laminated clays, a feature of glacial lakes, are generally composed of thin alternative layers of clay and silt creating a highly anisotropic material in terms of stiffness, strength and permeability. Layers of laminated clays are often found in basal tills. It is not uncommon for failures to occur in excavations of these basal tills if laminated clays are present.

31.4 Geotechnical classification

As with all soils the geological processes govern the geotechnical properties. Water-lain glacial soils produced at the end of the glacial period and not subject to further glacial erosion will behave in a similar manner to other water-lain soils. However, the process of deposition may create an anisotropic structure (e.g. laminated clay) or a spatially variable structure (e.g. outwash deposits primarily of coarser materials). Supraglacial tills are gravitationally deposited but not through water. Therefore they could have similar properties, but not features, to water-lain deposits.

Sub-glacial or basal tills (i.e. deformation and lodgement) are either deformed or 'smeared' into place under great pressure due to the weight of ice. These are the predominant till types. For example, Eyles (1979) suggested that Icelandic glaciers produce between 1% and 7% of supraglacial till compared to basal till. This may take place under frozen or unfrozen conditions. In either case it should result in a very dense material. This density is not just a result of pressure but is a result of a combination of pressure and shear. Basal tills include those that are essentially deformed bed layers, which are likely to contain glaciotectonic features such as faults and folds with pre-existing features being readily identified (see **Figure 31.3**). These dense tills can contain pockets of less dense material. These pockets of material retain their density because of the fact that the till was frozen or unfrozen. If unfrozen the density of the surrounding till was such that excess pore waters could not dissipate. This can also occur because the structural form of the inclusion meant that the surrounding till provided a structural seal to the pocket of material. These pockets of less dense material, often remnants of melt-out tills, can be a hazard (see **Table 31.1**).

Sub-glacial tills may be classed as lodgement tills based on their fabric but in fact because of the periods of glaciation many could be considered deformation tills: i.e. tills that retain much of the original features of the bed, having been transported a short distance with little abrasion. Therefore the genetic classification may not be the best indicator of geotechnical properties. Meer *et al*. (2003) came to the conclusion that there is only one type of basal till, deformation till, since all basal tills contain deformation features.

It is possible to create a classification of glacial soils based on their geotechnical performance. This classification combines the genetic classification with observations of glacial soils to highlight the features that will impact on the interpretation of ground investigation and the behaviour of the tills during construction and in service.

McGown and Derbyshire (1977) suggested that it is more appropriate to use a classification that is based on superimposing the dominant soil fraction on the genetic scheme (**Table 31.4**). The classification suggests that the dominant soil fraction and the type of till govern the relative value of the engineering properties. This qualitative assessment shows that the density and compressibility decrease as the dominant soil fraction size increases; i.e. a clay matrix lodgement till is likely to be the densest and stiffest of the tills. This is not surprising given the amount of shearing and abrasion that takes place during transportation. A feature of all these tills is the fact they can be lightly or normally consolidated despite the pressures involved during deposition and the relief of those pressures due to ice melt. The link between consolidation theory and the mechanical characteristics of soils developed for gravitationally sedimented soils

Till	Dominant soil fraction	Fabric features	Relative scales (1 (low) to 9 (high)) Density	Compressibility	Permeability	Anisotropy
Lodgement till	G	MACRO: Interlayering of glaciofluvials, joints, fissures, contortions. Consistent preferred clast orientation.	4–7	1	5–6	7
	W	MESO: Fissuring. Contortion. Moderate to very high consistency of preferred orientation of clasts.	5–8	2	2–3	
	Mg	MICRO: Moderate to high degree of parallelism of fines in sympathy with clast surfaces	6–8	2	4–5	
	Mc		6–8	3	2	
Melt-out till	G	MACRO: Occasional interlayering with glaciofluvials. Clast preferred orientation often retained from englacial state.	2–4	2–4	7–9	3–5
	W	MESO: Moderate to high preservation of preferred clast orientation from englacial state, especially in sub-glacial type.	2–6	3–5	4–5	
	Mg	MICRO: Open to moderately closed arrangements of fines with many englacial arrangements retained, especially in sub-glacial type.	2–6	3–6	5–8	
	Mc		2–7	4–7	3–4	
Flow till	G	MACRO: Inter layering with glaciofluvials common. Segregation, contortions, layering and fissuring in upper section and nose of flow.	3	2	7	7
	W	MESO: Aligned low angle orientation of clasts conforming to flow direction rather than ice direction.	4	2–4	4	
	Mg	MICRO: Rather compact parallel arrangement of fines related to flow rather than direction of ice movement.	5	2–4	6	
	Mc		5	2–5	3	
Deformation till	G	TOTAL FABRIC: Deformed bedrock and soil (including previous glacial deposits) structures related to ice movement direction.	5–8	1–3	2–8	3–7
	W					
	Mg					
	Mc					

G = Granular or clastic till
Mg = Granular matrix till
W = Well graded till
Mc = Cohesive matrix till

Table 31.4 Characteristics and geotechnical properties of glacial tills
Data taken from McGown and Derbyshire (1977)

does not apply to tills. This means that very strong, stiff glacial soils are not necessarily heavily overconsolidated. In fact the behaviour of till often depends on the relationship between the fabric and the geotechnical structure, with the fabric having a significant impact on the performance of the geotechnical structure (see **Table 31.1**).

Consider an element of basal till. The total vertical stress on the element is equal to the pressure of the ice plus the pressure due to the overlying till. When the glacier retreats (melts) the total stress reduces to the pressure due to the overlying till. The pore water will depend on whether the basal till was frozen or unfrozen and the hydrological characteristics of the bed of the glacier. **Table 31.5** shows the possible pore water regimes. If

Bed of glacier	Base of till	Underlying soil/rock	Pore pressure**	Degree of overconsolidation
Frozen	Frozen*	Aquifer	No excess	Heavy
Frozen	Frozen*	No aquifer	Excess	Normal
Unfrozen	Frozen*	Aquifer		Heavy
Unfrozen	Frozen*	No aquifer		Normal
Unfrozen	Unfrozen	Aquifer	No excess	Heavy
Unfrozen	Unfrozen	No aquifer	Excess	Normal

*pore pressures generated as ice melts
**the total stress is equal to the pressure due to the weight of ice

Table 31.5 Pore water pressure regimes within till during deposition

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the bed is frozen, then the pore water pressure is zero. As the glacier melts pore water pressure will be generated due to the weight of till as the bed melts. The total stress reduces as the weight of ice reduces. Therefore the degree of consolidation is low. In the case of a bed that is not frozen and the bed acts as an aquifer, the degree of consolidation is high because the effective stress reduces due to the weight of ice. It is possible, therefore, for a till to be 'normally consolidated' or 'heavily overconsolidated' depending upon the pore water regime and underlying geological conditions.

The shearing, abrasion and crushing processes further complicate the stress regime. The concept of overconsolidation ratio (i.e. the ratio of the maximum past effective stress to the current effective stress) applied to gravitationally compacted sediments can be applied to tills but since the maximum stress, related to the thickness of ice, is unknown and the deposition process introduces shear the concept of the degree of consolidation as an indication of soil properties is not valid and therefore characteristics cannot be deduced from stress history. Indeed density and the dominant matrix may be a better indication of likely behaviour.

31.5 Geotechnical properties

31.5.1 Classification data

It is important with glacial soils to link the geological characteristics with the geotechnical classification to ensure that the soils are correctly identified and the appropriate properties assigned. For example, a liquid and plastic limit test on laminated clay formed of clay layers separated by silt layers will produce very different results from tests on the two layers forming the laminated clay. The test may suggest a silty clay when in fact it is a laminated clay. Indeed in this extreme case liquid and plastic limit tests may not be appropriate. Linking empirical relationships developed for gravitationally compacted soils between liquid and plastic limit test results and mechanical properties is not advisable for tills.

Boulton and Paul (1976) showed that the plasticity indices for tills cluster around the T-line, a line parallel to the Cassagrande A-line. This may be due to the fact that the finer particles in tills comprise rock flour rather than clay minerals. This applies to many tills (**Figure 31.8**), which shows that tills vary from CL to CI in general. The limit tests are only carried out on the finer particles and therefore are only relevant to clay matrix dominated tills. Therefore one technique to identify tills is to assess the properties of the matrix. A clay matrix-dominant till is defined as well-graded till with a fines content of between 15% and 45% (McGown and Derbyshire, 1977). Barnes (1987) and Winter *et al.* (1998) showed that the matrix dry density reduced markedly when the stone content exceeded 40–50%. At that point a glacial soil is considered a granular soil and has the characteristics of such soils. Gens and Hight (1979) reported that tests on reconstituted soils are unaffected by the gravel content if the gravel content is less than 15%. Thus the grading curve is a useful indicator of characteristic behaviour.

Limit tests can be used to assess whether a till is weathered, since the weathering process increases the clay content due to the formation of clay minerals. It also increases the water content. Limit tests on weathered till still cluster around the T-line. Therefore it would be expected to find the limit test results to migrate down the T-line with depth because the amount of weathering reduces.

31.5.2 Particle size distribution

The wearing down of particles due to abrasion and crushing during the transportation process results in a broad range of particle sizes. The percentage of fine particles increases with distance from source. A feature of lodgement tills is that the distribution of particle sizes has fractal characteristics; i.e. the numbers of grains at each size when plotted on a double logarithmic plot are linearly related (**Figure 31.9**). Theoretically the slope of the line would be 2.58 (Sammis *et al.*, 1987) if the particles were produced solely by fracturing. In practice the slope is greater (Hooke and Iverson, 1995).

The particle size distribution in glacial soils can vary from pure clay to pure gravel. However, the majority of lodgement and deformation tills are clay matrix-dominant tills which sit between a silty clay (e.g. London Clay) and a sandy gravel (e.g. Thames Gravel) in terms of their grading (**Figure 31.10**). Note that boulders will not have been included in the grading curves, so the curves are not necessarily a true representation of the soil. The chart distinguishes between fine soils and sands, gravels and cobbles. Fine soils include clays, silts and rock flour, the rock flour being derived from the abrasive processes during deposition. Rock flour is clay and silt size but is neither clay nor silt; it is formed of particles of rock.

Clay-dominant sub-glacial tills are generally well graded. **Figure 31.11** shows a ternary diagram for the percentages of fines, sands and gravel (**Figure 31.11a**) and clay, silt and coarse particles (**Figure 31.11b**). Note that cobbles are removed either in the sampling process or in preparing the specimen. The gradings confirm that the tills are dominated by fine-grained particles. The averages show that the tills are distinctly different, as can be seen from **Table 31.3**.

31.5.3 Mechanical characteristics

31.5.3.1 Undrained shear strength

Sampling clay basal tills is difficult because of the stone content and the strength of the till. This leads to significant scatter in the results of strength and stiffness tests. It is for that reason that tests are often carried out on U100 samples, providing a single value of strength at the depth of the sample. It has been practice to carry out multi-stage tests on a single 100 mm sample (Anderson, 1974). The sample is taken to failure in the first stage. The same sample is taken to failure a second and third time at different confining pressures, with shear taking place on the

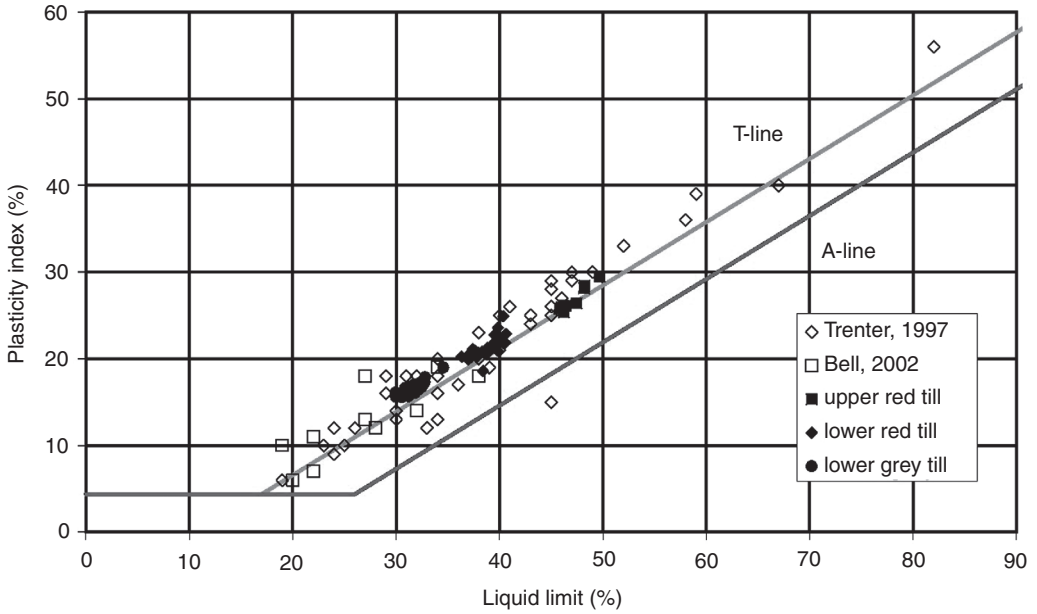


Figure 31.8 Typical Atterberg limits of glacial basal tills showing the tendency to cluster around the T-line

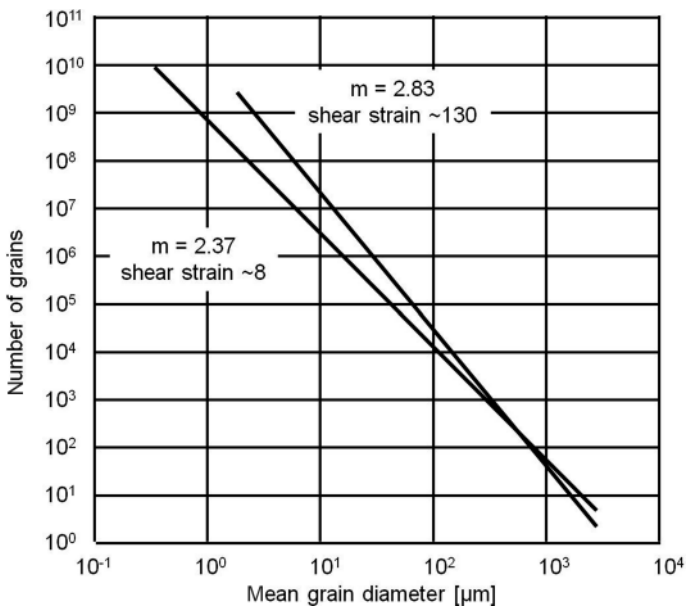


Figure 31.9 The fractal characteristic of lodgement tills
Reproduced from Iverson *et al.* (1996); International Glaciology Society

predetermined plane. Given the density of the till it is likely that it will exhibit brittle behaviour during the first stage, thus creating a predetermined failure plane for the subsequent stages. The implication is that the derived shear strength from a multi-stage test does not have any meaning. However, given the difficulty of preparing sub-samples from 100 mm diameter samples of basal tills because of the particle size distribution, in particular the gravel content, it is normal to test 100 mm samples. Given that there will be only one strength test from each sample it is prudent to increase the number of samples tested for strength. Much of the sample still exists after the strength test, so it is possible to use that sample for classification tests.

The measured undrained shear strength of clay matrix-dominant tills is very dependent on the fabric and method of sampling. The density of the till means that sample distortion using driven tubes is inevitable. Stones affect both the sampling and mode of failure. Fissures affect the mode of failure (**Figure 31.12**). McGown *et al.* (1977) suggested that a representative undrained strength can be obtained only from samples larger than those normally taken in ground investigations. This questions the value of U100 samples from ground investigations. Hence profiles of undrained shear strength in till are likely to show much scatter (**Figure 31.13**). McGown *et al.* (1977) used the fact that the relationship between intact strength and strength of fissured soil varied with sample size, leading to the conclusion that the representative strength is typically 0.75 of the average strength measured using U100 samples of fissured till.

In some cases, because of the composition and strength, it is difficult to obtain undisturbed specimens. Therefore an alternative method is used to indirectly measure undrained shear strength. For example, undrained strengths of some Irish tills (Farrell and Wall, 1990) are estimated from correlations between SPT blow counts and tests on remoulded specimens. A value of 6 ($s_u = 6N$) for Irish tills (Orr, 1993) is commonly used, but note that the correlation was based on tests on remoulded specimens. This empirical method is commonly used for gravitationally compacted soils. Stroud (1975) presented a correlation between undrained shear strength and blow count based on plasticity index for fissured clays. This

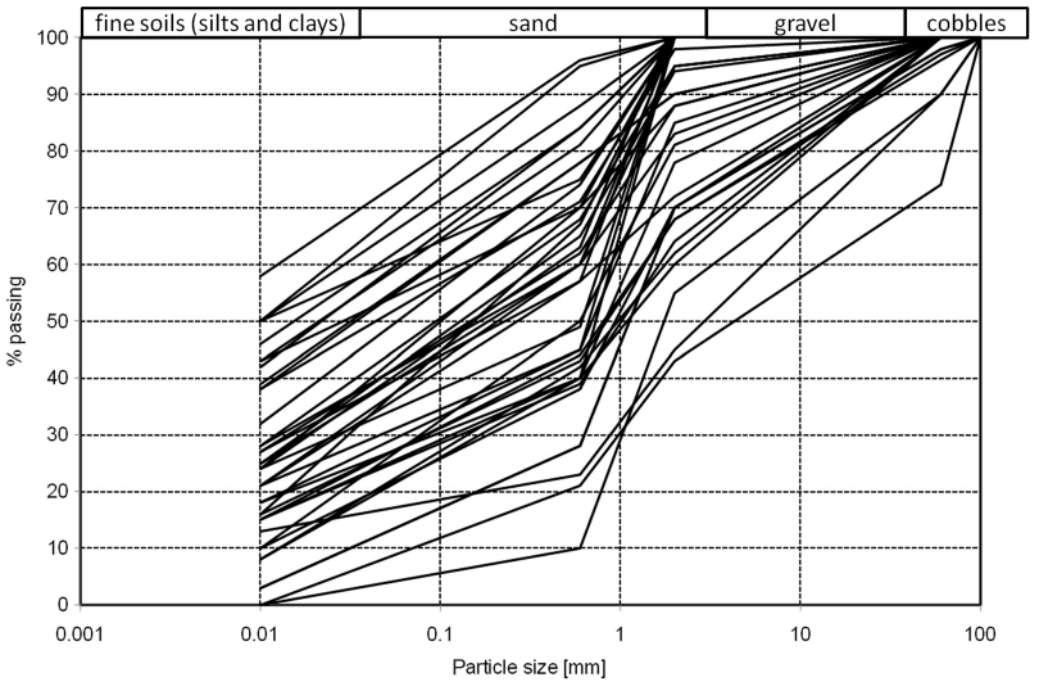


Figure 31.10 The grading of glacial tills; the data come from various sources from across the UK including those summarised by Trenter (1999)

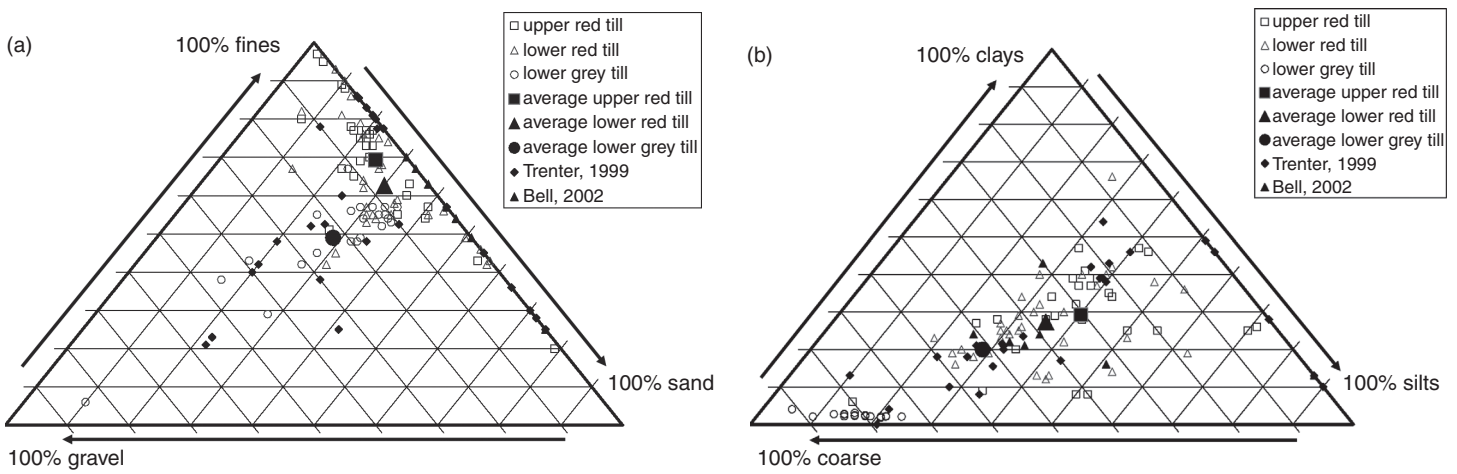


Figure 31.11 A ternary plot of a number of glacial basal tills showing (a) the relationship between fine and coarse particles and (b) between the fine particles

correlation used uncorrected values of SPT blow count. Ground investigations carried out in accordance with Eurocode 7 will provide corrected values.

Many tills are fissured so the correlation may apply to tills, but there is evidence that shear strengths estimated from SPT results using the Stroud correlation are significantly less than the *in situ* strength, suggesting the correlation factor is too low. This could be because the correlation has not been developed for such dense clays or it could be because the stiffness of these tills makes them behave in a partially drained manner. It is assumed that the clay tills can be considered as undrained clays when loaded because they are considered relatively impermeable. However, they are very stiff. This means that pore pressures generated during rapid loading (as in an undrained test) may be less than expected, since some of the load will be taken by the soil skeleton as water is compressible in reality. Further, *in situ* the pore pressures may dissipate more quickly than expected because the coefficient of consolidation is high. The coefficient of consolidation is the ratio of the stiffness to permeability. Given that the permeability of the basal till is similar to other clays but the stiffness can be much greater, the coefficient of consolidation is often much greater than that of sedimented

clays. Thus the dissipation of pore pressures could be much quicker than observed in sedimented clays. This may account for the apparently high undrained strengths in basal tills; in practice they are behaving in a partially drained manner.

The data used by Stroud, which covered plasticity indices between 15% and 65%, showed that the ratio between undrained shear strength and the SPT blow count varies between 7 and 4.1. **Figure 31.14** shows data from a site in northern England in which results of triaxial tests on undisturbed specimens are plotted against depth together with values of strength derived from SPT N values. There is evidence from piling contractors that quoted correlations between undrained shear strength and blow count are too low. This suggests that earthworks calculations should be based on a factor of 6 since the till is remoulded during construction, whereas foundation design calculations

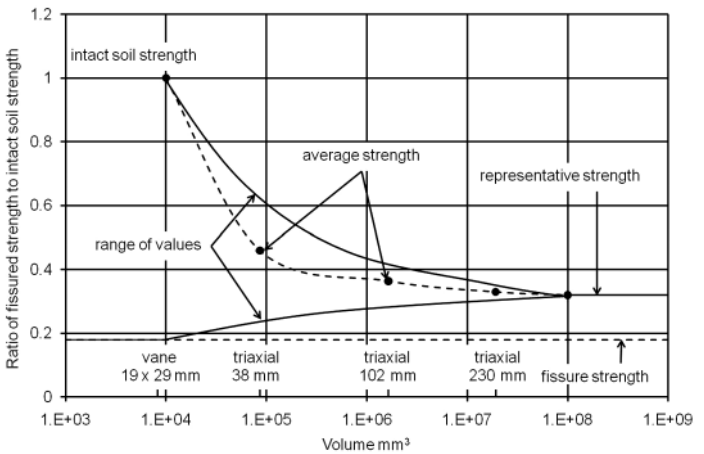


Figure 31.12 The effect of fissures on the strength of glacial till showing the ratio of strength of fissured strength to intact strength against the size of the sample expressed in terms of volume
Reproduced from McGown *et al.* (1977) © The Geological Society

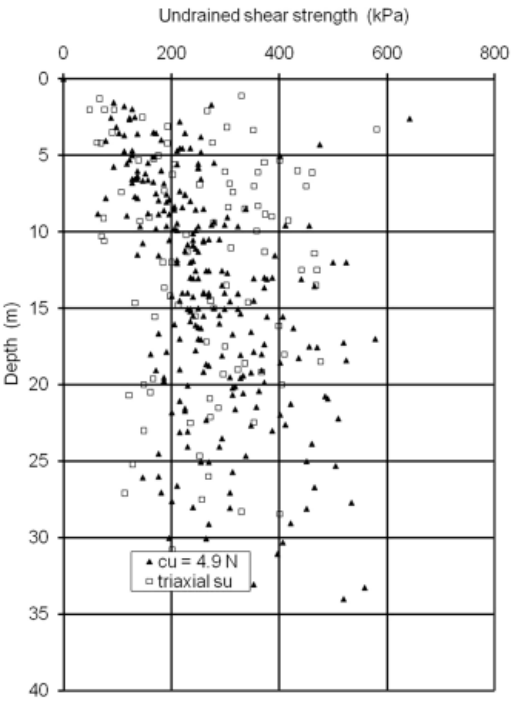


Figure 31.14 The relationship between SPT blow count and the triaxial undrained shear strength of some tills in northern England

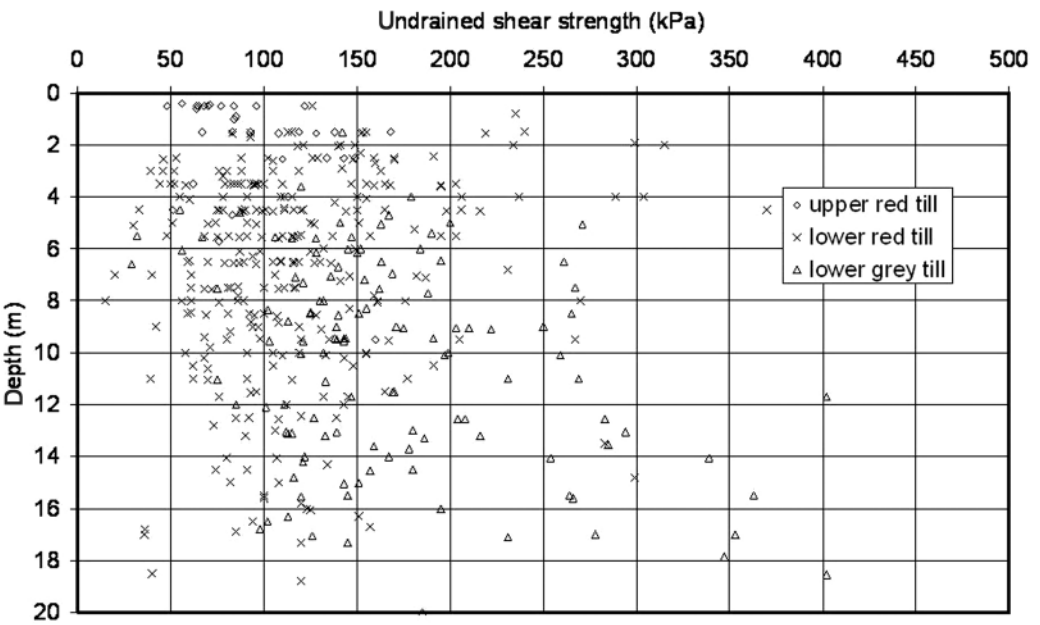


Figure 31.13 A typical profile of undrained shear strength of basal till highlighting the scatter in the data and the difficulty in selecting a characteristic value of strength (the upper and lower tills refer to the tripartite succession in northern England)

could be based on 9, since the *in situ* strength is mobilised during foundation loading. Note that sometimes 9 may be considered too low, but this should be challenged unless there is evidence from the investigation that supports this view.

The consequence of this response of basal clay tills is that foundations are often overdesigned and excavation techniques inappropriate. This reinforces the need for pile tests to assess the actual performance rather than the predicted performance.

31.5.3.2 Effective strength parameters

As with undrained shear strength, the fabric and composition of glacial soils have an effect on the interpreted values of c′ and ø′ It is important to stress that c′ and ø′ are based on the assumption that there is a linear relationship between confining pressure and deviator stress independent of the level of confining pressure and, for a given soil, independent of the variation in fabric between specimens. In tills, the latter assumption is invalid, resulting in a wide variation in values of c′ and ø′ for tills, even if they are from the same till deposit. Increasing the density of a till increases its strength and there is an optimum fines content for a maximum strength. Thus, density is an indication of strength. Hence, when interpreting laboratory test results, it is useful to disregard low values of strength from high density samples because the fabric may have been the dominant factor in determining the strength; i.e. failure may have been triggered by gravel inclusions or shear planes. This emphasises the need for quality descriptions of the samples tested and for caution when selecting characteristic strengths from a limited number of tests. The fabric may be the dominant factor that influences the mass behaviour and not the material strength.

There have been attempts to relate ø′ with PI (**Figure 31.15**), percentage fines and density (**Figure 31.16**) for clay matrix-dominant tills. It is clear that such relationships exist but they may not be appropriate for till because of the effects of fabric and sampling.

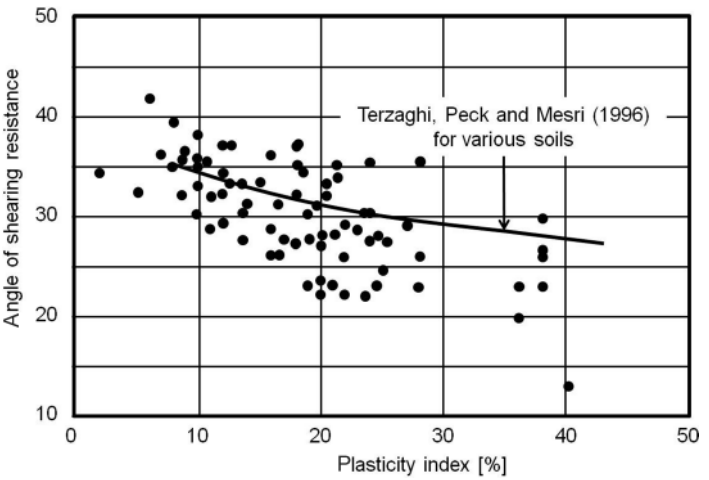


Figure 31.15 Relationship between angle of shearing resistance and plasticity index for a number of basal tills
Reproduced with permission from CIRIA C504, Trenter (1999), www.ciria.org

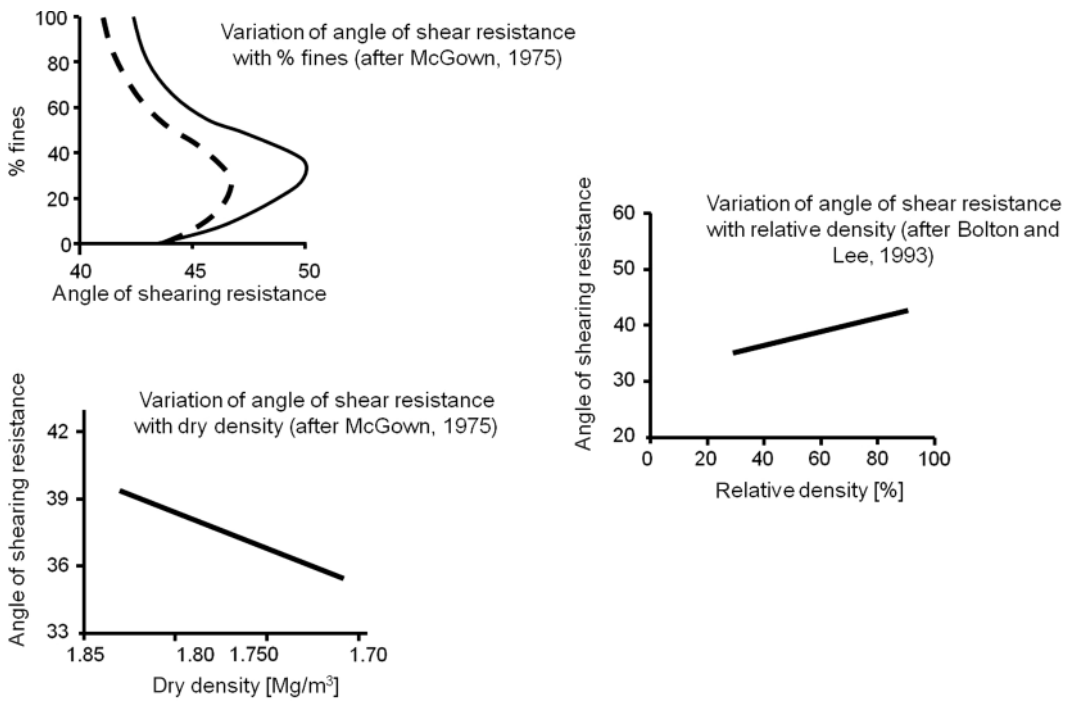


Figure 31.16 Relationship between percentage fines, density and angle of shearing resistance
Reproduced with permission from CIRIA C504 Trenter, (1999), www.ciria.org

There are two approaches that could be used to evaluate a characteristic strength. In both cases (**Figure 31.17**) the samples have to be grouped together according to their description and classification data, noting that two tills from the same site can have the same description but different liquid limits. This is because the source of the tills may be different resulting in different particle compositions. The first approach is to plot the deviator stress and mean stress for all samples on the same graph and identify the failure envelope from these data (**Figure 31.18**). Spurious data points related to density and fabric should be given a lower weighting. This gives the characteristic value of strength provided sufficient samples are tested.

The alternative way of producing a characteristic strength, especially when there are a limited number of samples, is to test reconstituted glacial till (Burland, 1990) with all coarse material removed. In this case, the laboratory prepared specimens are gravitationally compacted to as high a density as possible, ideally using a hydraulic press. The density will still be less than the *in situ* density so the test results could be considered a lower bound value. Basal tills were spread into place under vertical loads of up to 10 000 kPa; typical laboratory tests use pressures of up to 700 kPa. The scatter due to fabric is removed when testing reconstituted till (**Figure 31.19**) but the interpreted value of strength lies below the values obtained from tests on intact samples.

Note, however, in both cases the strength of the glacial till mass may be dominated by lenses or pockets of laminated clay and sand and gravel.

31.5.3.3 Stiffness

Stiffness of soils is notoriously difficult to measure, is test dependent and depends on the quality of the soil being tested. In tills this is further compounded by the fact that the tills are spatially variable to such an extent that measurements

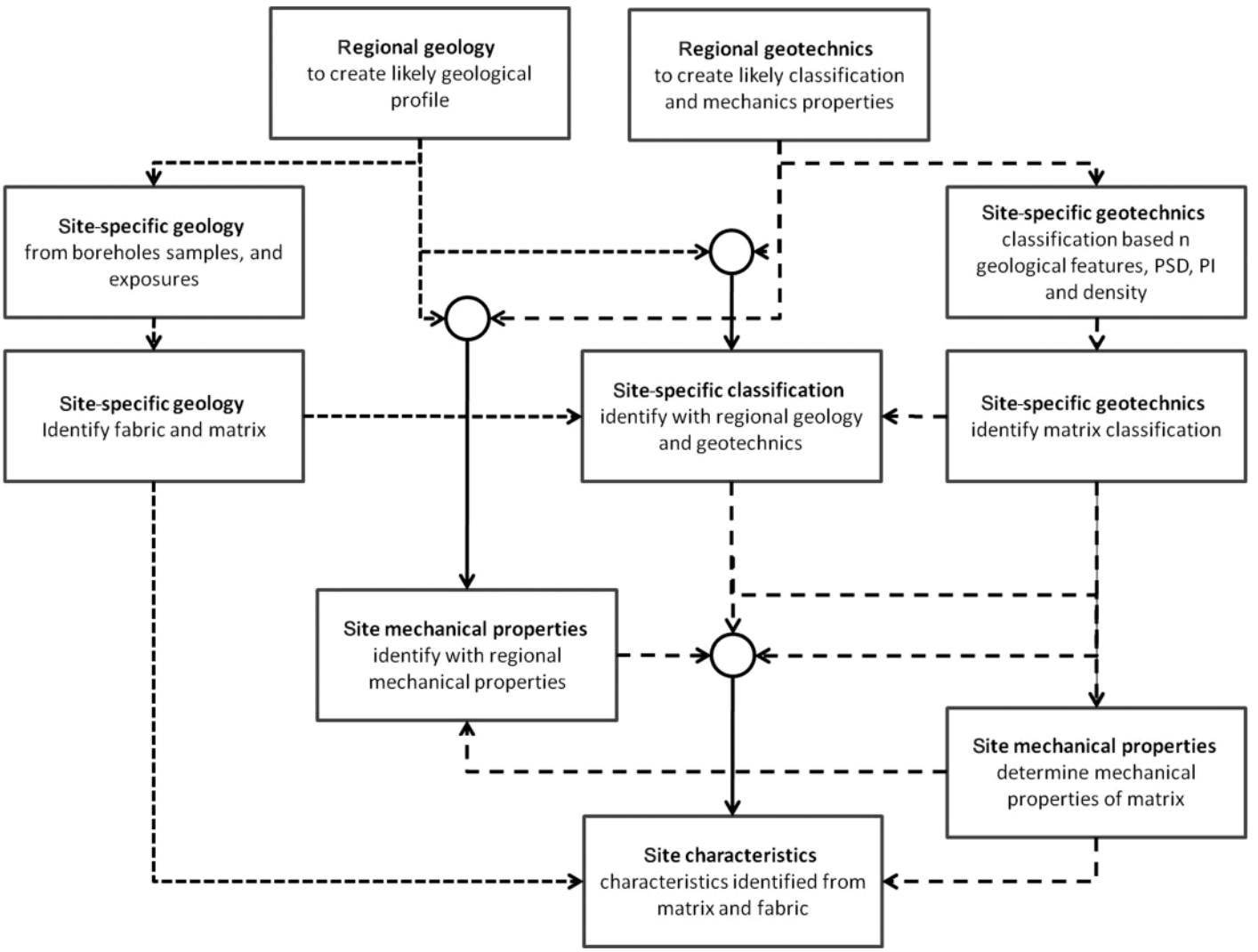


Figure 31.17 A flow chart for the assessment of soil characteristics; in particular the strength of the till

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of the stiffness of elements of tills may not be representative of the mass stiffness. The comments on the impact of fabric and sample size on strength apply to measurements of stiffness.

No matter which method is used it is important to test a quality element of till. This means high quality samples such as wireline and block samples from matrix dominated till, which are only possible on major projects because of the cost. In general, U100-driven tube samples will be available. In that case it may be prudent to determine stiffness from tests on reconstituted till though, as with strength, the value obtained will be less than the *in situ* value. However, as **Figure 31.20** shows, results (the shaded area) can be obtained from intact samples of matrix dominated tills.

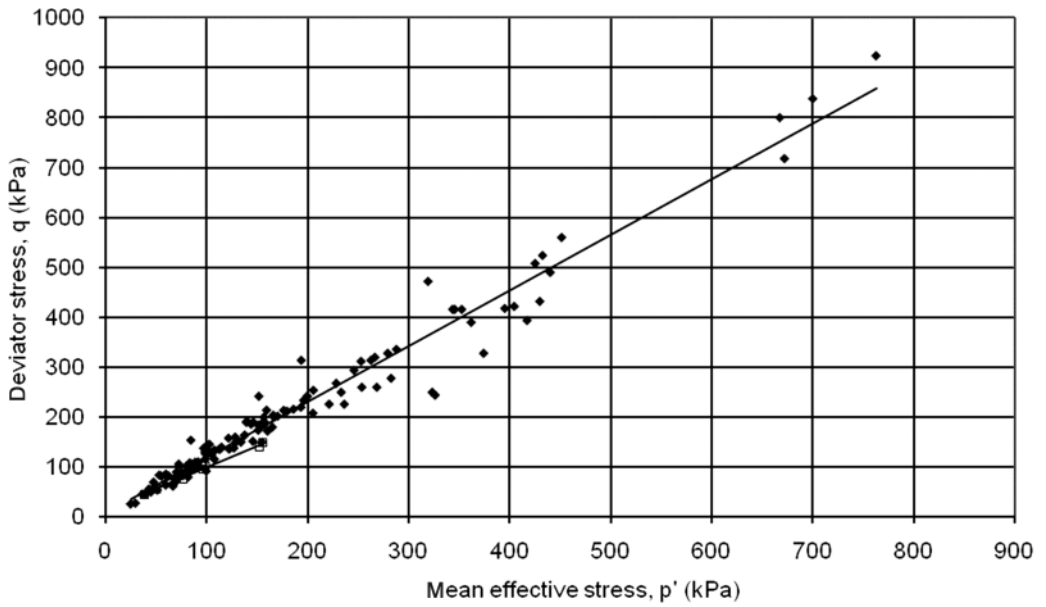


Figure 31.18 Characteristic strength derived from tests on individual specimens. The ranges of cohesion (0 kPa to 27 kPa) and angles of friction (15° to 34°) were reduced to characteristic values of cohesion of 2.5 kPa and angle of friction of 31°; the two trend lines refer to the best fit to the data from 0 to 100 kPa (p') and 0 to 800 kPa

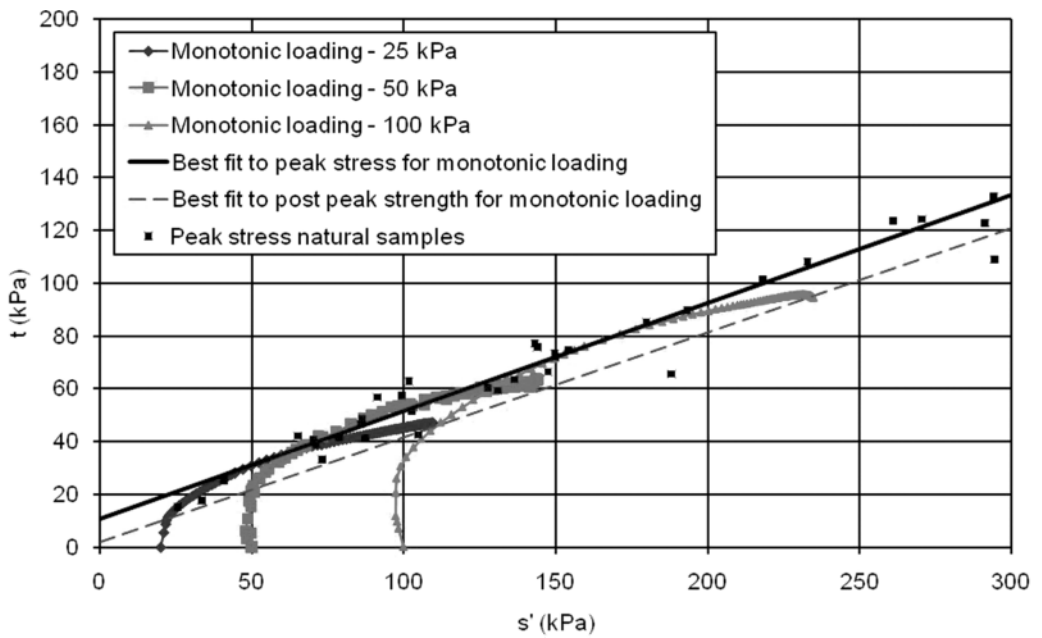


Figure 31.19 Comparison between tests on reconstituted tills and tests on intact specimens showing that the strength of the reconstituted till lies below that of the natural tills

It is possible to measure stiffness *in situ* (e.g. pressuremeter, plate and cone). In these cases the installation of the measuring device inevitably disturbs the till, leading to an underestimate of the *in situ* stiffness, leading in turn to the use of empirical correlations. Non-destructive geophysical methods can be used to obtain the maximum shear modulus (G_{max}) with some degree of accuracy. The shear modulus strain degradation curve obtained from quality tests on elements of till can then be normalised to G_{max}. These techniques are expensive and may be used on large projects, but a combination of geophysical tests and tests on reconstituted samples of till may be the best combination to determine the stiffness characteristics of till.

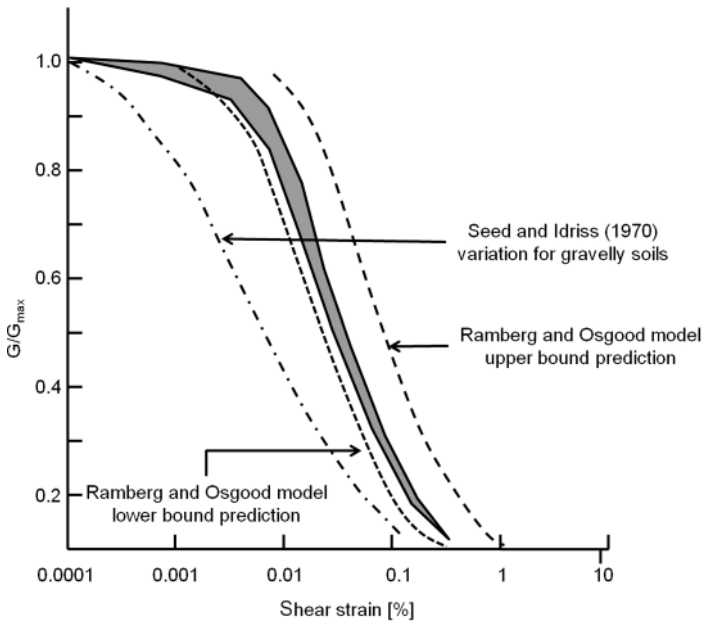


Figure 31.20 Degradation of stiffness from resonant column tests on samples of glacial till
Reproduced from Chegini and Trenter (1996)

31.5.4 Hydraulic conductivity

The *in situ* hydraulic conductivity of glacial soils will be dominated by the fabric and structure of the soil; hence, correctly describing the till and identifying its features are critical. The presence of fissures in clay tills and the laminations in glaciolacustrine deposits imply that the *in situ* hydraulic conductivity may be several orders of magnitude greater than that of the intact matrix. The implication is that the hydraulic conductivity measured in the laboratory will depend on sample size and orientation as well as density and grading, but *in situ* the fabric may be the dominant factor. Hence, as with strength, scatter in values of hydraulic conductivity is expected and a representative value may be difficult to obtain from laboratory tests. Lloyd (1983) suggested that the permeability of clay matrix-dominant tills would be between 10^{-11} and 10^{-8} m/s; and if fissured between 10^{-10} and 10^{-7} m/s. It is possible to determine the intrinsic value of hydraulic conductivity from reconstituted till (**Figure 31.21**). In practice, if the hydrological characteristics are critical, then it is best to undertake *in situ* borehole tests using a test pocket that is sufficiently long enough to take into account the representative fabric and features of the till. This means that it is important to establish a quality 3D geological model that adequately describes those features which could influence the mass hydrological characteristics.

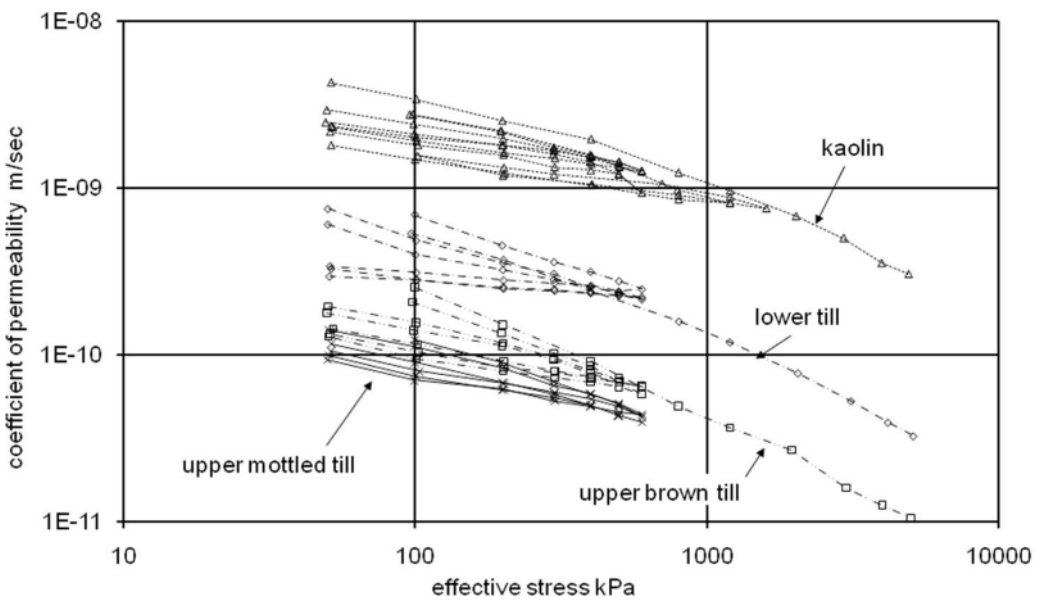


Figure 31.21 Hydraulic conductivity of glacial tills (compared to kaolin) from constant flow tests on triaxial samples
Reproduced from Clarke and Chen (1997)

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31.6 Routine investigations

Much of the development work on exploratory boreholes in UK site investigation practice took place in the south-east of England (e.g. Cooling and Smith, 1936; Cooling, 1942) where soft alluvial clays, stiff clays, and sands and gravels are found. They evolved from well-boring techniques that fixed the diameter of the boreholes and sampling devices which led to the U100 sampling tube and the Standard Penetration Test (SPT), both of which had their origins in American 'dry' sampling techniques. This then fixed the size of the test specimens.

The subject of soil mechanics was being developed at the same time as the site operations and this led to the introduction of the oedometer, triaxial and shear box tests. The testing techniques have essentially remained the same since then. While the techniques have proved successful in areas in which clays and sands are the predominant soil type, they are not necessarily the best for other areas where heterogeneous mixtures of soil particles are more commonly found. For example, it is difficult to select design parameters from standard tests on tills because of the difficulties in sampling and testing these soils using standard equipment. It may be possible to develop other techniques, but they could not be used in practice because parameters are test dependent and confidence in design methods is based on standard tests.

Table 31.6 lists the sampling methods suggested by Trenter (1999) giving an indication of the quality of the sample. The U100 driven tube is the most common method of sampling matrix-dominant tills in the UK. The quality of the sample will depend on the amount and size of gravel and cobbles present, the strength of the matrix and the presence of shear planes. A detailed description of those samples tested is required to help evaluate the significance of the results of the tests. For example, the failure of a sample in a triaxial test may be a result of the fabric and not the strength of the matrix.

31.7 Developing the ground model and design profile

31.7.1 The ground model

Figure 31.14 highlights the difficulty in assigning characteristic values to this spatially variable soil. Clarke *et al.* (2008) have suggested that it is possible to use a regional database to create characteristic values. In that way data from a new site can be compared to the regional database and, if found to be statistically aligned with that database, then the characteristic values for the region can be used.

Box plots (**Figure 31.22**) are a visual means of assessing the normality of the data. From an engineering point of view the differences in Atterberg limits between the sites are not significant, hence the regional values have been identified. This approach can also be applied to water content and density.

Schneider (1999) suggested that the characteristic value for a new investigation is simply the mean less half the standard deviation, since the regional coefficient of variation is likely to be unknown. **Table 31.7** shows that this can lead to a characteristic value with about 70% of the results exceeding that value; i.e. it is an overcautious estimate of the mean.

The first step in establishing the characteristic values from a new site is to determine whether the soils are tills. This could be proven from the geological description, Atterberg limits

Glacial soil type (typical description)	Equipment/method of sampling	Comment
Matrix-dominant (sandy clay with gravel)	U100 driven tube	Generally suitable depending upon the amount and size of gravel and cobbles present; maintain frequent checks for circularity of sampler; change cutting shoes regularly and keep inside of sampler lightly oiled
	Block	Suitable: adjust block size to cope with largest particles present
	Rotary core double tube plus liner	Suitable for strata definition but swelling on contact with flushing medium may preclude use for laboratory test purposes
	Wireline	Generally suitable. Note that the bit is not routinely brought to the surface to permit inspection for wear, as with the conventional system
Clast-dominant (clayey sandy gravel with occasional cobbles)	U100 driven tube	Generally unsuitable because of the gravel content: the sample will be disturbed; using double tubes may make it possible to recover a (disturbed) sample sufficient for lithological description
	Block	Depends upon amount and plasticity of matrix: where sufficient to form a coherent block during excavation, the procedure may be used; adjust block size to cope with largest particles present
	Rotary core double tube plus liner	Sometimes suitable for strata definition but core retention may be difficult; bit wear may be high; usually unsuitable for laboratory test purposes
	Wireline	May be suitable, depending on amount, nature and angularity of the particles. Note that the bit is not routinely brought to the surface to permit inspection for wear, as with the conventional system

Table 31.6 Guide to selection of sampling methods in glacial tills
Data taken from Trenter (1999) (CIRIA C504)

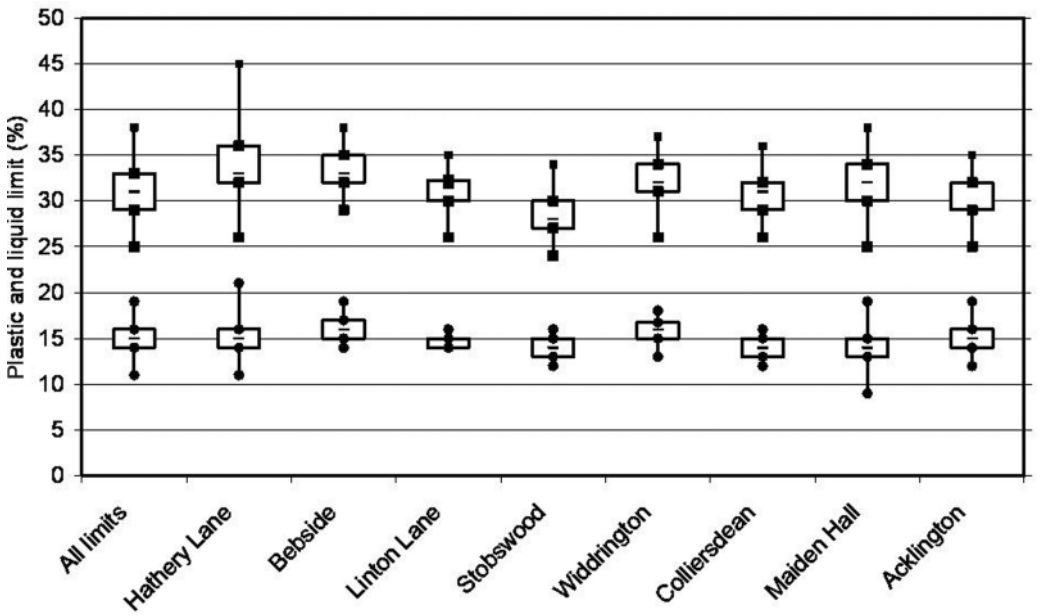


Figure 31.22 Box plots showing the regional variation of the Atterberg limits for glacial tills; the median is the data point in the middle of a box which represents the samples between the first and third quartile. The extensions refer to the limits of the data. The names refer to opencast mines in northern England

Parameter	Regional				Site specific					
	Mean	Characteristic values			Mean	Characteristic values				
		Schneider (low)	Cautious mean (low)	Cautious mean (high)		Schneider (low)	Cautious mean (low)	Cautious mean (high)	Bayes mean (low)	Bayes mean (high)
Till A										
γ b (Mg/m3)	2.00	1.960	1.993	2.003	2.11	2.07	2.09	2.13	2.10	2.11
w (%)	21.5	19.74	21.28	21.72	20.8	18.2	19.7	21.9	20.8	20.8
PL (%)	20.3	18.98	20.17	20.53	20.1	18.3	19.2	21.0	20.1	20.1
LL (%)	46.5	43.18	46.06	46.93	42.4	38.8	40.6	44.3	42.5	42.6
su (kPa)	99	76	96	102	89	71	77	100	89	89
Till B										
γ b (Mg/m3)	2.06	2.023	2.059	2.065	2.16	2.12	2.13	2.19	2.16	2.16
w (%)	18.1	16.56	18.02	18.25	16.4	13.6	15.5	17.4	16.48	16.48
PL (%)	17.6	16.39	17.51	17.69	15.6	14.3	15.0	16.3	15.7	15.7
LL (%)	38.7	36.02	38.49	38.89	34.6	31.6	33.2	36.0	34.7	34.7
su (kPa)	100	78	98	101	89	60	65	104	86	86

Table 31.7 The effect a regional database has upon the selection of characteristic values
Data taken from Clarke *et al.* (2008)

and grading curves. If this combination indicates that the soils are likely to be typical of the region, then Bayesian statistics can be used to determine the characteristic values. An example is given in **Table 31.7**, which shows the values based on the data set and on *a priori* knowledge of the regional data set. This leads to less cautious design parameters and hence a more economical, design. The Bayesian low mean or high mean is used depending on the worst condition being assessed.

31.7.2 Intrinsic properties

Design parameters can be selected using a deterministic model based on empirical or theoretical relationships or a stochastic

model based on a statistical analysis of the data or a combination of these. The former includes relationships between the Atterberg limits and undrained shear strength of remoulded clays (e.g. Wroth and Wood, 1976; Carrier and Beckman, 1984); *in situ* strengths of normally consolidated clays, density and Atterberg limits (e.g. Skempton, 1957); and overconsolidated soils and Atterberg limits (e.g. Yilmaz, 2000). Given the spatial variability of tills it is recommended that a consistent framework for the interpretation of design parameters is used which is based on a geotechnical model that encompasses a study of the deposition and post-depositional processes, soil mechanics theory and experimental data, including tests on reconstituted tills.

Burland (1990) suggested that the compressibility and strength of reconstituted clay could provide a consistent framework for the interpretation of properties of natural clay. Reconstituted clay is a natural clay mixed with water such that the water content is at about 1.25 w_L; the true liquid limit. The properties of this slurry are inherent to the material but independent of its natural state. In the case of a clay matrix-dominant till the slurry should be made from the clay matrix. This is not practical given the difficulty of separating out the fine particles. However, removing all particles exceeding 2 mm produces a soil that is essentially the matrix of the till.

This framework is shown in **Figure 31.23** in which the average shear strength for three tills is plotted against the void index, I_v:

$$I_v = \left(\frac{e - e^*_{100}}{C_c^*} \right) \tag{31.1}$$

where e^*_{100} is the void ratio at a vertical effective stress of 100 kPa on the intrinsic compression line and C_c^* is the slope of the intrinsic compression line (ICL). SCL is the sedimentation compression line, which is the compression line for natural or intact soils. Burland (1990) suggested that these two intrinsic properties are empirically related to the void ratio at the liquid limit, e_L:

$$e^*_{100} = 0.114 + 0.581 e_L \tag{31.2}$$

$$C_c^* = 0.256 e_L - 0.04 \tag{31.3}$$

The intrinsic compression line represents the compression curve for reconstituted clays; the intrinsic strength line, ISuL, represents the strength of reconstituted soils (Chandler, 2000). **Figure 31.23** shows that results of tests on reconstituted tills (Clarke *et al.*, 1998) fit to the ISuL which suggests that this framework can be used to characterise the tills.

The majority of the data from tests on samples of natural tills lie to the left of the ISuL. Two of the tills lie about the trend line for the regional database. The scatter in the data reflects the nature of till but a combination of the geological history, description and the classification characteristics suggests that the tills are the same and part of the regional database. The third till lies about a separate but parallel regional trend. The fact that the data points lie to the left of the ISuL confirms that they are overconsolidated. The single till on the lower trend line is there because the pre-consonsolidation pressure exceeded that of the other two tills. The slope of the lines is such that the average void ratio is very nearly constant, which

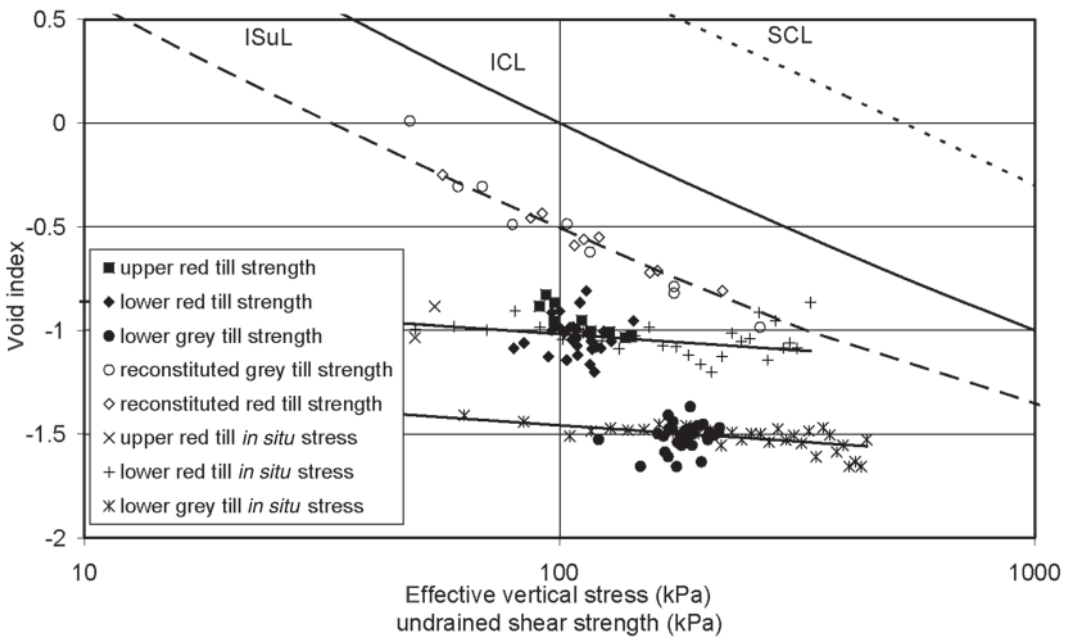


Figure 31.23 The properties of three basal tills using the intrinsic framework proposed by Burland (1990) and Chandler (2000)
Reproduced from Clarke *et al.* (1998) © The Geological Society

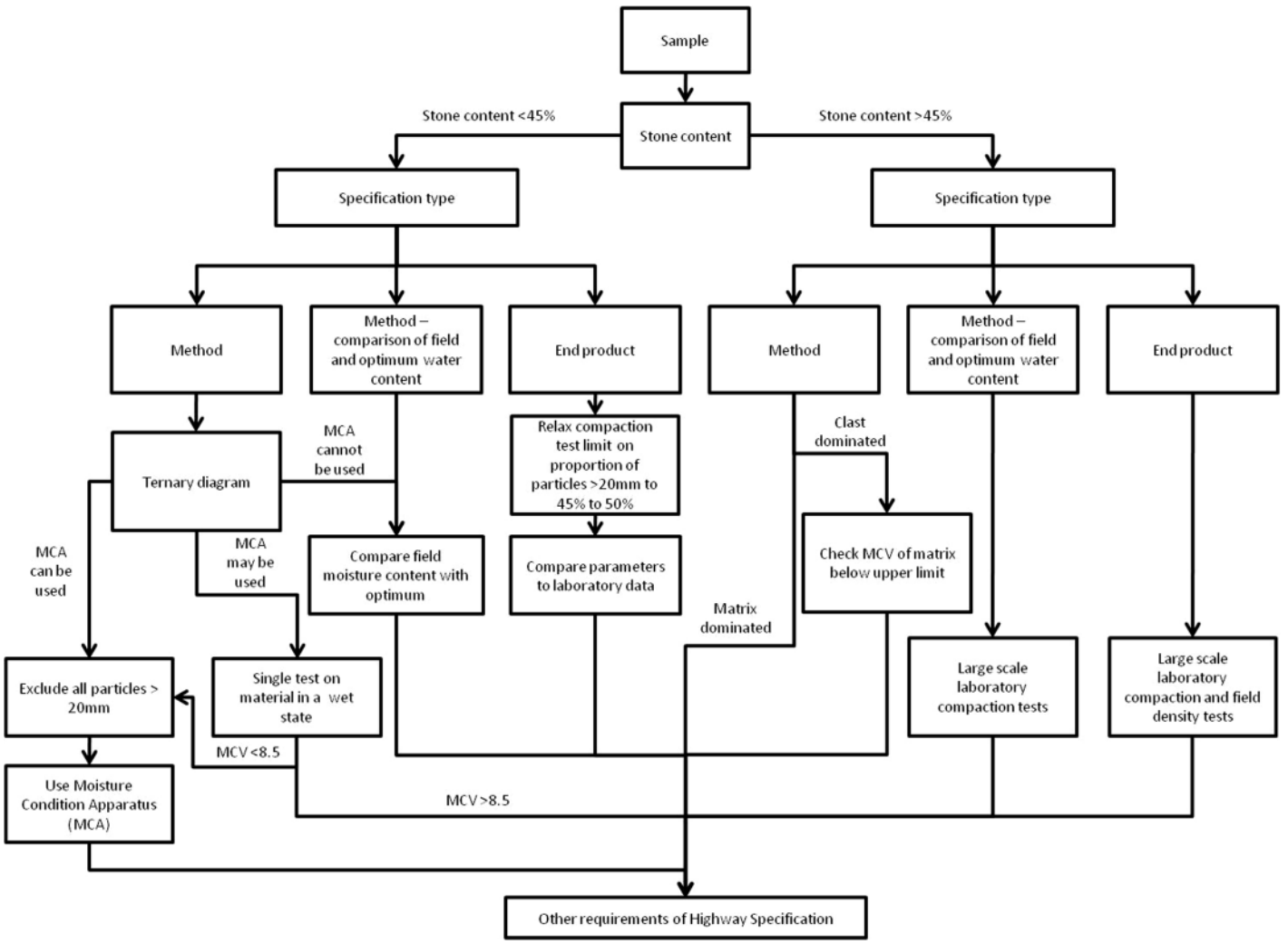


Figure 31.24 Flow chart for the acceptability of tills for earthworks
Reproduced from Winter *et al*. (1998) © The Geological Society

implies that the characteristic shear strength profile could be considered to be vertical.

31.8 Earthworks

The previous sections have focused on the selection of design parameters for excavations, foundations and retaining structures. Till is also used as a construction material for earthworks and landfills. In these cases it is necessary to prove that the material can be compacted to the correct density, the compacted material is strong enough to resist failure in the design of embankments and, for landfill liners, it is ductile enough to remain relatively impermeable when subject to deformation.

It is most likely that a matrix dominated till could be used for landfill construction including liners. The acceptable criteria are likely to be based on a 5–10% air voids line. Tills have a variable mean particle density (PD) which implies, in practice, that it is possible for site tests to plot above the 0% air voids line, because of the high PD particles in the sample. It is unlikely that the water content of the till will be altered so it is important to determine the hydraulic conductivity, strength and ductility of the till at that water content. Typically a strength of 50 kPa is required (Terraconsult, 2010) to allow clay to be compacted. The maximum hydraulic conductivity is 10^{-9}m/sec. A reduction in water content increases the strength, making it more difficult to compact and possibly producing a brittle material that will fracture on straining, leading to an unacceptable increase in hydraulic conductivity. Further details of the testing regime can be found in the Environment Agency's guide to landfill design (Terraconsult, 2010).

The thickness of a compacted layer specified by the Highways Agency is 300 mm, which means that all particles in excess of 125 mm have to be removed. Thus it is still possible to use matrix-dominant and clast-dominated tills. However, there is a limit to the size of particles in the control tests (e.g. compaction and moisture condition value). Stones (20 mm to 37.5 mm) and cobbles have to be removed before testing. If the stone

Property	'Minimum' requirement	Test
Permeability/hydraulic conductivity	Depends on Environmental Permit	BS 1377:1990, Part 6: Method 6
Remoulded undrained shear strength	Typically ≥ 50 kPa or other site-specifically defined value	BS 1377:1990, Part 7: Method 8
Plasticity index (Ip)	10% ≤ IP ≤ 65%	BS 1377:1990, Part 2: Methods 4.3 and 5.3
Liquid limit	≤ 90%	
Percentage fines	<0.063 mm (63 μm) ≥ 20 % but with a minimum clay content (particles < 2 μm) of 8 %.	BS 1377:1990, Part 2: Methods 9.2, and 9.5
Percentage gravel	> 5 mm ≤ 30%	
Maximum particle (stone) size	2/3rd compacted layer thickness	Typically 125 mm but must not prejudice the liner, for instance by larger particles sticking together to form larger lumps

Table 31.8 Properties of tills for landfill liners
Data taken from Terraconsult (2010)

content is small, i.e. they are discrete elements in the sample, then it is possible to test the matrix (Winter *et al.*, 1998). This is the case for up to 50% stone content. Above that the stones govern the behaviour. An increase in stone content increases the maximum dry density and lowers the optimum water content but this is not important since the stone of high density and low water content is replaced by a low density high water content, matrix. **Figure 31.24** shows the method of selecting tills for earthworks. Note that if those earthworks include landfill liners then there are further limits indicated in **Table 31.8**.

31.9 Concluding comments

Glacial soils are a hazard but, through a strategic approach to the ground investigation linked to an understanding of the impact the geological processes have upon the geotechnical behaviour, it is possible to mitigate against the risk. There are two extremes. The first is to base the design parameters on the properties of the dominant material; the second is to base the design on the zones of weakness. The former can lead to local failure; the latter to overdesign. Therefore, investing in the ground investigation to establish the extent of the zones of weakness and taking sufficient samples to establish the characteristic values is prudent. This should be carried out with a thorough understanding of the ground model developed from previous investigations and observations of exposures of glacial soil.

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It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 13 *The ground profile and its genesis*
- Chapter 17 *Strength and deformation*
- Chapter 40 *The ground as a hazard*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 32
Collapsible soils

Ian Jefferson School of Civil Engineering, University of Birmingham, UK
Chris D. F. Rogers School of Civil Engineering, University of Birmingham, UK

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Collapsible soils present significant geotechnical and structural engineering challenges the world over. They can be found in many forms – either naturally occurring or formed through human activities. However, an essential prerequisite is that an open metastable structure develops through various bonding mechanisms. Bonds can be generated via capillary forces (suctions) and/or through cementing materials such as clay or salts. Collapse occurs when net stresses (via loading or saturation) exceed the yield strength of these bonding materials. Collapse is most commonly triggered by inundation through a range of different water sources, although the impact varies with different sources yielding different amounts of collapse. To engineer in and mitigate the effects of collapsible soils, it is essential to recognise their existence, which may not be easy, and to gather vital geologic and geomorphologic information. Collapsibility should be confirmed through direct response to wetting/loading tests using laboratory and field methods. The key challenge faced with collapsible soils is the spatial extent and the degree of wetting that will take place. Care is needed to ensure that appropriate and realistic assessments are undertaken. Ultimately, if treated using one of a suite of the possible improvement techniques available, then the potential for collapse can be eliminated effectively.

32.1 Introduction

Collapsible soils are extremely common and can be formed naturally owing to various geologic and geomorphologic processes, or be the result of human activity. These processes, although different in nature, allow the development of an open metastable structure: the essential prerequisite to the formation of a collapsible deposit (Dudley, 1970). Volume changes that occur are more sudden than those experienced through consolidation processes and typically occur in material that is non-plastic or of very low plasticity, confined and initially dry (Houston *et al.*, 2001).

Although collapsible soils are found only in arid regions, arid environments tend to favour their formation. Naturally occurring collapsible soils are formed typically from debris flow (e.g. alluvial fan materials), as wind-blown sediments (e.g. loess), as cemented high salt content metastable soils (e.g. sabkha, see Chapter 29 *Arid soils*), and as tropical residual soil (see Chapter 30 *Tropical soils*). In addition, collapsible soils can be formed artificially through poor compaction control or where compaction is dry of optimum (e.g. non-engineered fill, see Chapter 34 *Non-engineered fills*), or as waste materials (e.g. fly ash beds, Madhyannapu *et al.*, 2006). However formed, common to almost all collapsible soils are both low densities and a relatively stiff and strong state when dry (as illustrated in **Figure 32.1**). Notable exceptions to this are post-glacial sensitive 'quick' clay soils found mainly in Canada, Alaska and Scandinavia, but these are arguably a special case, being geographically centred and in a saturated collapsible state. An additional exception is saturated slide material such as saturated sands that has been allowed to flow on slopes that have experienced liquefaction (Nieuwenhuis and de Groot, 1995).

Collapsible soils can be defined as soils in which the major structural units are initially arranged in an open metastable packing through a suite of different bonding mechanisms. If the soil is loaded beyond the yield strength of the bonding material, collapse will occur – this results in a rearrangement of particles to form a denser stable configuration (see **Figure 32.2**). Thus the collapse itself is controlled both microscopically and macroscopically. An appreciation of both these elements is essential if the true nature of collapse, and therefore its effective remediation, are to be fully understood.

Collapse is often triggered by a combination of increased stress (load) and the addition of water leading to increased



Figure 32.1 Example of loess in its pre-collapsed state exhibiting its relatively stiff soft rock state when dry, allowing the formation of man-made caves, Slovakia

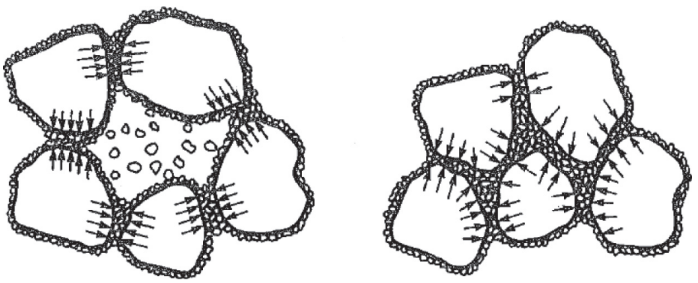


Figure 32.2 An illustration of collapse through inundation
Reproduced from Houston *et al.* (1988) with kind permission from ASCE

degrees of saturation. However, collapse is most often triggered by an increase in water content. There are a variety of sources of water ingress that can initiate collapse, many associated with urban environments. These include landscape irrigation, broken water or sewer pipes, run-off or poor drainage control, groundwater recharge, or water content changes through capillary rise.

An example of the effects of urbanisation and associated landscaping was illustrated by a commercial building in New Mexico. After winning the city's most beautiful lawn and landscaping award, achieved through heavy watering, the building suffered US$ 500 000 foundation damage owing to soil collapse (Houston *et al.*, 2001). The result of infiltration with water results in a relatively sudden volume compression, and is often associated with loss of strength. This can clearly have important geotechnical consequences, including loss of serviceability resulting in expensive remediation or, on occasion, complete failure. This was illustrated by the structural failures caused by damage to foundations on collapsible soils in Egypt (Sakr *et al.*, 2008), and the collapse problems caused by hydrogeological changes associated with the construction of a dam in Brazil (Vilar and Rodrigues, 2011).

Other reported problems have occurred in embankment bases (Thorel *et al.*, 2011), dam embankments (Peterson and Iverson, 1953), road embankments (Knight and Dehlen, 1963) and fill (Charles and Watts, 2001). It can be particularly problematical when collapse causes damage to historic buildings (Herle *et al.*, 2009), e.g. the cracking that developed in the wall of a 15th century pagoda in Lanzhou, China, after the introduction of an irrigation scheme (see **Figure 32.3**).

From this it is clear that, given the correct depositional environment, collapse has the potential to occur in any soil. In fact, given the correct stress environment, a collapsible soil may exhibit expansive behaviour (Derbyshire and Mellors, 1988; Barksdale and Blight, 1997; see also section 32.3.3 below). It is therefore necessary to understand the process of collapse if problems associated with collapsible soils are to be avoided or mitigated. As collapse is for the most part triggered by increases in water content, problems are often



Figure 32.3 Collapse-induced cracking in a 15th century pagoda
Reproduced from Billard *et al.* (2000); John Wiley & Sons, Inc.

encountered in urban and built environments and so the impact occurs where it has potential to cause the most harm. This can be particularly problematical as in many parts of the world collapsible soils exist in areas of high seismic actively (Houston *et al.*, 2003). The consequences can be catastrophic, e.g. the Haiyuan earthquake in which induced loess landslides in 1920 killed over 200 000 people (Derbyshire *et al.*, 2000; Zhang and Wang, 2007). Moreover, the potential for collapse will remain until full collapse has been induced either naturally through flooding and/or loading (earthquakes), or artificially by human activity. Human activity can be either accidental via poor drainage control, or deliberate through ground improvement, e.g. dynamic compaction.

32.2 Where are collapsible soils found?

Rogers (1995), Lin (1995), Bell and de Bruyn (1997) and Houston *et al.* (2001, 2003) discuss in detail the various forms of collapsible soils found worldwide. Collapse occurs because

soil has certain inherent properties. Typical features that are found with most collapsible soils include:

- an open metastable structure;
- a high voids ratio and low dry density;
- a high porosity;
- a geologically young or recently altered deposit;
- a deposit of high sensitivity;
- a soil with inherent low interparticle bond strength.

Thus many soils can and will exhibit collapsible behaviour. This is illustrated in **Figure 32.4** along with the various formation processes that can yield a collapsible soil.

Collapsible soils include naturally occurring soil formations, such as tropical residual soils, loess and quick clays, and anthropogenic soils such as uncompacted or poorly compacted fill. However, any compacted soil can exhibit collapse if the confining pressure is high enough (see section 32.3.3 below). Further details of tropical and anthropogenic soil collapse can be found in Chapters 30 *Tropical soils* and 34 *Non-engineered fills* respectively. Additional information on the collapse behaviour of residual soils is also provided by Barksdale and Blight (1997) and Roa and Revansiddappa (2002). Examples include decomposed gneiss (Feda, 1966), decomposed granites (Brink and Kanty, 1961) and granitic gneiss (Pereira and Fredlund, 2000).

Other soils exhibiting collapse include collapsible gravels (Rollins *et al.*, 1994), granitic sands in South Africa (Jennings and Knight, 1975), collapsible sands of the southern African east coast (Rust *et al.*, 2005) and quick clays (Bell, 2000). Loess is probably the most commonly encountered naturally occurring collapsible soil, covering some 10% of the worlds landmass (Jefferson *et al.*, 2001). Many of the practices and engineering approaches used to deal with collapsible soils stem from the treatment of loess soils.

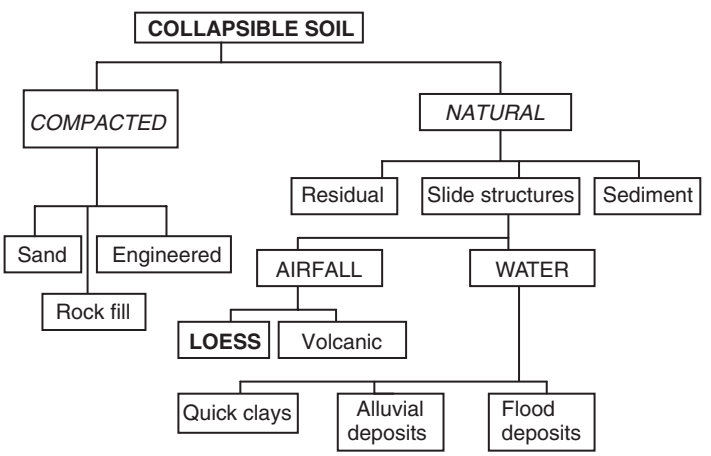


Figure 32.4 Classification of collapsible soils (Rogers, 1995)

32.2.1 Wind-blown soils – collapsible loess soils

Loess consists essentially of silt-sized (typically 20–30 μm) primary quartz particles that form as a result of high energy earth-surface processes such as glacial grinding or cold climate weathering (Rogers *et al.*, 1994). These particles are transported from the source by rivers. Subsequent flooding by these rivers allows the quartz silt particles to be deposited on flood plains (Smalley *et al.*, 2007). On drying out, these particles are detached and transported by the prevailing winds until deposition leeward at distances ranging from tens to hundreds of kilometres. Cementing materials are often added after deposition or dissolved and re-precipitated at particle contacts (Houston *et al.*, 2001). This process has resulted in the almost continuous deposit draped over the landscape from the North China plain to southeast England (where it is generally referred to as 'brickearth') – see **Figure 32.5**.

It is possible to isolate five major loess regions worldwide: North America, South America, Europe (including western Russia), central Asia and China (Smalley *et al.*, 2007). These loess regions often underlie highly populated areas and major infrastructure links, making them vulnerable to soil collapse. The areas of most widespread concern are concentrated in eastern Europe, Russia and, to a growing extent, China (see Derbyshire *et al.*, 1995, 2000), although potentially serious problems of collapse exist wherever loess is found.

Figure 32.5 indicates the approximate distribution of loess/brickearth greater than 0.3 m in thickness in the UK. Significant thicknesses (greater than 1 m) are restricted to north and east Kent (see Fookes and Best, 1969; Derbyshire and Mellors, 1988), south Essex (see Northmore *et al.*, 1996) and the Sussex coastal plains. In Essex, deposits of up to 8 m have been found, although thicknesses of 4 m or so are

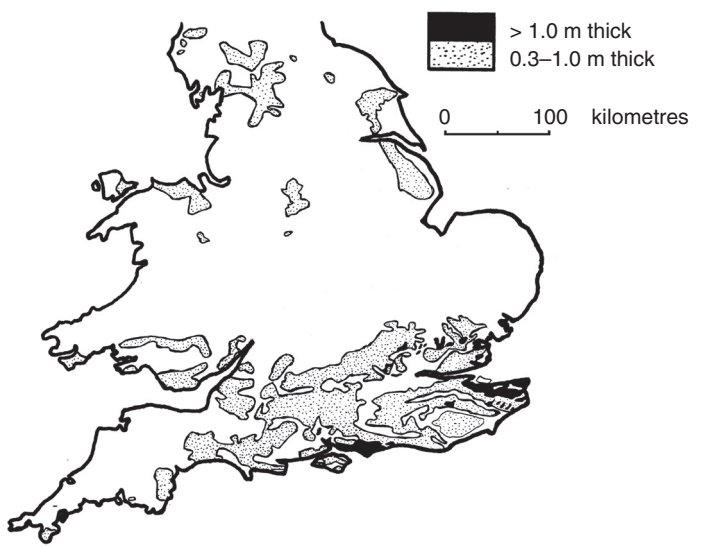


Figure 32.5 Distribution of 'brickearth' (loess) deposits in southern England and Wales
Modified from Catt (1985); Allen & Unwin (as reproduced in Jefferson *et al.*, 2001)

more typical (Northmore *et al.*, 1996). However, substantial deposits can reach tens and even hundreds of metres thickness worldwide (see Smalley *et al.*, 2007 for further details).

Regional trends in loess occur in the type of deposit found around the world and these can be determined through textural and mineralogical distinctions. However, there is generally a progressive decrease in modal size with distance from its source material – suggesting that sorting of loess material occurs with wind direction during deposition (Catt, 1985).

Originally, loessic deposits would have been more extensive but they have been removed by post-depositional erosion, colluviation, deforestation/agriculture and resource stripping activities (Catt, 1977). As a result, modern loess deposits are often only found overlying relatively permeable strata.

32.2.2 Other collapsible deposits

As discussed in sections 32.2 and 32.3, many soils exhibit collapsibility. Apart from those already highlighted above, a number of water-sediment deposits exhibit collapse potential. Two main groups exist: alluvial deposits and quick clays. Alluvial deposits reported to cause collapse problems include alluvial fans, alluvial flood plain deposits and mud flow deposits, with several case histories of collapse problems in alluvial fan deposits being provided by Rollins *et al.* (1994).

By comparison, saturated quick clay has become unstable owing to its post-glacial depositional environments, allowing an open structure to form via slow sedimentation under shallow marine conditions (Rogers, 1995; Locat, 1995). The resulting open fabrics are maintained by small amounts of carbonate cementation of clay minerals, with salt leaching often having occurred. Quick clays typically have liquidity indices greater than 1, with liquid limits often less than 40% (Bell, 2000). When disturbed, the particles in quick clays are remoulded into closely packed configurations. As the water content remains unchanged during the collapse process, quick clays become oversaturated and may flow as a viscous liquid. This can have devastating effects as witnessed by the 1978 landslide in Rissa, Norway, where, after the initial landslide, a series of minor slides developed which eventually covered an area of 3 300 000 m^2; or more recently by the large landslide in Leda clay in Ontario, Canada in 1993 (see Bell (2000) for further discussion).

32.3 What controls collapsible behaviour?

Collapsibility occurs owing to the various geomorphologic and geologic processes as well as human activities during soil formation. The key to this is understanding the nature and processes that take place during provenance (P), transportation (T) and deposition (D) of the soil particles: it is the PTD sequence of a deposit that produces an open metastable collapsible structure of relatively high void ratio (Sun, 2002; Jefferson *et al.*, 2003b; Smalley *et al.*, 2007). The PTD approach conceptualises the processes of soil genesis, subsequent transportation and ultimate disposition, so elucidating how these influence the engineering behaviour of the final deposit.

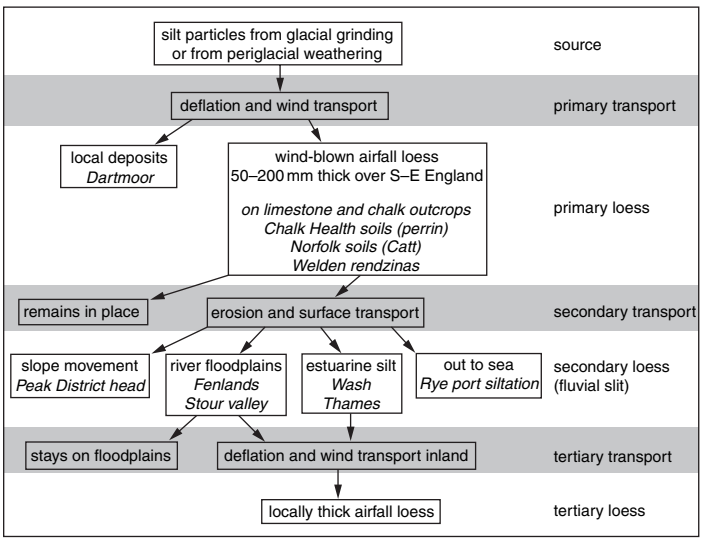


Figure 32.6 A PTD geomorphological model of loess in southern Britain. Note: Norfolk soils (Catt *et al.*, 1971); Chalk Heath soils (Perrin, 1956)
Reproduced from Jefferson *et al.* (2003b); East Midlands Geological Society

However, some deposits have been through secondary or tertiary PTD sequences, often reducing or even eliminating collapse potential. An example to illustrate this is given in **Figure 32.6**, based on a PTD model of loess found in southern Britain, with further details discussed by Pye and Sherwin (1999) and Derbyshire and Meng (2005).

Wind-blown collapsible soils such as loess often consist of different zones of variable collapsibility, both laterally and with depth, as a result of their geomorphology. The prevailing wind direction generates zones of material laterally changing in nature from sandy loess through to clayey loess deposits; see **Figure 32.7**. It should be noted that the terms clayey, silty and sandy loess are defined by particle size analysis (see Holtz and Gibbs, 1952). In turn, the degree of saturation and potential collapsibility often follows similar trends. For example, the loess in Bulgaria (**Figure 32.7**) exhibits an increase in saturation as the modal size reduces (Jefferson *et al.*, 2002). The thicknesses of loess formed often follow the same pattern – with the greatest thicknesses occurring nearest to the source material.

In addition, deposits such as loess have been formed over many thousands of years and during this time climactic conditions have varied considerably. As a result, the loess soil sequence has alternating layers of loess (formed during cold periods) and clay-rich palaeosols (formed during warm periods) – see **Figure 32.8** for illustration.

The alternating nature of loess formation significantly influences the engineering behaviour and ultimately the nature of collapse, and the location (depth) where collapse occurs. This will dictate the nature of the infiltration patterns of water into the soil and as a result can yield collapse in unexpected locations. Moreover, this can influence the

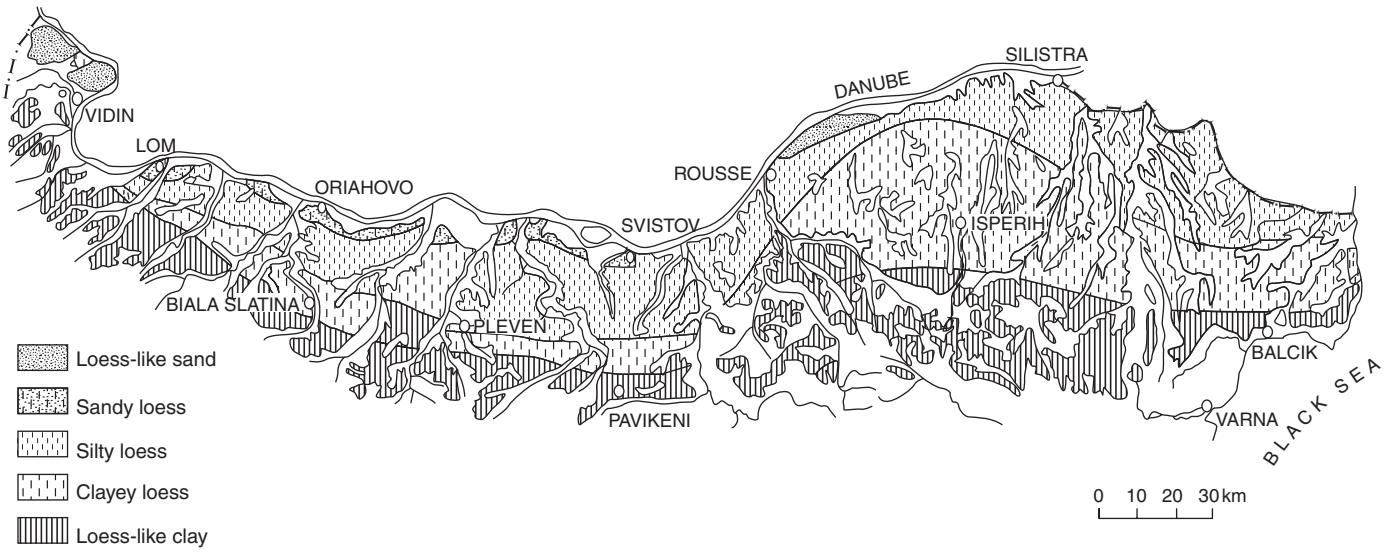


Figure 32.7 Distribution of collapsible loess soils from Danube flood plain in Bulgaria
Reproduced from Minkov (1968); Marin Drinov Academic Publishing House/BAS Press

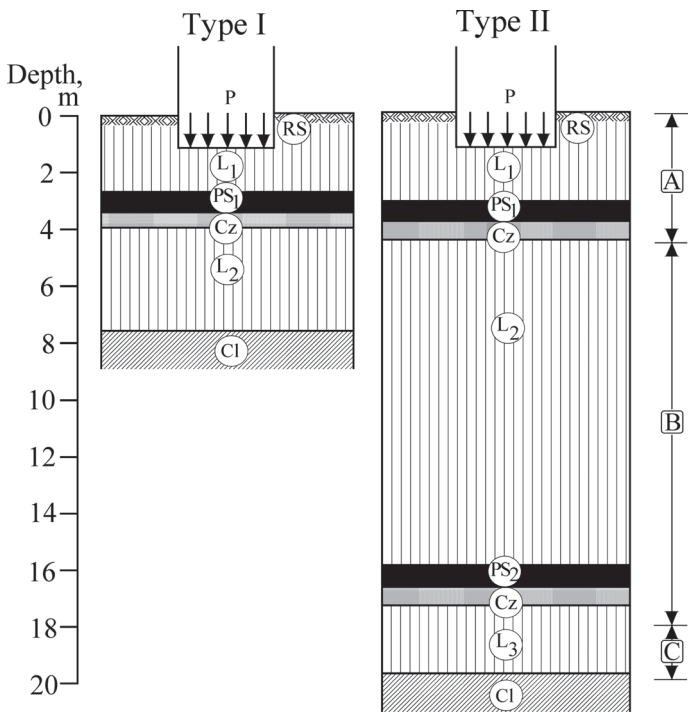


Figure 32.8 Types of collapsible loess soils
Reproduced from Jefferson *et al.* (2005), with permission from Elsevier

effectiveness of any ground improvement approach used to remove collapsibility.

As a result of the various geomorphologic processes involved in the formation of loess, these deposits typically have three zones of relative collapsibility:

- Zone 1: a zone at depth of collapsed material due to overburden pressure.
- Zone 2: a collapsible zone.
- Zone 3: a surface crust (which requires additional load to cause collapse). This has led countries with extensive loess deposits (including Eastern Europe and the former Soviet Union) to develop classification schemes related to the collapse of loaded foundations (see **Figure 32.8**). Two types of collapsibility occur in such schemes:

  Type I mainly loaded collapsibility (collapse deformation under overburden pressure of $\delta_n < 5$ cm); and

  Type II mainly unloaded collapsibility ($\delta_n > 5$ cm, where δ_n is the collapse deformation).

Type I loess is usually of small thickness (shown in **Figure 32.8**) and contains one or two palaeosols (PS), together with an associated carbonate zone (Cz). Collapse occurs after the foundation stress exceeds a certain critical stress, which can be determined by laboratory tests or tests in the field (see section 20.4 below).

Type II loess has greater thickness – up to 50 m or more. **Figure 32.8** shows a typical case of a Danubian terrace with a deposition of loess of about 20 m. In this case, three loess horizons (L_1, L_2 and L_3) are separated by two palaeosols (PS_1 and PS_2). In the Type II loess, three zones can be distinguished:

(i) upper zone A – no unloaded collapsibility but with potential loaded collapsibility;

(ii) middle zone B – unloaded collapsibility; and

(iii) lower zone C – uncollapsible (or collapsed).

Loess in zone C has previously collapsed under overburden pressure. Loess in zone A has had no unloaded collapsibility, since here the overburden pressure is small (i.e. it will not collapse under self-weight). However, it can be collapsible under additional load. In zone B, unloaded collapsibility occurs – it often contains thicker loess horizons with lower density and

higher porosity, n (i.e. $n > 50\%$), and it has a higher silt content than in zone C.

Further details of the different geomorphologic processes that generate collapsibility are given in Chapters 30 *Tropical soils* and 29 *Arid soils*, and an excellent treatment of the broader subject is provided by Fookes *et al.* (2005).

32.3.1 Bonding mechanisms and fabric

For collapse to occur, an open structure with relatively large voids must exist together with a source of strength to hold soil particles in position, resisting shearing forces associated with the current stress environment. To achieve this, bonding between particles or grains of sufficient strength must occur which, when weakened by the addition of water and/or an additional load, allows particles to slide over one another – resulting in collapse. There are generally considered to be three main bonding mechanisms present in collapsible soils (Barden *et al.*, 1973; Clemence and Finbarr, 1981; Rogers, 1995), namely:

(i) capillary or matric suction forces (see **Figure 32.9(a)**);

(ii) clay and silt particles at coarser particle contacts (see **Figure 32.9(b-d)**);

(iii) cementing agents, such as carbonates or oxides (see **Figure 32.9(e)**).

The fabric of a collapsible soil takes the form of a loose skeleton built of grains (generally quartz in the case of loess) and micro-aggregations (in the case of loess assemblages of clay or clay and silty-clay particles, see **Figure 32.10**). In some soils, such as loess, carbonate diagenesis may strengthen the meniscus clay bridges between silt grains and further influence collapse potential. Recent observations by Milodowski *et al.* (2012) suggest that there are three variants of the clay bridges: (1) simple clay meniscus films, (2) clay films developed on a scaffold of an earlier meniscus of fibrous calcite and (3) clay films permeated or encrusted by microcrystalline calcite and/or dolomite. Hence the strength can vary both laterally and vertically over relatively short distances. This, together with the durability of the cementing material, needs to be taken into account for engineering purposes.

It should be noted that the microfabric of loess soils that have experienced reworking (cf. **Figure 32.6**) are often anisotropic and as a result exhibit much reduced collapsibility (Pye and Sherwin, 1999). In addition, as with other collapsible soils, domains of pelletised material can form and be arranged in a loose framework. Further details are given in Klukanova and Frankovska (1995), Jefferson *et al.* (2003b) and Milodowski *et al.* (2012). Derbyshire and Meng (2005) provide further discussion of fabrics associated with loess soils in China.

Other collapsible soils, such as quick clay, have essentially the same open metastable fabric, but are generated under different geomorphologic conditions. Further details are provided by Locat (1995), Bentley and Roberts (1995) and Bell (2000); see also section 32.2.2 above. Further details related to other collapsible deposits are given elsewhere in this manual (see Chapters 34 *Non-engineered fills* and 30 *Tropical soils*).

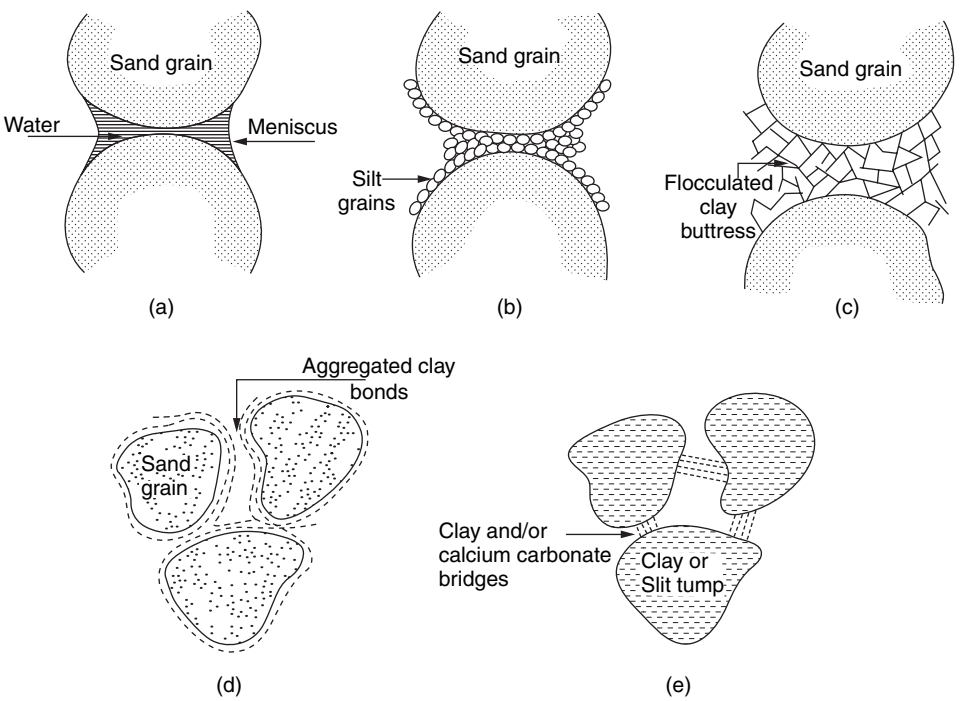


Figure 32.9 Typical bonding arrangement formed in collapsible soils
Reproduced from Popescu (1986), with permission from Elsevier

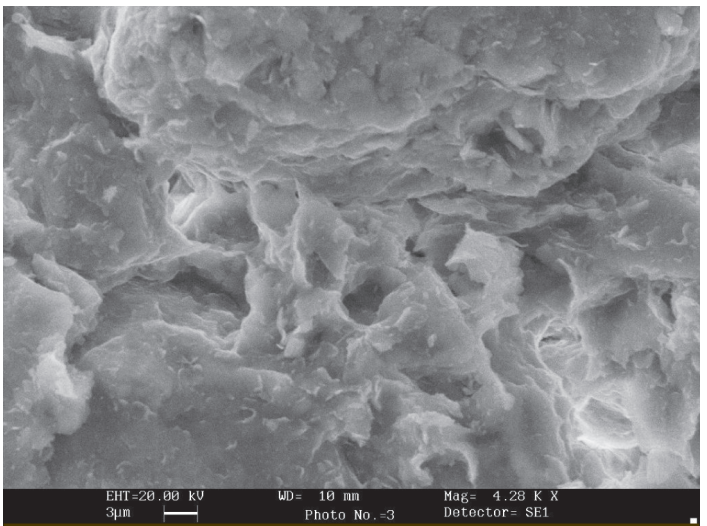


Figure 32.10 Scanning electron micrograph of bonded quartz particles in Brickearth (loess) from Ockley brickworks, Sittingbourne, Kent

Reproduced from Jefferson *et al.* (2001); all rights reserved

It should be noted that although fabric is widely recognised as important in explaining collapse behaviour, it lacks a simple quantitative descriptor (Alonso, 1993 as cited by Pereira and Fredlund, 2000).

32.3.2 Mechanisms of collapse

Collapse in cemented soils typically involves the destruction of all three bonding types. In contrast, collapse, in uncemented dry soils is solely due to the destruction of capillary forces. The strength derived from suction and cementing can be characterised in similar ways. However, on wetting suction will be lost, whereas chemical bonding is likely to be less affected by a change in suction. However, salt and clay bonds that occur at particle contacts will tend to be removed or weakened after inundation and hence collapse occurs.

Petrographic evidence indicates that collapse in cemented loess soils occurs in three stages after inundation (Klukanova and Frankovska, 1995; Milodowski *et al.*, 2012):

Stage 1 Dispersion and disruption of clay bridges or buttresses between loosely packed silt grains, leading to initial rapid collapse of inter-ped matrix.

Stage 2 Load taken up via contact between adjacent compact silt peds, which rearrange into a closer packing.

Stage 3 With increased loading, progressive deformation and shearing of peds occur, resulting in further collapse as peds disaggregate and silt particles collapse into now-unsupported inter-ped areas.

Similar observations concerning the collapse process in compacted soils have been made by Pereira and Fredlund (2000), who suggest:

Phase 1 (Pre-collapse): High matric suction generates a metastable structure that suffers small volume deformations with decreased suction. No particle slippage occurs and structure remains intact.

Phase 2 (Collapse): Intermediate matric suction and a significant decrease in volume occur, altering the structure through bond breakage.

Phase 3 (Post-collapse): Saturation is approached and no further decrease in volume (or reductions in matric suction) will occur.

Pereira and Fredlund (2000) further observed that as the net confining pressure increases, the wetting-induced collapse becomes greater and the matric suction associated with phases 1 and 3 will be higher.

During the collapse process, the nature of pores also changes. Studies on loess have shown that most of the macropores (100–500 µm) are typically destroyed, leaving smaller intergrain and interaggregate pores (8–100 µm) (Osopov and Sokolov, 1995). Similar observations with respect to collapse mechanisms were made by Klukanova and Frankovska (1995) and Feda (1995), and more recently on residual soils by Roa and Revansiddappa (2002).

Cerato *et al.* (2009) observed that compacted soils with a greater number of smaller clods showed greater collapse, with collapse largely dependent on the interaggregate and intra-aggregate pore distribution. Moreover, when clods are in a drier than optimum water content, stronger state, higher yield stresses result and the overall soil structure is less prone to collapse. In some fill materials the parent material may also lose some strength, or aggregates within the fill may soften as its water content increases – resulting in a possible collapse (Lawton *et al.*, 1992; Charles and Watts, 2001; Charles and Skinner, 2001).

For uncemented soil, collapse is related to the destruction of capillary (matric suction) forces, with water infiltration producing wetting fronts. The volume change associated with collapse is confined to the wetted zone (Fredlund and Gan, 1995). As matric suction can be visualised as isochrones (akin to excess pore water pressures seen in consolidation) analysis can be undertaken in much the same way. Further details of this, including experimental observations, are provided by Fredlund and Gan (1995). Collapse deformations that occur owing to suction reductions have been found to depend mainly on soil density and the stress state under which collapse occurs (Sun *et al.*, 2007).

Pereira and Fredlund (2000) highlighted key features in the collapse of compacted soils:

- For any type of soil compacted dry of optimum water content, collapse can occur.
- High microforces of shear strength exist through bonding, chiefly via capillary action.
- Compressibility gradually increases and shear strength gradually decreases in collapsible soils during saturation.

- Soil collapse progresses with increasing degree of saturation. However, above a critical degree of saturation, no further collapse occurs.
- Collapse is associated with localised shear failure (see further discussion in section 32.4.5 below).
- During wetting-induced collapse under constant load and anisotropic oedometer conditions, horizontal stresses increase.
- For a given mean normal total stress under triaxial conditions, the magnitude of axial collapse increases and radial collapse decreases with increased stress ratio.

Fill collapse potential is thus controlled by placement conditions, water content history and stress history. Further discussion on this subject and of other poorly compacted materials is provided by Charles and Skinner (2001), Charles and Watts (2001), and elsewhere in this manual; see Chapter 34 *Non-engineered fills*.

32.3.3 Modelling approaches – collapse prediction

Most collapsible soils exist in a partially saturated state. The suctions that develop are made up of two components: matric suction and osmotic suction, the sum of which is known as total suction. Discussion of these aspects and their implications is presented in Chapter 30 *Tropical soils* and a detailed treatment is also provided by Fredlund and Rahardjo (1993) and Fredlund (2006).

This approach has important limitations and a more complete model has been developed by Alonso *et al.* (1990). Their approach extended the Modified Cam Clay model to unsaturated soils and introduced the loading–collapse (LC) surface to define yielding due to either external loading (total stress) or saturation (loss of suction). Experimental evidence for this has been presented by a number of authors, e.g. Jotisankasa *et al.* (2009). The LC model further demonstrated the stress path dependency of collapse and explains why, under lower net stress, water inundation may cause swelling and at higher net stresses, collapse occurs. Therefore any modelling approaches used should treat soils as potentially expansive or collapsible in the same framework. However, it should be noted that collapse, unlike swelling, is an irreversible process.

The effects of bonding and bond yield strength are important aspects in many collapsible soils. Further details are discussed by Leroueil and Vaughan (1990), Maâtouk *et al.* (1995), Malandraki and Toll (1996) and Cuccovillo and Coop (1999).

Reviews of the constitutive models used to assess partially saturated soils are provided by D'Onza *et al.* (2011) and Sheng (2011), with discussion on their limitations provided by Zhang and Li (2011). Overall, constitutive models for partially saturated soils deal with the mechanical stress–strain and hydraulic suction–saturation relationships.

Constitutive models have allowed a number of numerical approaches to be developed. However, these often need a range of parameters for analysis and so may not be cost-effective for many projects. However, this can be mitigated to some degree using approaches developed by Nobar and Duncan (1972) and Farias *et al.* (1998) that utilise, for example, stress–strain curves for dry and saturated behaviour. A similar approach has been advocated by Charles and Skinner (2001) and Skinner (2001) when dealing with fill. In addition, discrete element methods (DEMs) are allowing micromechanical aspects of collapsible soils to be examined (see Liu and Sun, 2002, for further details).

32.4 Investigation and assessment

In order to provide an economical and efficient engineering solution four basic steps must be undertaken when dealing with collapsible soils (after Popescu, 1986):

(i) identification – determine whether a collapsible soils exists;

(ii) classification – if a collapsible soils exists, how significant is it?

(iii) quantification – assess the degree of collapse that will occur;

(iv) evaluation – assess the design options.

However, one of the greatest problems with collapsible soils is that their existence and the extent of their collapse potential are often not recognised prior to construction (Houston *et al.*, 2001). Thus it is essential to first identify a collapsible soil and then to estimate its collapse potential, particularly (but not exclusively) on sites containing water-sensitive soils.

Engineers often mistake, or simply do not recognise the presence of, a collapsible soil. Current standards relating to soil field descriptions, used by engineers, tend to group all fine materials together (e.g. silts and clays) under a common descriptor, which does not help in this regard. Whilst there are practical reasons for this, such groupings potentially reduce the ability of engineers to identify and assess whether a soil is collapsible. Moreover, even though a considerable database of knowledge exists globally, much of this work tends to be lost owing to use of formats and terms unfamiliar to engineers or simply suffers from language barriers (Jefferson *et al.*, 2003a).

Popescu (1986), Houston and Houston (1997) and Houston *et al.* (2001) provide excellent overviews of the key aspects associated with the identification and characterisation of collapsible soils. When characterising a collapsible soil, Houston *et al.* (2001) suggest the following stages be undertaken:

(i) reconnaissance;

(ii) use of indirect correlations;

(iii) laboratory testing;

(iv) field testing.

These aspects are, in general, common to the investigation of expansive soils and reference to discussions in Chapter 33 *Expansive soils* would be useful.

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32.4.1 Reconnaissance

Using reconnaissance to gather useful geologic and geomorphologic information can be useful in anticipating collapse by providing clues on what to look out for. The first step is to understand geologic and geomorphologic settings (see section 32.3 above). For example, Lin (1995) found that there was a strong correlation between geomorphologic information and collapsibility. It may be that the underlying assumption should be that a deposit is collapsible until confirmed otherwise – as in the case of Beckwith (1995), who recommends that alluvial fans should all be assumed to be collapsible. Charles and Watts (2001) make a similar suggestion when dealing with partially saturated fill – until there is adequate evidence to the contrary. Further clues to the likelihood of collapsible soils can be gained from prior history and environmental factors.

Lin (1995) highlighted how collapse of loess soils was influence by age, overburden pressure, and the degree of saturation and suction, illustrating the difficulties associated with determining whether or not a soil is collapsible. Popescu (1992) highlighted other aspects that need to be considered when assessing the collapsibility of soils. These include:

(i) Internal factors

- mineralogy of particles;
- shape and distribution of particles;
- nature of interparticle bonding/cementing;
- soil structure;
- initial dry density (which is often low);
- initial water content.

(ii) External factors

- availability and nature of water;
- applied pressure;
- time permitted for water percolation to occur;
- stress history;
- climate.

Thus a detailed programme of classification and quantification is necessary to fully assess collapse potential across a site.

Details of reconnaissance and its role in site investigations are dealt with elsewhere in the manual – see Chapter 45 *Geophysical exploration and remote sensing*. Further information can also be found in Fookes *et al*. (2005).

32.4.2 Indirect correlations

Various collapse coefficients related to loess have been produced, which include ever more parameters, e.g. Basma and Tuncer (1992) and Fujun *et al*. (1998). However, these are unnecessarily complex and the traditional collapse potential, for example the one described by Gibbs and Bara (1962), is better owing to its relative simplicity.

Many criteria and correlations have been proposed in the assessment of collapse potential based on soil properties such as natural water content, void ratio or index properties (see Rogers *et al.*, 1994; Northmore *et al*., 1996; Bell, 2000). However, these can be misleading as they are often based on remoulded and approximate soil properties; therefore, inappropriate evaluation can thus occur (Northmore *et al.*, 1996). Many of the correlations available have met with only moderate success owing to their weak correlations and considerable scatter (Houston *et al.*, 2001). Thus, it is more efficient and economical to use either laboratory or field tests when assessing collapse potential. This has the added advantage of providing not only identification, but also assessment data.

32.4.3 Laboratory testing

The most effective method to access collapsibility is through collapse tests. The actual collapse potential is traditionally measured using double and single oedometer tests (Jennings and Knight, 1975), which have been subsequently modified by Houston *et al.* (1988). The amount of collapse strain produced when the test specimen is flooded under a given pressure indicates a sample's susceptibility to collapse.

Figure 32.11 illustrates a typical response (Houston *et al.*, 1988), where a seating stress of 5 kPa has been used to establish an initial state. Any compression under this stress is attributed to sample disturbance. The initial compression curve A–B–C represents the response of the soil at its *in situ* water content. Pressure is applied until the stress on the sample is equal to (or greater than) that expected in the field. At this point, the sample is inundated and compression measured (C–D in **Figure 32.11**), after which further loading is undertaken, corresponding to line D–E. This equates to the three phases of collapse described by Pereira and Fredlund (2000).

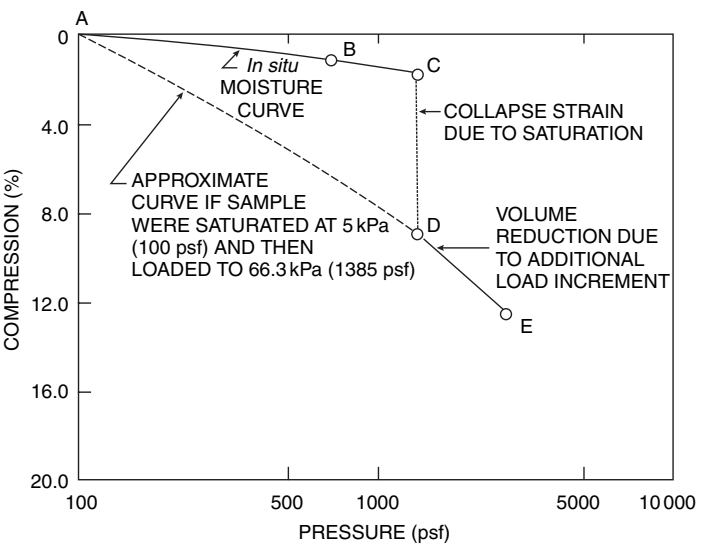


Figure 32.11 Compression curves for modified Jennings and Knight oedometer test (N.B. 1 pound per square foot (psf) = 0.0479 kPa)
Reproduced from Houston *et al.* (1988), with kind permission from ASCE

The amount of collapse of a layer is found by multiplying the thickness of the layer by the collapse strain, using values corresponding to the final stress at the midpoint of the zone in question. Further details are provided by Houston *et al.* (1988), who note that collapse strains vary both laterally and with depth, so requiring integration of strains along a vertical column of zones to estimate surface settlements.

The double oedometer test uses two near-identical specimens of soil and incremental stress increases are applied to one specimen in its natural state and to the other specimen that has been immediately saturated at under a seating stress of typically 5 kPa. Based on the double oedometer test, the degree of collapsibility can be assessed and used to provide an indication of the potential severity of collapse. **Table 32.1** provides details presented by Jennings and Knight (1975), which indicate a 1% collapse can be regarded as metastable. However, this cut-off varies across the world, with values of 1.5% taken in China (Lin and Wang, 1988) and values in excess of 2% in the USA used to indicate soils susceptible to collapse (Lutenegger and Hallberg, 1988).

Collapsible fill identification is often best achieved by testing samples at various water contents and dry densities over the range of stress levels expected (Houston *et al.*, 2001; Charles and Skinner, 2001).

Although only approximate, double oedometer tests do give a repeatable and reproducible qualitative indication of collapse. Often it is the collapsibility risk that is more important to assess than the actual amount of collapse that will occur. However, traditional oedometer tests suffer from sample disturbance effects and often reach saturations not commonly encountered in the field (Rust *et al.*, 2005). The extent of sampling effects has been debated, as have the relative merits of the use of block and tube sampling when testing collapsible soils (see Houston *et al*., 1988; Day, 1990; Houston and El-Ehwany, 1991; Neely, 2010). Northmore *et al.* (1996) observed that, upon flooding in the oedometer, certain specimens of brickearth became saturated almost instantaneously with a rapid intake of water into the pore space.

Thus, at best, traditional oedometer collapse tests should be considered as index tests; for full collapse evaluation, a field trial should be conducted. When estimating collapse settlements, significant suctions may remain after wetting and soil will remain only partially collapsed (see section 32.5.1 for further discussion). This must therefore be accounted for in any assessments.

Collapse (%)	Severity of problem
0–1	No problem
1–5	Moderate trouble
5–10	Trouble
10–20	Severe trouble
>20	Very severe trouble

Table 32.1 Collapse percentage (defined as $\Delta e/(1+e)$, where e is the void ratio) as an indication of potential severity
Data taken from Jennings and Knight (1975)

Other test methods have been employed to characterise collapsibility; these include suction-monitored oedometers (e.g. Dineen and Burland, 1995; Jotisankasa *et al.*, 2007; Vilar and Rodrigues, 2011), Rowe cells (Blanchfield and Anderson, 2000) and triaxial collapse tests (e.g. Lawton *et al.*, 1991; Pereira and Fredlund, 2000; Rust *et al.*, 2005). Further details are provided by Fredlund and Rahardjo (1993) and Rampino *et al.* (2000). Tarantino *et al.* (2011) provide a review of techniques used for measuring and controlling suctions, whilst Fredlund and Houston (2009) discuss protocols for the assessment of unsaturated soil properties in geotechnical practice. Vilar and Rodrigues (2011) provide a useful example of suction measurement in the assessment of collapse. Further details are presented in Chapter 30 *Tropical soils*.

32.4.4 Field testing

Field methods have traditionally used plate loading tests (Reznik, 1991, 1995; Rollins *et al.*, 1994) and more recently, pressuremeter tests to determine collapse potential (Smith and Rollins, 1997; Schnaid *et al.*, 2004). Francisca (2007) provides details of the use of standard penetration tests (SPTs) to evaluate the constrained modulus and collapsibility of loess in Argentina, with higher N values being recorded in soils of a lower collapse potential. However, care is needed to ensure uniformity of stress state in the collapse region. This is often the main disadvantage with *in situ* collapse tests and has led a number of researchers to develop more sensitive test methodologies.

For example, Handy (1995) devised a stepped blade method to evaluate lateral stress changes. Methods to determine response to wetting were developed by Houston *et al.* (1995b) (the downhole plate test) and Mahmoud *et al.* (1995) (the box plate load test). A brief but useful overview of interpretation and comparison of collapse measurement techniques, including their relative merits, is provided by Houston *et al.* (1995a). These include minimal sample disturbance, large soil volumes tested and degree of wetting likely to be similar to the prototype.

Houston *et al.* (2001) present field investigations of the collapse potential of a low plasticity silt using an *in situ* test apparatus (see **Figure 32.12(a)**). Boxes of concrete or steel were lowered onto a concrete pad and filled with soil, the base of the foundation was inundated with water and movements were recorded (see also Mahmoud *et al.*, 1995). The relationship between partial collapse, matrix suction and degree of saturation (**Figure 32.12(b)**) highlights the importance of understanding the likely changes in water content of the soil surrounding a structure – something which Houston *et al.* (2001) suggests can be very difficult to predict accurately.

Houston *et al.* (2001) also provide a comparison between collapse predicted for a soil investigated using *in situ* methods (not the same tests as presented in **Figure 32.12**) and laboratory methods (**Figure 32.13**), and suggest that there are difficulties involved with the *in situ* approach. The load applied can

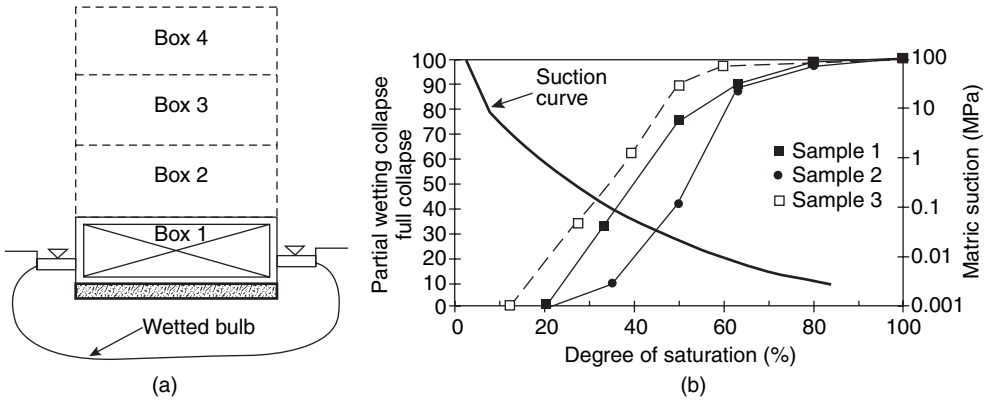


Figure 32.12 (a) Experimental layout to measure collapse of low plasticity silt; (b) the relationship between partial wetting collapse, matrix suction and degree of saturation (sample 1, 2 and 3 refer to three different silt soils of low plasticity)

Reproduced from Houston *et al.* (2001) with kind permission from Springer Science+Business Media

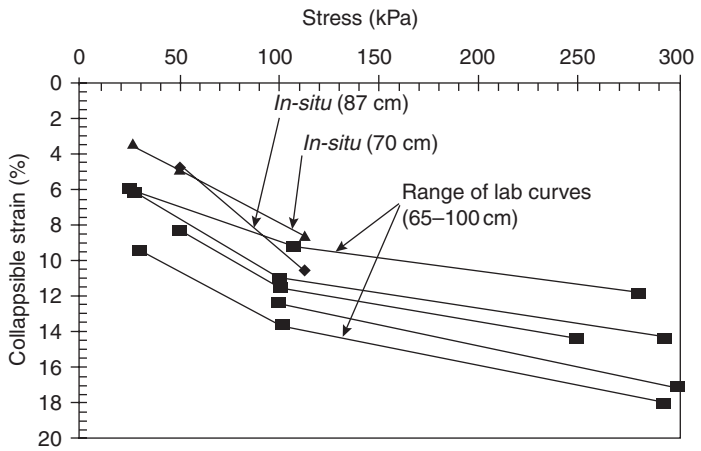


Figure 32.13 Comparison of collapsible strains for a soil investigated *in situ* and in the laboratory , using plate sizes of 70 cm and 87 cm in the plate bearing tests

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be controlled but the region affected by wetting and the degree of wetting can be very difficult to control.

Smith and Rollins (1997) investigated the use of a borehole pressuremeter to investigate the collapse potential of an arid soil. Once at the desired depth within the borehole, the pressuremeter expands radially to apply a pressure to the annulus of the borehole and measures the dry modulus of the soil (E_D). A set volume of water is discharged through the pressuremeter into the surrounding soil; the modulus during collapse (E_C) and wet modulus (E_W) post-collapse are then measured (see **Figure 32.14**). Smith and Rollins (1997) suggest that the moduli ratios E_C/E_D and E_W/E_D can be used to predict the collapsibility of a potentially collapsible soil (**Table 32.2**).

With recent improvements in technology, geophysical approaches have been advanced as a method to determine collapse potential (Evans *et al.*, 2004; Rodrigues *et al.*, 2006). The power of geophysical approaches to assess when a collapsible zone is present and its extent across a site was illustrated by Northmore *et al.* (2008), see **Figure 32.15**.

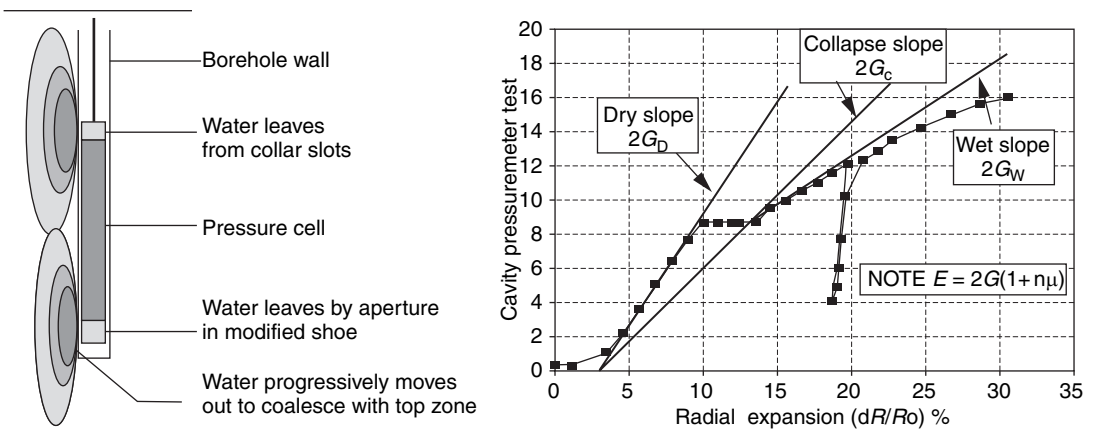


Figure 32.14 Illustration of pressuremeter (water only illustrated on left hand for clarity) and determination of the dry, collapsible and wet moduli, where *G* is shear modulus and μ is Poisson ratio

Reproduced from Smith and Rollins (1997); ASTM

Northmore *et al*. (2008) illustrated how depth and lateral extent of both collapsible and a non-collapsible loessic brickearth could be ascertained using a suite of different geophysical approaches, including electromagnetic (EM31 and EM34), electrical resistivity and shear wave profiles. Further details are provided by Gunn *et al.* (2006) for shear wave measurements and Jackson *et al.* (2006) for resistivity measurements. However, it is vital that adequate calibration is undertaken through laboratory testing. For this, traditional double oedometer and Panda probe profiling are used, where the Panda probe is a lightweight dynamic cone penetrometer which allows detailed physical soundings down to around 5 m to be taken (Langton, 1999). Once achieved, a full assessment of the presence of a potentially collapsible soil across the site can be made.

An important aspect with any field evaluation is the number of tests needed to adequately characterise the collapse potential of a loess soil. Houston *et al.* (2001) discuss statistical approaches developed to evaluate the minimum number of tests required to satisfactorily characterise a site and its collapse potential.

Criteria			Collapse potential
E_C/E_D		E_W/E_D	
≈1	and	≈1	Not collapsible
<1		≈1	Moderate collapse
≈1		<1	Moderate collapse
<1		<1	Possible severe collapse

Table 32.2 Use of modulus ratios to predict collapsibility of potentially collapsible soil
Data taken from Smith and Rollins (1997)

32.4.5 Assessment of wetting

The most challenging task for improving collapsible soils is the assessment of wetting extent and degree of potential future wetting. This is particularly true in arid or semi-arid environments where collapsible soils have not been wetted to any significant depth. However, even with collapsible deposits found in more humid environments, the deposits are often situated where significant wetting at depth has not occurred, or only part saturation has taken place, rendering the deposit still partially collapsible (Northmore *et al.*, 1996; Charles and Watts, 2001).

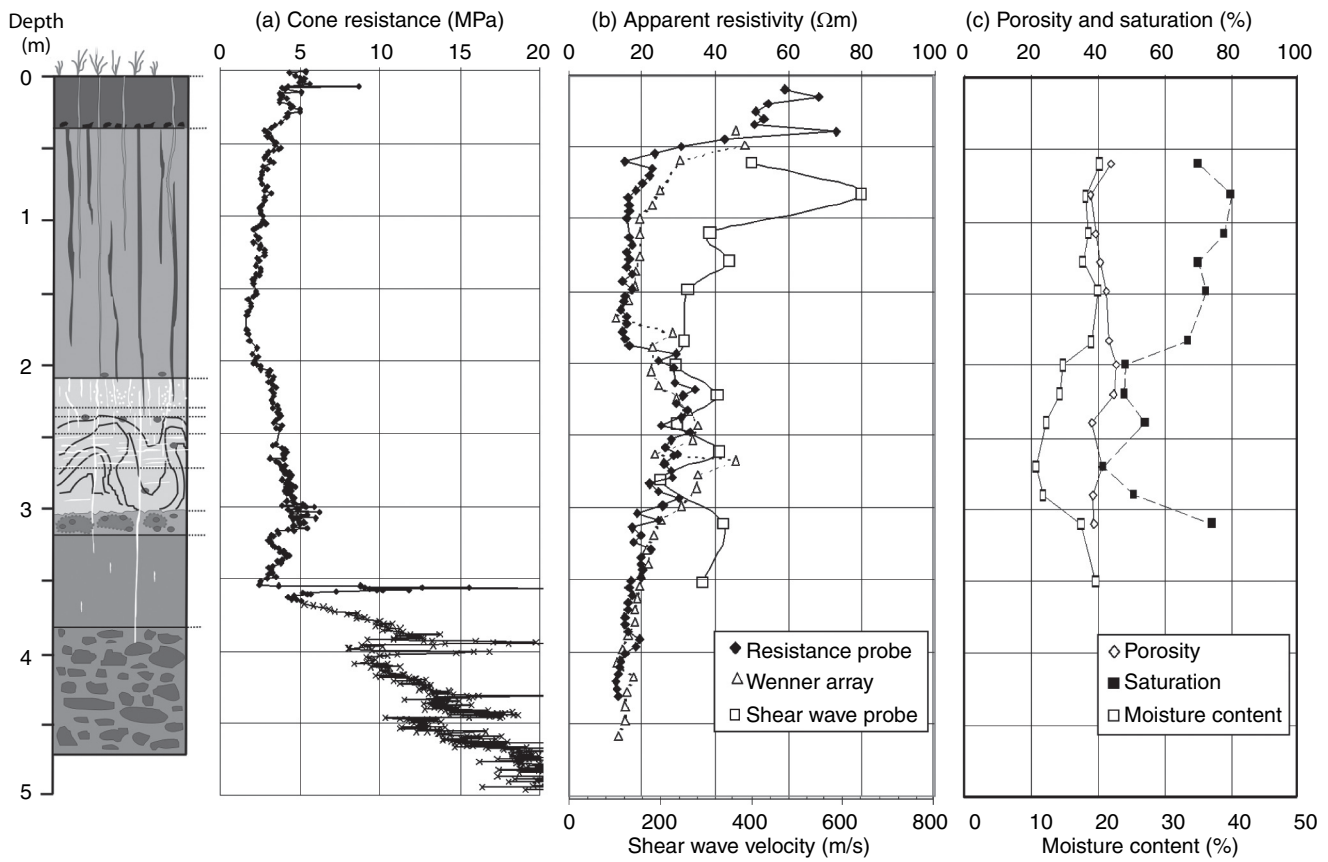


Figure 32.15 Profile through collapsible (~2–3 m) and non-collapsible (~0.5–2 m) brickearth – with collapsible deposits existing between 2 to 3 m, correlating to a drop in the degree of saturation
Reproduced from Northmore *et al.* (2008) all rights reserved

In general, detrimental effects of collapse occur in the zones both under foundations and within the probable wetting front (Houston *et al.*, 1988). Ponding tests may be used which can give an indication of depth and lateral extent of water migration, from which the best foundation option can be determined. El-Ehwany and Houston (1990) present results from laboratory infiltration wetting fronts and, by comparing these with observed rates, predict the depth of wetting versus time. They then describe how this information can be used to predict collapse settlements, taking account of partial wetting.

Clearly an assessment of the extent and degree of wetting is essential to determine collapse potential and the scope and requirements for any treatment processes. Many practitioners tend to be conservative and assume the degree of wetting equates to 100%, particularly if the collapsible zone is near to the surface and does not extend too deeply (Houston and Houston, 1997). Full wetting of a collapsible soil would only be expected with rising ground water. However, this is not often the case, and saturation only usually reaches between 35 and 60%, particularly when downward infiltration occurs. Hence the additional costs associated with such a conservative assumption may not be warranted (Houston *et al.*, 2001). El-Ehwany and Houston (1990) found that 50% saturation produces 85% of full collapse, agreeing approximately with the observations presented in **Figure 32.12 (b)**. They suggest that full collapse is achieved at between 65 and 70% saturation. Lawton *et al.* (1992) and others have shown that partial saturation will first trigger partial collapse, with full collapse occurring at saturation values as low as 60% – a figure that Bally (1988) agrees with. However, Osopov and Sokolov (1995) considered that full collapsibility would be realised only when saturation exceeded 80%.

This has implications for the prediction of collapse settlement, based on laboratory tests, which overestimate collapse strain and generally produce a greater degree of saturation than is achieved in the field; estimates put the overestimation at around 10% (El-Ehwany and Houston, 1990). However, this is not particularly large given the nature and accuracy of settlement predictions in general.

Wetting effects can be modelled using unsaturated stress state variables: net normal stress ($\sigma - u_a$) and matric suction ($u_a - u_w$) – further details can be found in Fredlund and Rahardjo (1993). The matric suction changes during wetting can be indicated by soil–water characteristic curves (SWCCs); further details are discussed in Chapter 30 *Tropical soils*. Houston *et al.* (2001) provide a range of SWCCs for collapsible loess soils from around the world. Further details of the evaluation of wetting using suction measurements have been discussed by Walsh *et al.* (1993).

All collapsible soils will experience partial collapse under partial wetting conditions. However, the shape and position of the partial collapse curve depend on the soil type, fines content and type of bonding present. Overall, it is clear that assessment of the extent and degree of wetting is the most difficult part of collapsibility evaluation.

32.5 Key engineering issues

32.5.1 Foundation options

Four basic approaches exist when dealing with design solutions in collapsible soils (Popescu, 1992):

(i) Use very stiff raft foundations and a rigid superstructure to minimise the effects of differential settlements (e.g. **Figure 32.16**). This tends to be expensive and not universally successful.

(ii) Ensure sufficient flexibility of the foundation and superstructure to accommodate ground movements without damage. This approach may be more applicable to smaller, lower cost buildings. Alternatively, a split rigid building can be flexibly connected.

(iii) Bypass the collapsible layer by use of piles.

(iv) Control or alter ground conditions through one or more of the various improvement techniques available (see section 32.5.5 and Chapter 25 *The role of ground improvement*).

Where the thickness of collapsible soil is relatively small, foundation recommendations are straightforward: the foundation level should be set below the collapsible soil layer. If this is not the case, then some form of pre-treatment is necessary to remove collapse potential.

If the collapsible layer is deep or is of significant thickness below the surface, pile foundations are used. However, Grigoryan (1997) has reported several cases where piles used in collapsible soils have experienced loss of bearing capacity and excessive settlements immediately after inundation through negative skin friction effects. In addition, the presence of a collapsible layer may adversely affect the performance of piles during the life of a building.

Kakoli (2011) provides one of the few detailed reviews and assessments of piles used in collapsible soils, drawing mainly on the work of Grigoryan and Grigoryan (1975). They suggested that in collapsible soils, negative skin friction exists for a few hours but disappears after pile settlement. Chen *et al.* (2008) present load tests for piles in collapsible soils subjected to inundation. They measured negative skin friction across five sites in China, finding values of negative skin friction between

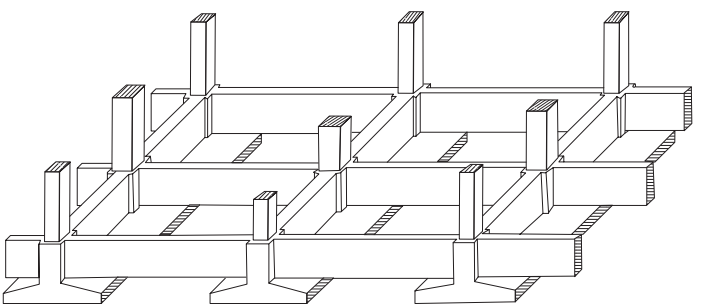


Figure 32.16 Continuous footing design for collapsible soils
Reproduced from Zeevaert (1972); Van Nostrand Reinhold Co.

around 20 and nearly 60 kPa. Kakoli (2011) presents numerical works (using PLAXIS) in the assessment of the effects of inundation on the performance of piles in collapsible soils, achieving results comparable with those presented by Chen *et al.* (2008). He further demonstrates how the effects of inundation increase collapse potential. Further details of the general behaviour of piles in collapsible soils are provided by Redolfi and Mazo (1992).

Often less expensive foundation options can be employed: Bally (1988) and Poposecu (1992) provide some examples of foundation practice in collapsible soils and Jefferson *et al.* (2005) illustrate this with a case study from Bulgaria.

32.5.2 Transport and utility infrastructure

As with foundations, transport infrastructure (roads and railways) can experience problems due to collapse. Particular problems for roads are the non-uniform collapse and non-uniform wetting of sub-grades that occur along the length of a road. These cause rough, wavy surfaces and have the potential to result in many miles of extensive damage to road structures (Houston, 1988). The potential damage to railways is more severe owing to their intolerance of longitudinal differential settlement, and the same is true for pipelines unless they are sufficiently flexible to permit longitudinal differential or lateral movements. Moreover if water pipelines or sewers suffer fracture due to differential movement, water leakage can exacerbate the collapse phenomena. Damage can also occur to associated structural and geotechnical assets, including bridges, slopes and cuttings.

Stress applied to sub-grade from highways has two components: overburden stress and imposed traffic loading (determined using common pavement analytical tools). It is likely that once wetting has occurred, short duration loading from heavy lorries would be sufficient to cause full collapse. It should be noted that collapse strains can occur at any depth at which significant wetting occurs (see section 32.4.5, above), regardless of the relative balance between overburden and imposed loads. Clearly changes in soil type along the length of a highway are important and routine non-destructive tests such as the falling weight deflectometer are useful here (Houston *et al*., 2002).

Drainage management is of particular concern, as most collapsible soils have higher permeabilities and suctions than road foundation materials. In addition, the pavement interrupts evaporation and changes water content regimes relative to the surrounding land. Hence significant risks of wetting collapsible soil sub-grades exist; the extent and depth of wetting is controlled by the sources of water, for example rising groundwater or surface ponding from poor drainage.

The approaches discussed above in section 32.4 and below in section 32.5.5 essentially apply to roads built on collapsible soils. Further details specific to roads are provided by Houston (1988) and Houston *et al.* (2002). Examples of parameters used in pavement design associated with collapsible soils are provided by Cameron and Nuntasarn (2006) and Roohnavaz *et al*. (2011).

32.5.3 Dynamic behaviour

Collapsible soils are particularly susceptible to liquefaction and dynamic settlement owing to their highly contractive nature during shearing. This can result in devastating consequences, with the Haiyuan landslide being a prime example (see Zhang and Wang, 2007). However, detection of the liquefaction and dynamic settlement potential of collapsible soils is difficult as they often have sufficient cementing when dry to prevent significant deformations during dynamic or earthquake loading (Houston *et al*., 2001). If, however, they become wetted, these bonds weaken and their liquefaction and dynamic settlement potential can significantly increase. Post-wetting behaviour is therefore of particular importance in earthquake-prone regions, especially if collapse is triggered by rising groundwater (Houston *et al.*, 2003). Moreover, these soils often have insufficient fines to render them non-liquefiable if saturated. Hence compressions induced upon wetted collapse may be inadequate to mitigate liquefaction and dynamic settlement potential (Houston *et al.*, 2001).

Houston *et al.* (2003) provide a brief overview of research that examines dynamic behaviour of collapsible soils. Their results show how cyclic stress ratios (CSRs) are strongly dependent on the degree of saturation (see **Figure 32.17**). Here, failure is taken to occur at CSR causing 10% strain. Cyclic stress ratios can be defined as the ratio of maximum shear stress (related to the cyclic shear stress amplitude of the earthquake) to vertical effective stress.

Liquefaction in loess soils is complicated by their microstructural aspects and so the process is generally less well understood than liquefaction in sands (see Hwang *et al.*, 2000; Wang *et al.,* 2004). However, improvement approaches that have proved successful in reducing liquefaction potential include dynamic compaction, use of compaction piles and methods that alter the soil's structure such as grouting (Wang *et al.*, 2004) – see section 32.5.5 for more details.

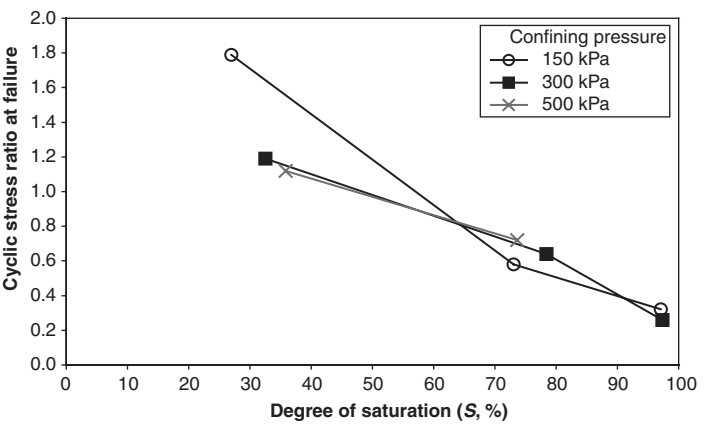


Figure 32.17 Variation of cyclic stress ratio at failure with degree of saturation
Reproduced from Houston *et al.* (2003)

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Wang *et al.* (1998, 2004) provide details of lessons learnt when dealing with earthquake-induced problems in loess soils through the use of dynamic triaxial tests and resonant column tests. Key earthquake-induced problems are seismically induced landslides (e.g. the 1920 Haiyuan landslide – Zhang and Wang, 2007); seismically induced subsidence, and liquefaction. Similar problems can occur in other collapsible soils due to earthquakes, for instance seismically induced landslides triggered in quick clays (Stark and Contreras, 1998).

32.5.4 Slope stability

Slope stability issues related to collapsible soils are a problem associated extensively with loess soils, although other collapsible deposits such as quick clay can exhibit significant problems, e.g. the Rissa landslide (see section 32.2.2 above), with further details discussed by Bell (2000).

However loess regions, such as the loess mantled mountainous region of Gansu in China, have suffered more than 40 000 large scale landslides over the last century (Meng and Derbyshire, 1998). Landslides in this region are caused by seismic shocks and severe summer monsoonal rains. As a consequence of this rainfall, loess karst and sinkholes features are additional hazards.

Landslides in loess are very diverse owing to a broad range of conditions in which they occur (Derbyshire and Meng, 2005). The principal types of landslide in loess are shown in **Figure 32.18** with detailed descriptions given in Meng and Derbyshire (1998) and Meng *et al.* (2000b). It should be noted that Tan-ta are small adjustment failures, generally less than 10 m in diameter and at a depth of a few metres, occurring predominately on steep slopes in loess. After initiation of the slide, rapid disintegration takes place, often with high sliding velocities (Meng *et al*., 2000b).

The considerable experience from China had led to success with a number of control measures. These include:

- landscaping, e.g. stepped slopes;
- retaining structures, e.g. retaining walls, underground drainage trenches and retaining piles.

Using these methods, most shallow and moderately deep landslides can be treated. Often a combination of approaches has proved to be best; for instance, using a combined structure of retaining walls and drainage ditches. However, terrace landslides can prove difficult to handle and often require a horizontal drainage borehole to alleviate groundwater pressures. Because of these pressures loess liquefaction, induced by slight slope displacement, can threaten whole slope stability.

Further details are provided by Meng and Derbyshire (1998) and Meng *et al.* (2000a), including details of successful treatment for landslide control. In addition, Dijkstra *et al.* (2000a) provide a detailed account of laboratory and *in situ* strength measurements with respect to slope stability in loess; details of modelling of landslides are presented by Dijkstra *et al.* (2000b).

32.5.5 Improvement and remediation

A wide variety of improvement processes exist for collapsible soils. Some of the more exotic ones have only been tried at an experimental stage. Evstatiev (1988), Houston *et al*. (2001)

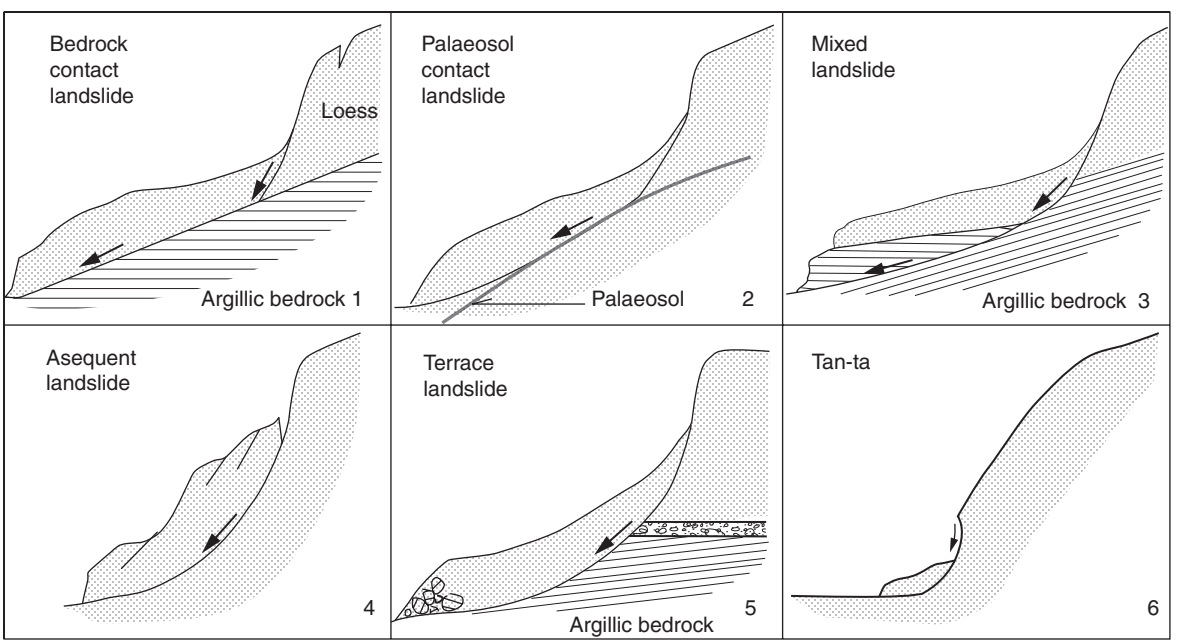


Figure 32.18 Principal types of loess landslides
Reproduced from Meng and Derbyshire (1998) © The Geological Society of London

and Jefferson *et al*. (2005) provide excellent overviews of a range of possible treatment techniques used to improve collapsible loess soils.

Ultimately the best technique depends on several factors (after Houston *et al.*, 2001):

(i) when the collapsible loess soil was discovered;

(ii) how stress is to be applied to the soil;

(iii) depth and extent of the collapsible zone;

(iv) sources of wetting;

(v) costs.

Details are provided throughout the literature, in particular the experience from Eastern Europe, especially Russia, e.g. Abelev (1975), Lutenegger (1986), Ryzhov (1989), Evstatiev (1995) and Evstatiev *et al.* (2002). Deng (1991), Wang (1991), Zhong (1991), Zhai *et al.* (1991), Fujun *et al.* (1998) and Gao *et al*. (2004), amongst others, allow insights into treatment techniques commonly employed in China, while researchers such as Clemence and Finbarr (1981), Rollins and Rogers (1994), Rollins and Kim (1994), Pengelly *et al.* (1997), Houston and Houston (1997), Rollins *et al*. (1998), Houston *et al.* (2001) and Rollins and Kim (2011) provide reviews of ground improvement approaches used to treat collapsible soils in North America. Rollins and Rogers (1994) provide an overview of the various advantages and limitations of a number of different possible treatment methods to reduce/eliminate collapsibility.

Table 32.3 provides an overview of techniques that can be used to treat loess ground and reduce/remove its collapse potential.

It is very common to find bands of clayey material (palaeosol) within some collapsible soils, such as loess deposits. However, it still uncertain what role these bands play in the overall behaviour of the loess mass. It is likely that they will act as planes of weakness. What is certain, though, is that the presence of these bands will influence the effectiveness of any method used to treat a collapsible soil. These bands will affect the transmission of compaction energy and the route taken by stabilising fluids through the soil.

Post-construction treatment typically involves some form of chemical stabilisation – typically grouting – or alternatively some form of underpinning, both of which can prove expensive. The following methods have been used in cases where collapsing loess has damaged existing buildings: silicate grout injection (silicatisation), jet-grouting, underpinning by root piles (pali radici), squeeze grouting (injection by 'tube à manchettes') and stabilisation by in-depth heating.

An alternative proposed by Houston *et al*. (2001) involves controlled differential wetting via separately controllable trenches built around the foundation slab. This approach has been used to tilt a structure in a controlled way. Initial trials demonstrated that, firstly, it was possible to re-level the foundation thereby eliminating any future collapse potential, and, secondly, its control was relatively straightforward by allowing site owners to control flow rates from each trench. However,

Depth (m)	Treatment method	Comments
0–1.5	Surface compaction via vibratory rollers, light tampers	Economical, but requires careful site control, e.g. limits on water content
	Pre-wetting (inundation)	Can effectively treat thicker deposits but needs large volumes of water and time
	Vibrofloatation	Needs careful site control
1.5–10	Vibrocompaction (stone columns, concrete columns, encased stone columns)	Cheaper than conventional piles but requires careful site control and assessment. If uncased, stone columns may fail with loss of lateral support on collapse
	Dynamic compaction; rapid impact compaction	Simple and easily understood but requires care with water content and vibrations produced
	Explosions	Safety issues need to be addressed
	Compaction pile	Needs careful site control
	Grouting	Flexible but may adversely affect the environment
	Ponding/inundation/pre-wetting	Difficult to control effectiveness of compression produced
	Soil mixing lime/cement	Convenient and gains strength with time. Various environmental and safety aspects, and the chemical controls on reactions need to be assessed
	Heat treatment	Expensive
	Chemical methods	Flexible; relatively expensive
>10	As for 1.5–10 m, some techniques may have a limited effect	(See above)
	Pile foundations	High bearing capacity but expensive

Table 32.3 Methods of treating collapsible loess ground
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as yet there are few directly relevant precedents, which limit its take-up.

A number of case studies illustrate how collapsibility has been successfully treated and details of these are provided by Evans and Bell (1981), Jefferson *et al*. (2005) and Rollins and Kim (2011).

32.6 Concluding remarks

Collapsible soils are found throughout the world and are formed through various geomorphologic and geologic processes. These can be natural (through fluvial or aeolian processes) or man-made (via poor compaction). Whatever the processes involved, the key prerequisite is that an open metastable structure develops through bonding mechanisms generated via capillary forces (suctions) and/or through cementing materials such as clay or salts. Collapse occurs when net stresses (via loading or saturation) exceed the yield strength of the bonding material. Inundation is by far the most common cause of collapse and can be triggered through a range of different water sources. Different sources yield different amounts of collapse.

Thus a detailed knowledge of macroscopic and microscopic characteristics is vital to engineer these materials effectively and safely. Failure to recognise and deal with collapsible soils can have a significant impact on the built and urban environments – with catastrophic effects and potential loss of life. To engineer effectively in collapsible soils it is essential to recognise their existence, for which key geologic and geomorphologic information is vital. However, collapsibility should be confirmed through direct response to wetting/loading tests using laboratory and field methods. The key challenge with collapsible soils is to predict the extent and degree of wetting that will take place. Care is needed to ensure that appropriate and realistic assessments are undertaken, followed by treatment through a suite of the possible improvement techniques available. The collapsible potential can then be eliminated effectively.

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32.7.1 Further reading

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32.7.2 Useful websites

British Geological Survey; www.bgs.ac.uk

It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 14 *Soils as particulate materials*
- Chapter 40 *The ground as a hazard*
- Chapter 58 *Building on fills*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set in out Section 4 *Site investigation.*

Chapter 33
Expansive soils

Lee D. Jones British Geological Survey, Nottingham, UK
Ian Jefferson School of Civil Engineering, University of Birmingham, UK

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Expansive soils present significant geotechnical and structural engineering challenges the world over, with costs associated with expansive behaviour estimated to run into several billion pounds annually. Expansive soils are those which experience significant volume changes associated with changes in water content. These volume changes can either be in the form of swell or shrinkage, and are sometimes known as swell–shrink soils. Key aspects that need identification when dealing with expansive soils include soil properties, suction/water conditions, temporal and spatial water content variations that may be generated, for example, by trees, and the geometry/stiffness of foundations and associated structures. Expansive soils can be found both in humid environments where expansive problems occur with soils of high plasticity index, and in arid/semi-arid soils where soils of even moderate expansiveness can cause significant damage. This chapter reviews the nature and extent of expansive soils, highlighting key engineering issues. These include methods to investigate expansive behaviour both in the field and the laboratory, and the associated empirical and analytical tools to evaluate expansive behaviour. Design options for pre- and post-construction are highlighted for both foundations and pavements, together with methods to ameliorate potentially damaging expansive behaviour.

33.1 What is an expansive soil?

Essentially, expansive soil is one that changes in volume in relation to changes in water content. The focus here is on soils that exhibit significant swell potential and, in addition, shrinkage potential. There are a number of cases where expansion can occur because of chemically induced changes (e.g. swelling of lime-treated sulfate soils). However, many soils that exhibit swelling and shrinking behaviour contain expansive clay minerals, such as smectite, that absorb water. The more of this clay a soil contains, the higher its swell potential and the more water it can absorb. As a result, these materials swell and thus increase in volume when they become wet, and shrink when they dry. The more water they absorb, the more their volume increases – for the most expansive clays expansions of 10% are not uncommon (Chen, 1988; Nelson and Miller, 1992). It should be noted that other soils exhibit volume change characteristics with changes in water content, e.g. collapsible soils, and these are dealt with in Chapter 32 *Collapsible soils*.

The amount by which the ground can shrink and/or swell is determined by the water content in the near-surface zone. Significant activity usually occurs to about 3 m depth, unless this zone is extended by the presence of tree roots (Driscoll, 1983; Biddle, 1998). Fine-grained clay-rich soils can absorb large quantities of water after rainfall, becoming sticky and heavy. Conversely, they can also become very hard when dry, resulting in shrinking and cracking of the ground. This hardening and softening is known as 'shrink–swell' behaviour. The effects of significant changes in water content on soils with a high shrink–swell potential can be severe on supporting structures.

Swelling and shrinkage are not fully reversible processes (Holtz and Kovacs, 1981). The process of shrinkage causes cracks which, on re-wetting, do not close up perfectly and hence cause the soil to bulk out slightly, and also allow enhanced access to water for the swelling process. In geological timescales, shrinkage cracks may become in-filled with sediment, thus imparting heterogeneity to the soil. When material falls into cracks, the soil is unable to move back – resulting in enhanced swelling pressures.

The primary problem with expansive soils is that deformations are significantly greater than those that can be predicted using classical elastic and plastic theory. As a result, a number of different approaches have been developed to predict and engineer expansive soils, and these are highlighted throughout this chapter.

33.2 Why are they problematic?

Many towns, cities, transport routes and buildings are founded on clay-rich soils and rocks. The clays within these materials may be a significant hazard to engineering construction due to their ability to shrink or swell with changes in water content. Changing water content may be due to seasonal variations (often related to rainfall and the evapotranspiration of vegetation), or be brought about by local site changes such as leakage from water supply pipes or drains, changes to surface drainage and landscaping (including paving), or following the planting, removal or severe pruning of trees or hedges, as man is unable to supply water to desiccated soil as efficiently as a tree originally extracted it through its root system (Cheney, 1988). During a long dry period or drought, a persistent water deficit may develop causing the soil to dry out to a greater

depth than normal, leading to long-term subsidence. This is why expansive problems are often found in arid environments (see Chapter 29 *Arid soils*). As this water deficit dissipates it is possible that long-term heave may occur.

In the UK, the effects of shrinkage and swelling were first recognised by geotechnical specialists following the dry summer of 1947, and since then the cost of damage due to the shrinking and swelling of clay soils in the UK has risen dramatically. After the drought of 1975/76, insurance claims came to over £50 million. In 1991, after the preceding drought, claims peaked at over £500 million. Over the past 10 years the adverse effects of shrink–swell behaviour have cost the economy an estimated £3 billion, making it the most damaging geohazard in Britain today. The Association of British Insurers has estimated that the average cost of shrink–swell related subsidence to the insurance industry stands at over £400 million annually (Driscoll and Crilly, 2000). In the US, the estimated damage to buildings and infrastructure exceeds $15 billion annually. The American Society of Civil Engineers estimates that one in four homes have some damage caused by expansive soils. In a typical year, expansive soils cause a greater financial loss to property owners than earthquakes, floods, hurricanes and tornadoes combined (Nelson and Miller, 1992).

Swelling pressures can cause heaving, or lifting, of structures whilst shrinkage can cause differential settlement. Failure results when the volume changes are unevenly distributed beneath the foundation. For example, water content changes in the soil around the edge of a building can cause swelling pressure beneath the perimeter of the building, while the water content of the soil beneath the centre remains constant. This results in a failure known as 'end lift' (**Figure 33.1**). The opposite of this is 'centre lift', where swelling is focused beneath the centre of the structure or where shrinkage takes place under the edges.



Figure 33.1 Structural damage to house caused by 'end lift'
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Damage to foundations in expansive soils commonly results from tree growth. This occurs in two principal ways: (i) physical disturbance of the ground, and (ii) shrinkage of the ground by removal of water. Physical disturbance of the ground caused by root growth is often seen as damage to pavements and broken walls. An example of vegetation-induced shrinkage causing differential settlement of building foundations is provided in **Figure 33.2**. Vegetation-induced changes to water profiles can also have a significant impact on other underground features, including utilities. Clayton *et al*. (2010), reporting monitoring data over a two-year period of pipes in London Clay, found significant ground movements (both vertical and horizontal) of the order of 3–6 mm/m length of pipe, which generated significant tensile stresses when in the vicinity of trees. Such tree-induced movement has the potential to be a significant contributor to failure of old pipes located in clay soils near deciduous trees (Clayton *et al*., 2010). Further details are discussed in section 33.5.4.5.

33.3 Where are expansive soils found?

In the UK, towns and cities built on clay-rich soils most susceptible to shrink–swell behaviour are found mainly in the southeast of the country (**Figure 33.3**). In the southeast, many of the clay formations are too young to have been changed into stronger mudstones, leaving them still able to absorb and lose moisture. Clay rocks elsewhere in the country are older and



Figure 33.2 Example of differential settlement due to influence of trees

have been hardened by processes resulting from deep burial and are less able to absorb water. Some areas (e.g. around The Wash, northwest of Peterborough – see **Figure 33.3**) are deeply buried beneath other (superficial) soils that are not susceptible to shrink–swell behaviour. However, other superficial deposits such as alluvium, peat and laminated clays can also be susceptible to soil subsidence and heave (e.g. in the Vale of York, east of Leeds – see **Figure 33.3**).

Expansive soils are found throughout many regions of the world, particularly in arid and semi-arid regions, as well as

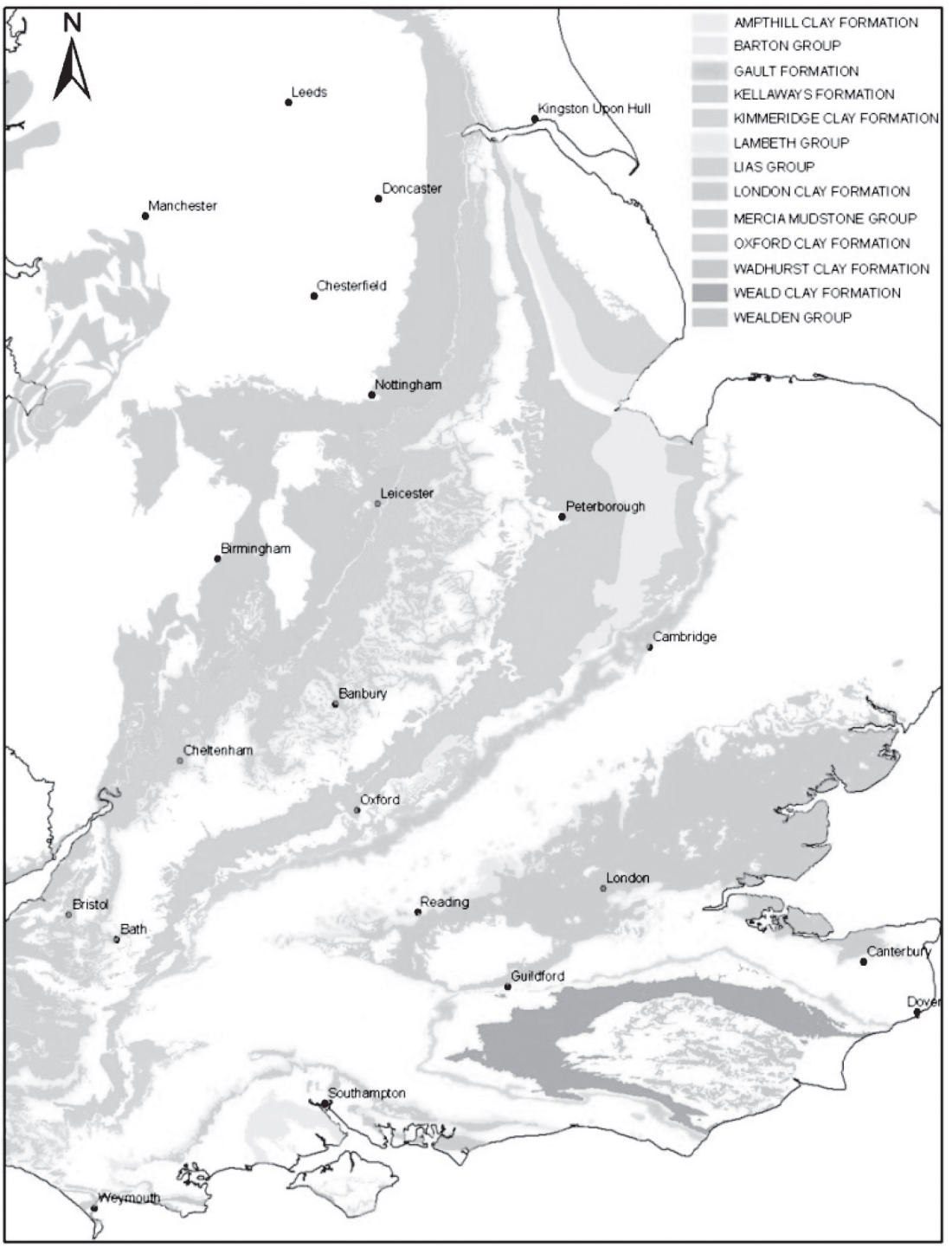


Figure 33.3 Distribution of UK clay-rich soil formations. A colour version of this figure is available online

those where wet conditions occur after prolonged periods of drought. Their distribution is dependent on geology (parent material), climate, hydrology, geomorphology and vegetation.

The literature is full of studies from all over the world, concerned with problems associated with expansive clays (e.g. Simmons, 1991; Fredlund and Rahardjo, 1993; Stavridakis, 2006; Hyndman and Hyndman, 2009). Expansive soils incur major construction costs around the world, with notable examples found in the USA, Australia, India and South Africa, to name but a few. In these countries, or significant areas of them, the evaporation rate is higher than the annual rainfall so there is usually a moisture deficiency in the soil. Subsequently, when it rains, the ground swells and so increases the potential for heave to occur. In semi-arid regions a pattern of short periods of rainfall followed by long dry periods (drought) can develop, resulting in seasonal cycles of swelling and shrinkage.

Due to the global distribution of expansive soils, many different ways to tackle the problem have been developed and these can vary considerably (Radevsky, 2001). The methods to deal with the problem of expansive soils differ in many ways and depend not only on technical developments, but also on the legal framework and regulations of a country, insurance policies and the attitude of insurers, experience of the engineers and other specialists dealing with the problem, and importantly the sensitivity of the owner of the property affected. In the UK in particular, there is high sensitivity to relative small cracks (see section 33.5.3, below). A summary of these issues is provided by Radevsky (2001) in his review of how different countries deal with expansive soil problems, and a detailed informative study from Arizona, USA has more recently been presented by Houston *et al*. (2011). The latter study demonstrated how the source of problems from expansive soils often stems from poor drainage, construction problems, homeowner activity and its adverse effects, and landscaping through the use of vegetation, or a combination of these. These aspects may cause more expansive soil problems than landscape type itself.

Overall, in humid climates, problems with expansive soils tend to be limited to those soils containing higher plasticity index (I_p) clays. However, in arid/semi-arid climates, soils that exhibit even moderate expansiveness can cause distress to residential property. This stems directly from their relatively high suction and the larger changes in water content that result when water levels change.

33.4 Shrink–swell behaviour

Excluding deep underground excavations (e.g. tunnels), shrinkage and swelling effects are restricted to the near-surface zone. Significant activity usually occurs to about 3 m depth, but this can vary depending on climatic conditions. The shrink–swell potential of expansive soils is determined by its initial water content, void ratio, internal structure and vertical stresses, as well as the type and amount of clay minerals in the soil (Bell and Culshaw, 2001). These minerals determine the natural expansiveness of the soil, and include smectite, montmorillonite, nontronite, vermiculite, illite and chlorite. Generally, the more of these minerals that are present in the soil, the greater the expansive potential. However, these expansive effects may become diluted by the presence of other non-swelling minerals such as quartz and carbonate (Kemp *et al*., 2005).

The key aspect of expansive soils behaviour is the soil vulnerability of water-induced volume change. When soils with a high expansive potential are present, they will usually not cause a problem as long as their water content remains relatively constant. This is largely controlled by (Houston *et al*., 2011):

- soil properties, e.g. mineralogy;
- suction and water conditions;
- water content variations, both temporally and spatially;
- geometry and stiffness of a structure, in particular its foundation.

In a partially saturated soil, changes in water content, or suction (increasing strength of the soil due to negative pore water pressures), significantly increase the chances of damage occurring. Changes in soil suction occur due to water movement through the soil due to evaporation, transpiration or recharge, which are often significantly influenced by interaction with trees through response to dry/wet periods of weather (Biddle, 2001). In a fully saturated soil, the shrink–swell behaviour is controlled by the clay mineralogy.

33.4.1 Mineralogical aspect of expansive soils

Clay particles are very small and their shape is determined by the arrangement of the thin crystal lattice layers that they form, along with many other elements which can become incorporated into the clay mineral structure (hydrogen, sodium, calcium, magnesium, sulfur). The presence and abundance of these dissolved ions can have a large impact on the behaviour of the clay minerals. In an expansive clay, the molecular structure and arrangement of these clay crystal sheets have a particular affinity to attract and hold water molecules between the crystalline layers in a strongly bonded ‘sandwich’. Because of the electrical dipole structure of water molecules, they have an electro-chemical attraction to the microscopic clay sheets. The mechanism by which these molecules become attached to each other is called adsorption. The clay mineral montmorillonite, part of the smectite family, can adsorb very large amounts of water molecules between its clay sheets, and therefore has a large shrink–swell potential. For further details of mineralogy of clay minerals and their influence of engineering properties of soils, see Mitchell and Soga (2005).

When potentially expansive soils become saturated, more water molecules are absorbed between the clay sheets, causing the bulk volume of the soil to increase, or swell. This same process weakens the inter-clay bonds and causes a reduction in the strength of the soil. When water is removed, by evaporation or gravitational forces, the water between the clay sheets is released, causing the overall volume of the soil to decrease, or shrink. As this occurs, features such as voids or desiccation cracks can develop.

Potentially expansive soils are initially identified by undertaking particle size analyses to determine the percentage of fine particles in a sample. Clay-sized particles are considered to be less than 2 μm (although this value varies slightly throughout the world), but the difference between clays and silts is more to do with origin and particle shape. Silt particles (generally comprising quartz particles) are products of mechanical erosion, whereas clay particles are products of chemical weathering and are characterised by their sheet structure and composition.

33.4.2 Changes to effective stress and role of suctions

Following any reduction in *total* stress, deformations will take place in the ground. A distinction can be made between (i) an immediate, but time-dependent elastic rebound, and (ii) swelling due to effective stress changes. In soils, as in rocks, rebound can be an important deformation process which encourages stress relief fractures and zones of secondary permeability which can localise delayed swelling. The amount of deformation depends on the undrained stiffness of the soil, which is equivalent to the modulus of elasticity for the soil, as reflected by its Young's modulus and Poisson's ratio. Subsequent swelling requires an *effective* stress decrease, and a movement of fluid into a geological formation or soil. The magnitude of strains associated with these processes depends on the drained stiffness, the extent of the stress change, the resulting water pressures in the soil or rock, and the new boundary conditions. The rate of volume change depends on the compressibility, expansibility and hydraulic conductivity of the sediment and surrounding materials. In stiff homogeneous materials with a low hydraulic conductivity, several decades may be necessary to complete the process.

Accurate laboratory measurements of the controlling elastic properties at small strains in both rebound and swelling (i.e. before yield takes place) are difficult, largely because of sampling disturbance (Burland, 1989). Further discussion of these difficulties, states of stress, and the other important concepts of consolidation/swelling in soils are treated in detail by many standard soil engineering texts (Powrie, 2004; Atkinson, 2007) – see also Section 2 *Fundamental principles* of this manual.

Shrinkage by evaporation is similarly accompanied by a reduction in water pressure and development of negative capillary pressures. Deformation follows the same principles of effective stress. However Bishop *et al*. (1975) have shown by laboratory studies that the degree of saturation of unconfined dried clay samples at a given water content was less than for a similar sample consolidated in a triaxial test to the same water content, i.e. there was some intake of air which affected both the modulus and strength of the soil. This process leads to a void ratio which is higher than for a clay consolidated to the same water content by simply increasing the confining load. Such a soil thus becomes inherently unstable and, if re-wetted, may collapse. Subsequent laboratory tests on partially saturated soils have shown that depending on their *in situ* stress conditions and fabric, some samples may also first swell and then collapse (Alonso *et al*., 1990). The processes of shrinkage due to evaporation have also been reviewed in detail using effective stress concepts by Sridharan and Venkatappa (1971).

33.4.3 Seasonal variations in water content

The seasonal volumetric behaviour of a desiccated soil is complex and this increases with severity of the shrinkage phenomena. This is reflected by the vertical *in situ* suction profile, water content profile and the degree of saturation (see **Figure 33.4**).

The relative values of suction depend on the composition of the soil, particularly its particle size and clay mineral content. The hydraulic conductivity of a soil may also vary both seasonally and over longer timescales. Secondary permeabilities can be induced through fabric changes, tension cracking and shallow shear failure during the swelling and shrinkage process which may influence subsequent moisture movements. For example, Scott *et al*. (1986) have shown in a microfabric study of clay soils that compression (swelling) cracks tended to run parallel to ground contours and dip into the slope at around 60º, and could usually be distinguished from shrinkage cracks which were randomly distributed. In the London Clay soils studied, for example, they found that the ratio between shrinkage and swelling discontinuities was about 2:1. Although not discussed, it seems likely that the nature and distribution of

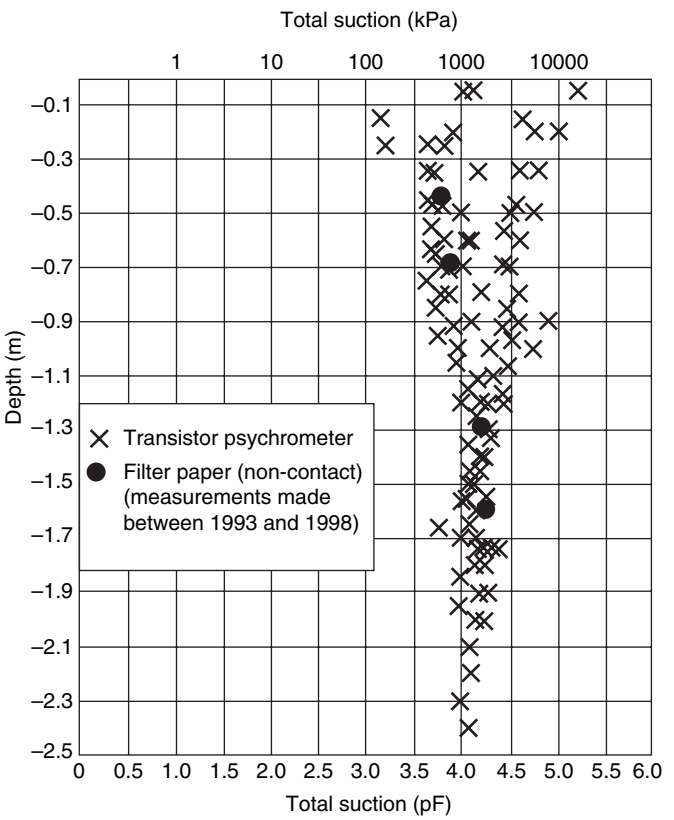


Figure 33.4 Examples of total suction profile
Reproduced from Fityus *et al*. (2004), with kind permission from ASCE

these discontinuities will also influence bulk volumetric seasonal strains.

Expansive soil problems typically occur due to water content changes in the upper few metres, with deep-seated heave being rare (Nelson and Miller, 1992). The water content in these upper layers is significantly influenced by climatic and environmental factors and is generally termed the zone of seasonal fluctuations, or active zone, as shown in **Figure 33.5**.

In the active zone, negative pore water pressures exist. However, if excess water is added to the surface or if evapotranspiration is eliminated, then water contents increase and heave will occur. Migration of water through the zone is also influenced by temperature, as shown in **Figure 33.5**, with further details provided by Nelson *et al*. (2001). Thus it is important to determine the depth of the active zone during a site investigation. This can vary significantly with different climatic conditions – it may be 5–6 m in some countries, but typically in the UK it is 1.5–2 m (Biddle, 2001). If the drying is greater than the rehydration, then the depth of this zone will increase, with 3–4 m having been observed in some cases in London Clay (Biddle, 2001). These effects are likely to become more significant with climate change.

The term 'active zone' can have different meanings. Nelson *et al*. (2001) provide four definitions for clarity:

1. *Active zone* The zone of soil that contributes to soil expansion at any particular time.
2. *Zone of seasonal moisture fluctuation* The zone in which water content changes due to climatic changes at the ground surface.
3. *Depth of wetting* The depth that water contents have reached owing to the introduction of water from external sources.
4. *Depth of potential heave* The depth at which the overburden vertical stress equals or exceeds the swelling pressure of the soil. This is the maximum depth of the active zone.

The depth of wetting is particularly important as it is used to estimate heave by integrating the strain produced over the zone in which water contents change (Walsh *et al*., 2009). Details of how this can be achieved and the relative merits of regional and site-specific approaches are considered in detail for a post-development profile by Walsh *et al*. (2009), with further discussion presented by Nelson *et al*. (2011); Aguirre (2011); and Walsh *et al*. (2011).

33.5 Engineering issues

As has been previously stated, many towns, cities, transport routes, services and buildings are founded on expansive soils. These may be solid (bedrock) geological strata in a weathered or unweathered condition, or superficial (drift) geological strata such as glacial or alluvial material, also in a weathered or unweathered condition. These materials constitute a significant

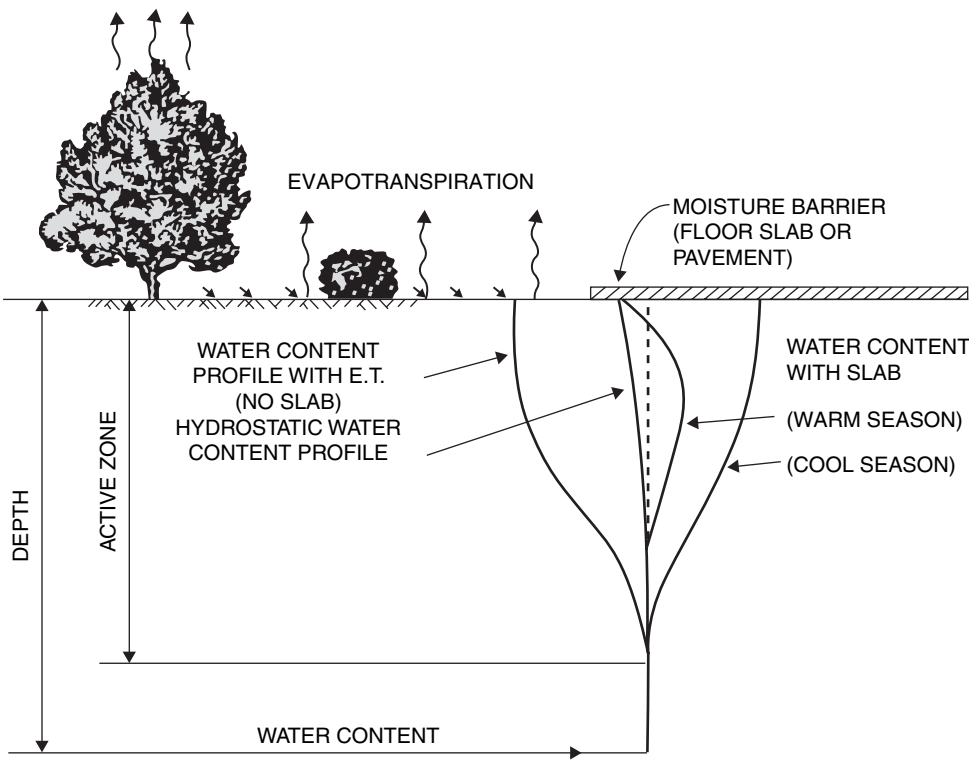


Figure 33.5 Water content profiles in the active zone
Reproduced from Nelson and Miller (1992); John Wiley & Sons, Inc

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hazard to engineering construction in terms of their ability to swell or shrink, usually caused by seasonal changes in moisture content. Superimposed on these widespread climatic influences are local ones, such as tree roots and leakage from water supply pipes and drains. The swelling of shrinkable clay soils after trees have been removed can produce either very large uplifts or very large pressures (if confined), and the ground's recovery can continue over a period of many years (Cheney, 1988). It is the differential, rather than the total, movement of the foundation, or superstructure, that causes major structural damage. The structures most affected by expansive soils include the foundations and walls of residential and other low-rise buildings, pipelines, pylons, pavements and shallow services. Frequently, these structures only receive a cursory site investigation, if any. It is usually sometime after construction that problems come to light. Damage can occur within a few months of construction, develop slowly over a period of 3–5 years, or remain hidden until something happens that changes the water content of the soil.

Houston *et al*. (2011) examined the type of wetting that occurs in response to irrigation patterns. They observed that deeper wetting was common with irrigation of heavily turfed areas, and that if ponding of water occurred at the surface, there was more likely to be greater distress to buildings through differential movements. Walsh *et al*. (2009) also note that when heave is deep seated, differential movements are less significant than when the source of heave is at shallower depths.

The structures most susceptible to damage caused by expansive soils are usually lightweight in construction. Houses, pavements and shallow services are especially vulnerable because they are less able to suppress differential movements than heavier multi-story structures. For more information about design parameters and construction techniques for housing and pavements, reference should be made to:

- *NHBC Standards: Building near trees* (NHBC, 2011a)
- *Preventing foundation failures in new dwellings* (NHBC, 1988)
- *Planning Policy Guidance Note 14: Development on unstable land: Annex 2: subsidence and planning* (DTLR, 2002)
- *BRE Digests 240– 242: Low-rise buildings on shrinkable clay soils* (BRE, 1993a)
- *BRE Digest 298: The influence of trees on house foundations in clay soils* (BRE, 1999)
- *BRE Digest 412: The significance of desiccation* (BRE, 1996)
- *Criteria for selection and design of residential slabs-on-ground* (BRAB, 1968)
- *Evaluation and control of expansive soils* (TRB, 1985).

In many respects, engineering in expansive soils is still based on experience and soil characterisation, and so is often perceived as difficult and expensive (especially for lightweight structures). Engineers use local knowledge and empirically derived procedures, although considerable research has been done on expansive soils – for instance, the database on performance (Houston *et al*., 2011). However, through careful consideration of key aspects associated with expansive soils, problems and difficulties can be dealt with in a cost effective way.

Two major factors must be identified in the characterisation of a site where a potentially expansive soil exists:

- the properties of the soil (e.g. mineralogy, soil water chemistry, suction, soils fabric);
- environmental conditions that can contribute to changes in water content of the soil, e.g. water conditions and their variations (climate, drainage, vegetation, permeability, temperature), and stress conditions (history and *in situ* conditions, loading and soil profile).

Normal non-expansive site investigations are often not adequate and a more extensive examination is required to provide sufficient information. This may involve specialist test programs, even for relatively lightweight structures (Nelson and Miller, 1992). Although there are a number of methods available to identify expansive soils, each with their relative merits, there are no universally reliable ones. Moreover, expansiveness has no direct measure and so it is necessary to make comparisons, measured under known conditions, as a means to express expansive behaviour (Gourley *et al*., 1993). However, the stages of investigation needed for expansive soils follow those used for any site (see Section 4 *Site investigation* for further details).

33.5.1 Investigation and assessment

It is important to recognise the existence, and understand the potential problems, of expansive soils early on during site investigation and laboratory testing, to ensure that the correct design strategy is adopted before costly remedial measures are required. It is equally important that investigations determine the extent of the active zone.

Despite the proliferation of test methods for determining shrinkage or swelling properties, they are rarely employed in the course of routine site investigations in the UK. Further details of tests commonly employed around the world are given by Chen (1988) and Nelson and Miller (1992). This means that few datasets are available for databasing the directly measured shrink–swell properties of the major clay formations, and reliance has to be placed on estimates based on index parameters, such as liquid limit, plasticity index and density (Reeve *et al*., 1980; Holtz and Kovacs, 1981; Oloo *et al*., 1987). Such empirical correlations may be based on a small dataset, using a specific test method, and at only a small number of sites. Variation of the test method would probably lead to errors in the correlation. The reason for the lack of direct shrink–swell test data is that few engineering applications have a perceived requirement for these data for design or construction.

33.5.1.1 Site investigation

A key difficulty with expansive soils is that they often exhibit significant variability from one location to another (i.e. spatial variability). These proper, adequate, site investigations in areas of potentially expansive soil are often worth the cost. Essential

to the investigation of any expansive soils is a good knowledge of local geology: the use of maps provides a framework for this. These maps are particularly useful when constructing transportation networks. In some countries such as the US, mapping includes identification of expansive soil potential (Nelson and Miller, 1992). As with any site investigation, field observations and reconnaissance can provide valuable data of the extent and nature of expansive soils and their associated problems. Some key features may be observed locally and important observations include:

1. *Soil characteristics*
   - spacing and width of wide or deep shrinkage cracks;
   - high dry strength and low wet strength – high plasticity soil;
   - stickiness and low trafficability when wet;
   - shear surfaces have glazed or shiny appearance.
2. *Geology and topography*
   - undulating topography;
   - evidence of low permeability by surface drainage and infiltration features.
3. *Environmental conditions*
   - vegetation type;
   - climate.

Sampling in expansive soils is generally done in the same way as for conventional soils, with care taken to minimise disturbances through, for example, water content changes or poor control during transportation. Further details are provided in Section 4 *Site investigation* of this manual, and an overview of practices specifically used for expansive soils in other countries is provided by Chen (1988) and Nelson and Miller (1992). However, the depth and frequency of sampling may need to be increased in expansive areas due to their high spatial variability.

33.5.1.2 *In situ* testing

A suite of different field tests can be used to evaluate expansive soils and these include:

- soil suction measurements using thermocouple psychrometers, tensiometers or filter paper methods;
- *in situ* density and moisture tests;
- settlement and heave monitoring;
- piezometers or observations wells;
- penetration resistance;
- pressuremeters and dilatometers;
- geophysical methods.

Expansive soils can be tested in the field using methods that rely on empirical correlation such as the standard penetration test (SPT) or the cone penetration test (CPT) to infer soil strength parameters (Clayton *et al*., 1995). Initial effective stresses can be estimated using a psychrometer (Fredlund and Rahardjo, 1993) or a suction probe (Gourley *et al*., 1994) which will measure the soil suction. The undrained shear strength of the soil can be determined using a shear vane (Bjerrum, 1967). The stiffness parameters of the soil can be determined using a plate loading test (BSI, 1999), along with its strength and compressibility. Other tests include the pressuremeter and the dilatometer (ASTM, 2010) which measure strength, stiffness and compressibility parameters.

Seismic test apparatus uses the transmission of elastic waves through the ground in order to determine its density and elastic properties (see Chapter 45 *Geophysical exploration and remote sensing*). Electrical resistivity methods have also shown promise as a method to determine swell pressure and shrinkage of expansive soils. Resistivity was found to increase as both swell pressure and shrinkage increased (Zha *et al*., 2006). More recently, Jones *et al*. (2009) successfully monitored tree-induced subsidence in London Clay using electrical resistivity imaging.

Monitoring should also be considered and a number of approaches can be used which are common with non-expansive soils. Key methods are: settlement and heave monitoring for volume change, and piezometers for pore water changes. Monitoring of water content profiles over several wet and dry seasons are used to establish the extent of the active zone (Nelson *et al*., 2001). In cases where the soil is not uniform or several strata exist, a correction can be applied using the liquidity index. Nelson and Miller (1992) provide an example of this calculation.

Examples of monitoring associated with expansive soils are provided throughout literature. Examples include Fityus *et al*. (2004), where a site near Newcastle, Australia, was instrumented, and soil water and suction profiles together with ground movements were determined over a period (1993–2000). In addition, the work of the BRE at their London Clay site near Chattenden, Kent, provides details of similar monitoring regimes over a number of years (Crilly and Driscoll, 2000; Driscoll and Chown, 2001). Stable benchmarks are important for any monitoring in expansive soils, and design details and installation instructions are given in many papers, e.g. Chao *et al*. (2006).

Further details can be found in Sections 4 *Site investigation* and 9 *Construction verification* of this manual. For specific discussions in the context of expansive soils, see Chen (1988), and Nelson and Miller (1992).

33.5.1.3 Laboratory testing

Considerable research work has been carried out on behalf of the oil and mining industries, especially in the US, on the swelling behaviour of ‘compact’ clays and mudrocks, in particular clay shales. Swelling pressure has caused damage in tunnels (Madsen, 1979), as is the case – usually at great depths – in the mining industry. In the oil industry, the swelling of shales and ‘compact’ clays in borehole and well linings has been a topic of interest. Laboratory test methods developed

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differ considerably from those applied by the civil engineering industry, and tend to duplicate the particular phenomena causing problems. For example, the moisture activity index test (Huang *et al*., 1986) duplicates changes in relative humidity in the air passing through mine tunnels, and consequent swelling of the tunnel lining. However, the confined swelling pressure test is relatively universal. As shrinkage is a near-surface phenomenon in the UK, much work has been done by the soil survey and agricultural organisations. Reeve *et al*. (1980) describe the determination of shrinkage potential for a variety of soils classified on a pedological basis.

For geotechnical purposes, a suite of different tests can be used to identify expansive soils and include Atterberg limits, shrinkage limits, mineralogical tests such as X-ray diffraction, swell tests and suction measurements (see Nelson and Miller, 1992 for further details). Undisturbed samples are normally used for one-dimensional response to wetting tests. However, it should be noted that when conducting swell tests in the laboratory, it is important to distinguish between swelling in compacted, undisturbed and reconstituted samples, which occurs due to significant differences in their respective fabrics.

Swell–shrink tests

Swelling tests may be broadly divided into those tests attempting to measure the deformation or strain resulting from swelling, and those which attempt to measure the stress, or pressure, required to prevent deformation due to swelling. These two types are referred to here as swelling strain and swelling pressure tests, respectively. Swelling strain tests may be linear, i.e. one-dimensional (1D) or volumetric, i.e. three-dimensional (3D). Swelling pressure tests are almost always one-dimensional and traditionally used oedometer-type testing arrangements (Fityus *et al*., 2005). However, shrinkage tests deal solely with the measurement of shrinkage strain in either 1D or 3D.

Standards do exist for shrink–swell tests but these do not cover all the methods in use internationally. Like many 'index'-type soils tests, some shrink–swell tests are based on practical needs and tend to be rather crude and unreliable. Whilst measurement of water content is easily achieved with some accuracy, the measurement of the volume change of a clay soil specimen is not, particularly in the case of shrinkage. Solutions to this problem have been found by the measurement of volume change in only one dimension, or by immersion of the specimen in a non-penetrating liquid such as mercury. However, the use of mercury in this way is far from ideal. Measurement of volume change in the case of swelling, where the specimen is assumed to be saturated, is only slightly less problematic. In this case, dimensional changes are required to be made whilst the specimen is immersed in water. This introduces the problem of either immersed displacement transducers or sealed joints for non-immersed transducers.

Nelson and Miller (1992) provide a detailed account of various swell and heave tests (with the oedometer being the most commonly used) which are often developed based on geographic regions with specific expansive soil problems. However, they can be considered applicable in general situations (Fityus *et al*., 2005). These tests determine the applied stress required to prevent swelling strain when a specimen is subjected to flooding. The ability to do this is enhanced by computer control, or by at least some form of feedback control. The determination of swelling pressure should not be confused with the determination of rebound strain under consolidation stresses in the oedometer test. In the latter case, the slope of the rebound part of the familiar voids ratio versus applied stress (e–log p') curve is referred to as the swelling index (C_s); that is the rebound or decompressional equivalent of the compression index (C_c). It is common, however, for measured swell potential to be low to medium when soil units across a region have high potential; this is the result of natural soil variability (Houston *et al*., 2011).

Mineralogical testing

In addition to the traditional approaches used, several parameters have been investigated which are either wholly or largely dependent on clay mineralogy. These are surface area (Farrar and Coleman, 1967), dielectric dispersion (Basu and Arulanandan, 1974), and disjoining pressure (Derjaguin and Churaev, 1987). The factors affecting swelling of very compact or heavily overconsolidated clays and clay shales may differ from those affecting normally consolidated or weathered clays. Physicochemical and diagenetic bonding forces probably dominate in these materials, whereas capillary forces are negligible. It is likely that the distance between clay platelets, the ionic concentration of pore fluids, and fluids used in laboratory tests relative to the clay mineral activity of such materials, are the key factors in swelling. Traditional concepts of Darcian permeability and pore water pressure are thrown into doubt in these compact clays and clay shales. Diffusion may be the principal mode of fluid movement in these very low permeability clays.

Use index tests

The volume change potential (VCP) (also known as the potential volume change, PVC) of a soil is the relative change in volume to be expected with changes in soil water content, and is reflected by shrinking and swelling of the ground; in other words, the extent to which the soil shrinks as it dries out, or swells when it gets wet. However, despite the various test methods available for determining these two phenomena, e.g. BS 1377, 1990: Part 2, Tests 6.3 and 6.4 *Shrinkage Limit* and Test 6.5 *Linear Shrinkage* and Part 5, Test 4 *Swelling Pressure* (BSI, 1990), they are rarely employed in the course of routine site investigations in the UK. Hence few data are available for databasing the directly measured shrink–swell properties of the major clay formations. Consequently, reliance is placed on estimates based on index parameters, namely, liquid limit, plastic limit, plasticity index, and density (Reeve *et al.*, 1980; Holtz and Kovacs, 1981; and Oloo *et al.*, 1987). No consideration has been given to the saturation state of the soil and therefore to the effective stress or pore water pressures within it.

The most widely used parameter for determining the shrinkage and swelling potential of a soil is the plasticity index (I_p). Such plasticity parameters, being based on remoulded specimens, cannot precisely predict the shrink–swell behaviour of an *in situ* soil. However, they do follow properly laid down procedures, being performed under reproducible conditions to internationally recognised standards (Jones, 1999). A 'modified plasticity index' (I'_p) is proposed in the Building Research Establishment Digest 240 (BRE, 1993a) for use where the particle size data, specifically the fraction passing through a 425 μm sieve, is known or can be assumed as 100% passing (**Table 33.1**).

The modified I'_p takes into account the whole sample and not just the fines fraction; it therefore gives a better indication of the 'real' plasticity value of an engineering soil and eliminates discrepancies due to particle size, for example in glacial till. This compares with a classification produced by the National House-Building Council which forms the basis of the NHBC 'foundation depth' tables (**Table 33.2**), which uses the same modified I'_p approach as presented in **Table 33.1**.

The concept of 'effective plasticity index' has been described (BRAB, 1968) to deal with multi-layered soils of different plasticity index.

Ultimately, swelling and shrinkage *potential* may be considered to be the ultimate capability of a soil to swell and shrink, but this potential is not necessarily realised in a given moisture change situation. These do not therefore represent the fundamental properties of a soil. However, potential may be described differently. For example, swelling potential is described by Basu and Arulanandan (1974) as 'the ability and degree to which swelling is realised under given conditions'. So there is already some confusion in terminology. Oloo *et al*. (1987) differentiate between intrinsic expansiveness (swell) and heave. They define intrinsic expansiveness as that property which 'relates change in water content, and thus change in volume, to the suction change' of a clay soil. Thus a soil of high intrinsic expansiveness will exhibit a large water content or volume change compared with one of low intrinsic expansiveness for a given suction change – all other things being equal. Oloo *et al*. (1987) state that no procedure has been developed to measure this property. Swell is defined as 'a measure of the volume strain, or axial strain, in a soil under a particular set of stress and suction conditions'. Heave is defined as 'the displacement of a point in the soil due to suction and stress changes interacting with the intrinsic expansiveness'. Heave is not a soil property.

Overall, there are many methods of testing for the shrinkage and swelling properties of clay soils. These methods are covered in detail in Jones (1999), where the pros and cons of each method are discussed and the reasons for the selection and rejection of methods is determined. Further evaluation of these tests is also provided by Fityus *et al*. (2005).

I'_p (%)	Volume change potential
> 60	Very high
40–60	High
20–40	Medium
< 20	Low

Note: $I'_p = I_p \times (\%<425\,\mu m) / 100\%$

Table 33.1 Classification for shrink–swell clay soils
Data taken from BRE (1993a)

I'_p (%)	Volume change potential
> 40	High
20–40	Medium
10–20	Low

Table 33.2 Classification for shrink–swell clay soils
Data taken from NHBC (2011a)

33.5.2 Shrink/swell predictions

Common to all geotechnical predictions of volume change is the need to define initial and final *in situ* stress state conditions. In addition this requires characterisation of the stress–strain behaviour of each soil profile. Initial stress states and constitutive properties can be evaluated using a suite of approaches (highlighted by many texts, e.g. Fredlund and Rahardjo,1993; Powrie, 2004) but it is the final stress condition that must usually be assumed. Guidelines are presented by Nelson and Miller (1992), with calculations based on knowledge of effective overburden stress (i.e. the increment of stress due to applied load and soil suction). However, each situation requires engineering judgement and consideration of environmental conditions.

Details of constitutive relationships for expansive soils have been reviewed and a useful description of these is given by Nelson and Miller (1992). These include unsaturated soil models dealing with matric and osmotic suctions. A detailed account of this, the theoretical basic, associated models used to predict partially saturated soils behaviour, together with test methods used to determine key soil parameters, is provided in Fredlund and Rahardjo (1993) and Fredlund (2006).

Overall prediction methods can be grouped into three broad categories: theoretical, semiempirical and empirical. They all rely on testing methods; particular care must be taken with empirical methods which are only valid within the bounds of the soil type, environment and engineering application for which they were developed.

A number of heave predictions are available that are based on oedometer or suction tests, and Nelson and Miller (1992) provide a detailed account of these, together with examples of associated predictions. For example Nelson *et al*. (2010) provide an illustration using free-field heave predictions and their use in foundation design, as well as methods for prediction heave rates.

33.5.2.1 Oedometer-based methods

Oedometer-based tests include one-dimensional and double oedometer tests (developed by Jennings and Knight, 1957).

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Double oedometer tests consist of two near-identical undisturbed samples, one loaded at its natural water content and the other inundated under a small load and then loaded under saturated conditions. The use of the oedometer has distinct advantages due to familiarity amongst geotechnical engineers.

Tests can be conducted as free swell tests where swelling is allowed to occur at a pre-determined pressure after water is added. The swell pressure is then defined as the pressure required to recompress the swollen sample to its pre-swollen volume. These tests, however, suffer the limitation that volume change can occur and that hysteresis is incorporated into the estimation of the *in situ* state. An alternative approach that overcomes these problems involves inundating a sample placed in the oedometer and preventing it from swelling. The swell pressure is then the maximum applied stress required to achieve a constant volume. Typical results from these tests are shown in **Figure 33.6**, with σ'_0 representing the stress when inundation occurred and σ'_s representing the stress equated to swelling pressure.

The constant volume test may overcome the difficulties of the free swell test, but as a result is more vulnerable to sample disturbance. To account for sample disturbance, Rao *et al*. (1988) and Fredlund and Rahardjo (1993) suggest simplifications to facilitate predictions using parameters measured by constant volume oedometer tests (pressures increase during swelling to maintain constant volume) using established techniques. This is illustrated in **Figure 33.7**.

Fityus *et al*. (2005) questioned this approach and considered that specialist apparatus not normally used in standard geotechnical engineering testing laboratories is needed to achieve meaningful results. However, not all authors agree, with Nelson and Miller (1992) believing good quality data and predictions can be obtained with such an approach. Moreover, a number of disadvantages exist, as tests where the specimen is fully wetted are conservative, as full saturation is not often reached in the field (Houston *et al*., 2011). Thus, swell tests based on submerged samples at the level of stress of interest will overpredict heave. The effect of partial wetting may be as important as the depth to which wetting has occurred (Fredlund *et al*., 2006).

33.5.2.2 Suction-based tests

Suction tests are used to predict soil response in much the same manner as with saturated effective stress changes. Various methods have been developed, e.g. the US Army Corps of Engineers Waterways Experiment Station (WES) method or the clod method, details of which (including advantages and limitations) can be found in Nelson and Miller (1992). Fredlund and Hung (2001) have subsequently developed suction-based predictions to evaluate volume changes from both environmental and vegetation changes – and they provide useful outline example calculations.

Nelson and Miller (1992) suggest that with careful sampling and testing it is possible to predict heave within a few centimetres. However, it is essential that the testing is conducted within the expected stress range in the field. Furthermore, experimental studies involving direct measurement of partially saturated properties is expensive and often time-consuming. For example, Chandler *et al*. (1992) provide details of suction measurements using the filter paper method, highlighting the need for careful calibration as results can be affected by temperature fluctuations, particle entrainment in the filter paper during testing, and hysteresis effects. Such approaches have a number of advantages as a means to estimate soil suction and hence suction profiles (see **Figure 33.4**).

For this reason, increasingly numerical and semiempirical methods use the soil–water characteristic curves (SWCCs) (Puppala *et al*., 2006). The SWCCs describe the relationship

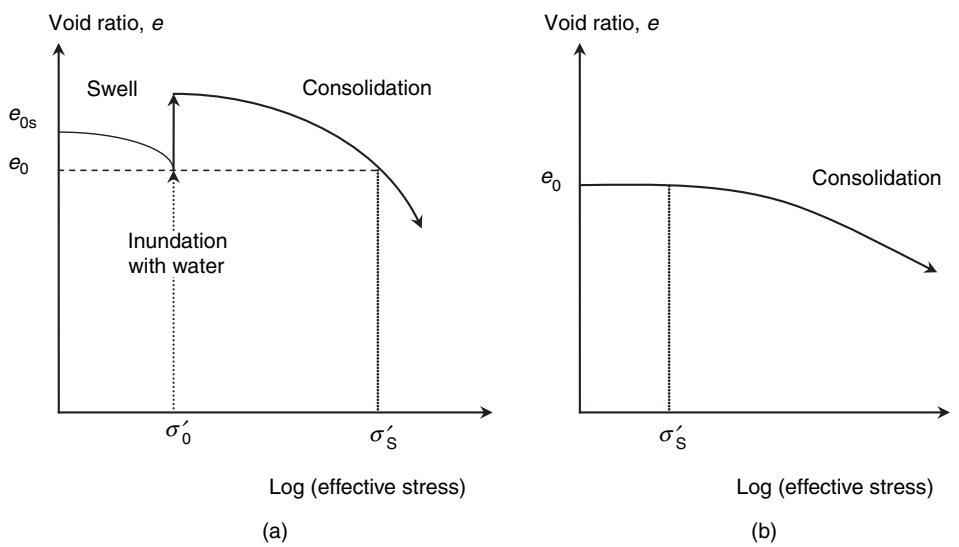


Figure 33.6 Typical oedometer swell test curves: (a) an illustration of a free swell test result; (b) an illustration of constant volume test results

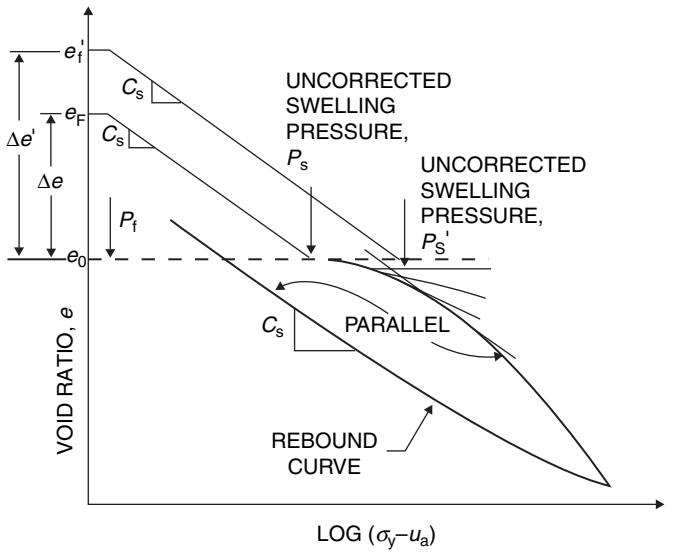


Figure 33.7 One-dimensional oedometer test results showing effect of sampling disturbance. Note: C_s is swell index; ($\sigma_y - u_a$) is overburden pressure; P_f is final stress state; e_f is final void ratio, and e_f' is final void ratio corresponding to corrected swell pressure, P_s'
Reproduced from Rao *et al.* (1988), with kind permission from ASCE

between water content (either gravimetric or volumetric) and soil suction. Alternatively, they can be used to describe the relationship between the degree of saturation and soil suction. A more detailed discussion and examples of typical SWCCs are also provided in Chapter 30 *Tropical soils*.

Only a limited number of investigations have been undertaken on expansive soils with Ng *et al*. (2000), Likos *et al*. (2003) and Miao *et al*. (2006) providing some example of these. Puppala *et al*. (2006) details SWCCs for both treated and untreated expansive soils. Further details of this are provided by Fredlund and Rahardjo (1993) with Nelson and Miller (1992) providing details in the context of expansive soils. However, it should be noted that suction measurements are subject to errors that can be substantial (Walsh *et al*. 2009).

Empirically-based methods are still common in geotechnical engineering (Houston *et al*., 2011). Heave is often estimated by the integration of strain over the zone in which the water contents change. However, uncertainty occurs and arises from three sources (Walsh *et al*., 2009):

1. the depth over which the wetting will occur;
2. the swell properties of the soil;
3. the initial and final suction over the depth of wetting.

Furthermore, care is needed with all models used, as small changes in input parameters can lead to significant changes in an estimated soil response. The real challenge is, therefore, to understand the relationship between soil–water stress level and volume changes, coupled with a prediction of the actual depth and degree of wetting that will occur in the field. Both are related to soil properties and control of site water (Houston *et al*., 2011).

Houston *et al*. (2011) compared predictions from a number of forensic studies from field and laboratory investigations in arid/semi-arid areas to those undertaken using numerical approaches (in this case, the simple 1D and 2D unsaturated flow model), with details of site drainage and landscape practices also considered. Comparisons were made after one year; they concluded that drainage conditions were the more important factor in the prediction of foundation problems. This study revealed that the effects of poor drainage and roof run-off ponding near a structure is the worst case scenario. Uncontrolled drainage and water ponding near foundations led to significant suction reduction to greater depths (0.8 m was found after one year), resulting in differential soil swell and foundation movement (see **Figure 33.8**).

33.5.2.3 Numerical approaches

1D simulations also dominate numerical studies, as unsaturated flow solutions are sensitive to accurate and detailed simulation of surface flux conditions, thus requiring an extremely tight mesh and time steps (Houston *et al*., 2011). This may result in very lengthy run times of several months, even for 1D assessments (Dye *et al*., 2011). However, Xiao *et al*. (2011) demonstrated how numerical simulations could be used to assess pile–soil interactions, providing an effective way to undertake sensitivity analysis, but noted that many parameters are needed when undertaking numerical assessments.

33.5.3 Characterisation

Many attempts have been made to find a universally applicable system for the classification of shrinking and swelling in order to characterise an expansive soil. Some have even attempted to produce a unified swelling potential index using commonly used indices (e.g. Sridharan and Prakash, 2000; Kariuki and van der Meer, 2004; Yilmaz, 2006) or from specific surface areas (Yukselen-Aksoy and Kaya, 2010), but these are yet to be adopted. Examples of various schemes commonly used around the world are illustrated in **Figure 33.9**. The various schemes that have been developed lack standard definitions of swell potential, since both sample conditions and testing factors vary over a wide range of values (Nelson and Miller, 1992).

33.5.3.1 Classification schemes

Most classification schemes give a qualitative expansion rating, e.g. high or critical. The different classification schemes can be categorised into four groups, depending on which method they employ to determine their results. These are:

1. free swell (see Holtz and Gibbs, 1956);
2. heave potential (see Vijayvergiya and Sullivan, 1974; Snethen *et al*., 1977);
3. degree of expansiveness (see US Federal Housing Administration (FHA), 1965; Chen, 1988);
4. shrinkage potential (see Altmeyer, 1956; Holtz and Kovacs, 1981).

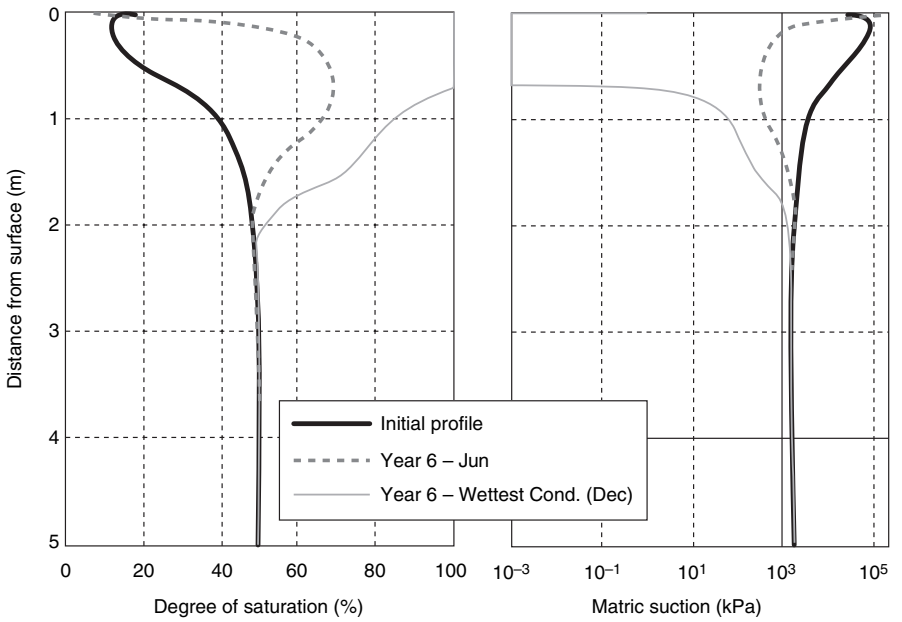


Figure 33.8 Profile for 1 year of roof run-off water ponding next to foundation after 6 years of desert landscape. Wettest and driest conditions in 1-D
Reproduced from Houston *et al.* (2011), with kind permission from ASCE

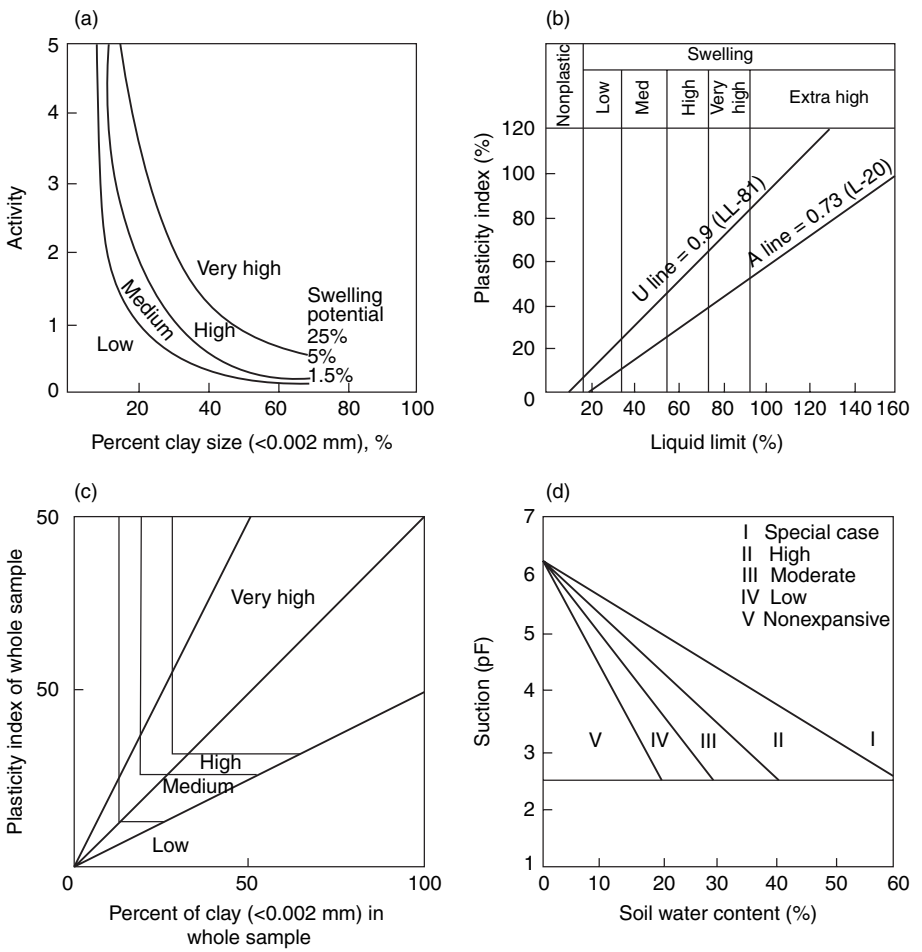


Figure 33.9 Commonly used criteria for determining swell potential from across the world
Reproduced from Yilmaz (2006), with permission from Elsevier

Since liquid limit and swelling of clays both depend on the amount of water a clay tries to imbibe, it is not surprising that they are related. Chen (1988) suggested that a relationship between the swelling potential of clay and its plasticity index can be established. While it may be true that high swelling soil will manifest high index properties, the converse is not always true.

Other classification schemes relate to expansion potential, based on the Skempton 'activity' plot (Skempton, 1953) and its development by Williams and Donaldson (1980) from Van der Merwe (1964). Details are described in Taylor and Smith (1986) with respect to various UK clay mudstone formations.

A host of schemes have been put forward for estimating shrink–swell, particularly in the US (see Chen, 1988; Nelson and Miller, 1992), most of which use swelling and suction as their basis (Snethen, 1984). Sarman *et al*. (1994) concluded that swelling was not related solely to clay mineral type, but also to pore-morphology. It was found that samples showing high swelling had a large pore volume combined with a high percentage of small-sized pores. The high swelling was attributed to the samples' ability to absorb and adsorb water. It was found that correlations between swelling and other parameters were unsuccessful.

With all classification schemes only indications of expansion are obtained with, in reality, field conditions varying considerably. Such ratings can be of little use unless the user is familiar with the soil type and the test conditions used to develop the ratings. Ratings themselves may be misleading and can, if used with design options outside the region where the rating was established, cause significant difficulties (Nelson and Miller, 1992). Classifications, therefore, should only be considered to provide an indication of potential expansive problems, and further testing is needed. If such schemes are used as a basis of design, the result is either over-conservative solutions or inadequate construction (Nelson and Miller, 1992).

33.5.3.2 UK approach

Whilst much study has been carried out worldwide to infer swelling and shrinkage behaviour from soil index properties such as plasticity (see section 33.5.1.1.3), few direct data are available in UK geotechnical databases (Hobbs *et al*., 1998). Two schemes that are commonly used within the UK are based on the BRE and NHBC schemes.

Volume change potential has more recently been defined for overconsolidated clays in terms of a modified plasticity index term (I'_p) by Building Research Establishment Digest 240 (BRE, 1993a) – see **Table 33.1**. This classification aims to eliminate discrepancies due to particle size.

High shrinkage potential soils may not behave very differently from low shrinkage ones, because environmental conditions in the UK do not allow full potential to be realised (Reeve *et al*., 1980). The National House-Building Council (NHBC, 2011a) classified volume change potential as shown in **Table 33.2**. This classification forms the basis of the NHBC's 'foundation depth' tables.

Since a set of soil properties will often not fit neatly into one category, the determination of shrinkage potential requires some judgement. The BRE (1993a) suggests that plasticity index and clay fraction can be used to indicate the potential of a soil to shrink, or swell, as follows:

Plasticity index (%)	Clay fraction (<0.002 mm)	Shrinkage potential
>35	>95	Very high
22–48	60–95	High
12–32	30–60	Medium
<18	<30	Low

The overlap of categories reflects the fact that figures were obtained from multiple sources.

33.5.3.3 National versus regional characteristics

A meaningful assessment of the shrink–swell potential of soil in the UK requires a considerable amount of high quality and well-distributed spatial data of a consistent standard. The British Geological Survey's *National Geotechnical Properties Database* (Self *et al*., 2008) contains a large body of index test data. At the time of writing, the database contained data from more than 80 000 boreholes, comprising nearly 320 000 geotechnical samples, with 100 000 containing relevant plasticity data.

The British Geological Survey (BGS) GeoSure *National Ground Stability Data* provides geological information about potential ground movement or subsidence, including the GeoSure shrink–swell dataset (Booth *et al*., 2011). It should be noted that this assessment does not quantify the shrink–swell behaviour of a soil at a particular site; it indicates the potential for such a hazard to be present with regard to the behaviour of the underlying geological unit throughout its outcrop.

The VCP of a soil provides the relative change in volume to be expected with changes in soil water content. This was calculated from the I'_p values and a classification made based on the *upper quartile* value (**Table 33.3**). This is based on the BRE (1993a) scheme shown in **Table 33.1**. In this way, a VCP was assigned to each of the geological units and a map of shrink–swell potential built (**Figure 33.10**).

Looking at clays on a national scale can give a good indication of the potential problems associated with them and provide

Classification	I'_p (%)	VCP
A	< 1	Non-plastic
B	1–20	Low
C	20–40	Medium
D	40–60	High
E	> 60	Very high

Table 33.3 Classification of VCP

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initial information regarding planning decisions. However, no two clay soils are the same in terms of their behaviour or their shrink–swell potential. Therefore, it is useful to look at a particular clay formation on a more regional basis. For illustration, the London Clay formation will be used.

The London Clay formation is of major importance in the fields of geotechnical engineering and engineering geology. This is because it has hosted a large proportion of sub-surface engineering works in London over the last 150 years. It has also been the subject of internationally recognised research in soil mechanics

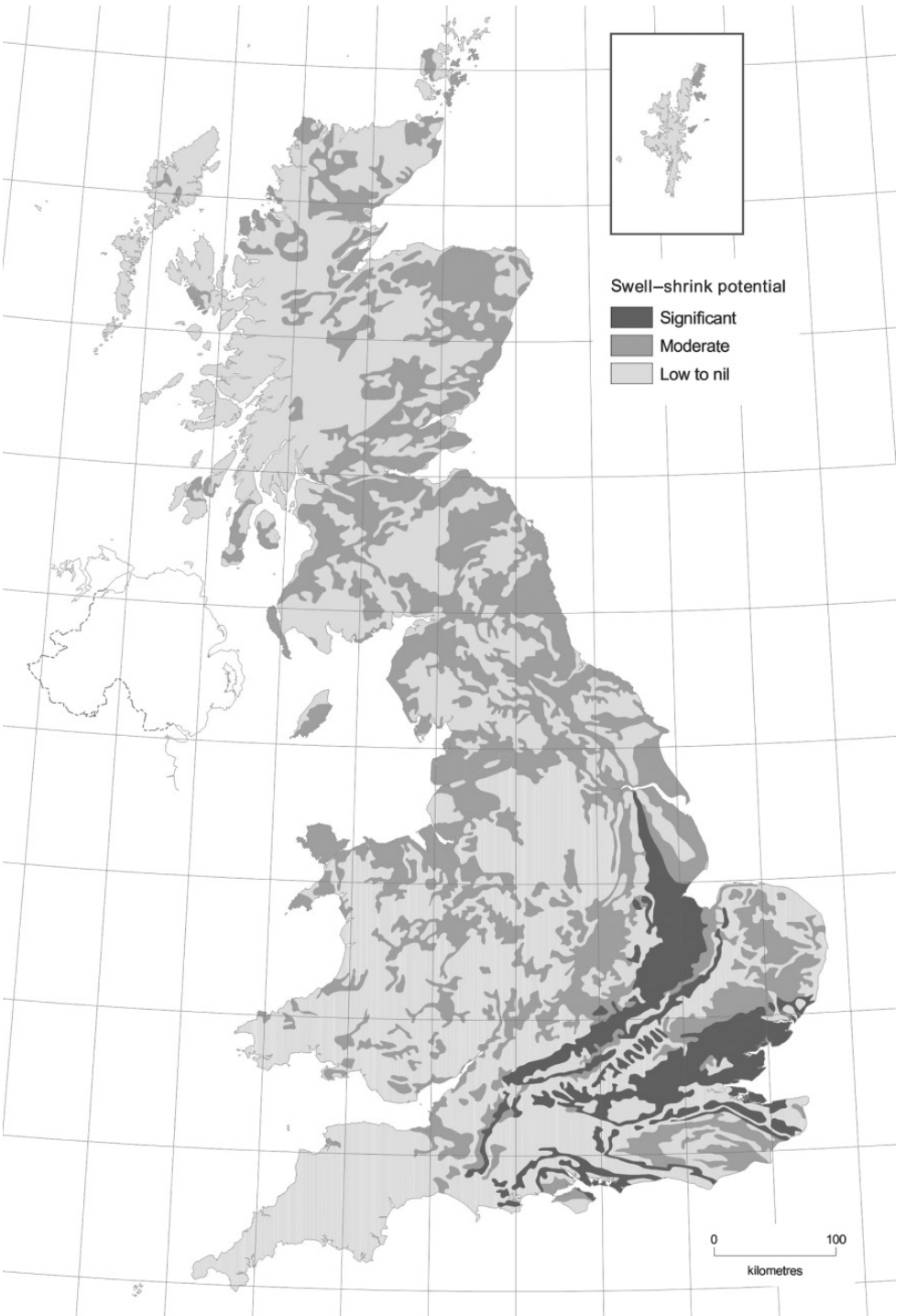


Figure 33.10 Shrink–swell potential map, based on VCP
Reproduced from Jackson (2004) © NERC, with permission from the British Geological Survey

over the last 50 years (Skempton and DeLory, 1957; Chandler and Apted, 1988 and Takahashi *et al*., 2005). The London Clay is subject to shrinkage and swelling behaviour, which has resulted in a long history of foundation damage within the outcrop.

Jones and Terrington (2011) follow the methodology described in Diaz Doce *et al*. (2011) using 11 366 samples across the London Clay outcrop, splitting it into four distinct areas based on geographical location, plasticity values and depth of overlying sediment. In this way, a more detailed assessment of the outcrop could be carried out, and a 3D model providing a seamless interpolation of the VCP of the London Clay was created. This model gives a visualisation of the I'_p values, allowing them to be examined at a variety of depths relative to ground level (**Figure 33.11**). This type of analysis indicates that 3D modelling methods have considerable potential for predicting the spatial variation of VCP within expansive clay soils, so long as they have large enough data sets.

33.5.4 Specific problems with expansive soils

The principal adverse effects of the swell–shrink process arise when either swelling pressures result in heaving (or lifting) of structures, or shrinkage leads to differential settlement. As a result, a number of mitigation and design options exist either in the form of specific foundation types, or through the use of a range of different ground improvement techniques. Excellent reviews of the full range of these are provided by both Chen (1988) and Nelson and Miller (1992), together with details provided by NHBC (2011a). A summary is provided in the following sections (33.5.4.1–33.5.4.4) highlighting the key features associated with these options. In addition, discussion of some of the key issues faced in the UK is provided (see section 33.5.4.5) where impact of vegetation is often the major cause of soil–structure problems faced by expansive soils.

33.5.4.1 Foundation options in expansive soils

A large number of factors influence foundation types and design methods (see Section 5 *Design of foundations*); these include climatic, financial and legal aspects, as well as technical issues. Importantly, swell–shrink behaviour often does not manifest itself for several months and so design alternatives must take account of this. Other issues, such as financial considerations, can place strain on this and so early communication with all relevant stakeholders is essential. Higher initial costs are often offset many times over by a reduction in post-construction maintenance costs when dealing with expansive soils (Nelson and Miller, 1992).

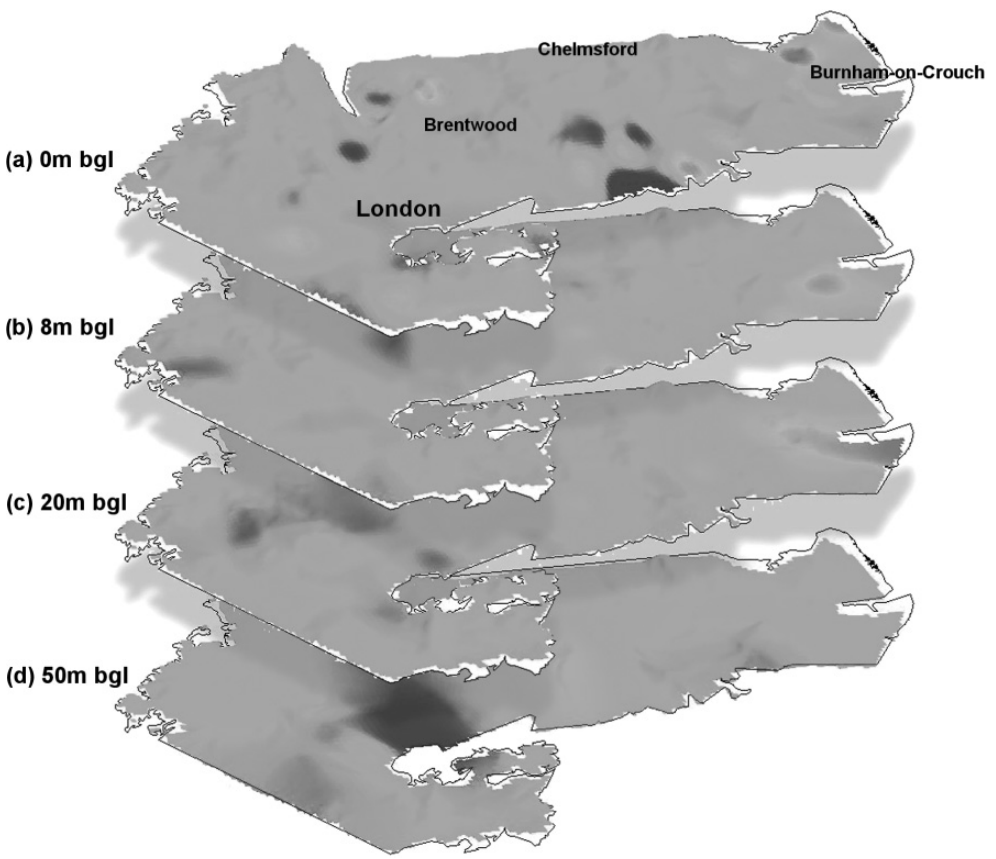


Figure 33.11 S-grid interpolations for area 3, showing surfaces at 0 m, 8 m, 20 m and 50 m bgl. [blue: medium, green: high, yellow/red: very high VCP]
Reproduced from Jones and Terrington (2011) © The Geological Society of London. A colour version of this figure is available online

Foundation alternatives when dealing with potentially expansive soils follow three options:

1. use of structural alternatives, e.g. stiffened raft;
2. use of ground improvement techniques;
3. a combination of (1) and (2).

As with any foundation option, the main aim is to minimise the effects of movement, principally differential. Two strategies are used when dealing with expansive soils:

- isolate structure from soil movements;
- design a foundation stiff enough to resist movements.

The major types of foundations used in expansive soils from around the world are pier and beam or pile and beam systems, reinforced rafts and modified continuous perimeter spread footings. These are summarised in **Table 33.4**; further details are provided by Chen (1988), Nelson and Miller (1992) and NHBC (2011a, 2011b, 2011c), and are discussed further below. It should be noted that terminology used to describe the foundation types listed in this table vary across the world with, for example, slab-on-grade used in the US for raft foundations.

Pier and beam; pile and beam foundations

These foundations consist of a ground beam to support structural loads, transferring the load to the piers or piles. A void is provided between the pier/pile and the ground beam to isolate the structure and prevent uplift from swelling. NHBC (2011a) provides guidance on minimum void dimensions. Floors are then constructed as floating slabs. The piers/piles are reinforced (with reinforcement taken over the whole length to avoid tensile failures) using concrete shafts with or without bell bottoms, steel piles (driven or pushed), or helical piles whose aim is to transfer loads to stable strata. Under-reamed bottoms and helical piers/piles can be effective in soils with a high swell potential, overcoming the impractical length that would otherwise be required with straight shaft piers/piles, or where there is a possibility of a loss of skin friction due to rising groundwater levels. If a stable non-expansive stratum occurs near the surface, the piers/piles can be designed as rigid anchoring members. If, however, the depth of potential swell is high, the piers/piles should be designed as elastic members in an elastic medium. **Figure 33.12** illustrates a typical pier and beam foundation from US practice. Very similar arrangements are used in the UK and are illustrated in NHBC (2011a, Figures 10 and 11, therein).

Design and construction procedures for each of these systems are provided in detail (including sample design calculations) by Chen (1988) and Nelson and Miller (1992). Additional discussion and example design calculations are provided by Nelson *et al*. (2007). It is important to ensure sufficient anchorage below the active zone. Pier/pile diameters are kept small (typically 300–450 mm). Any smaller, and problems will result in poor concrete placement and associated defects, e.g. void spaces. Another problem that can occur is 'mushrooming' near the top of the pier/pile, which provides an additional area for uplift forces to act upon. To avoid this, cylindrical cardboard forms are often employed and removed after the beam is cast to prevent a means to transmit swell pressures. The size of this void space depends on the magnitude of potential swell, with 150–300 mm often being used. In the upper active zone, shafts should be treated to reduce skin friction and hence minimise uplift forces. It is important that any chosen approach does not provide potential pathways to allow water to ingress to deeper layers, as this will cause deep-seated swelling.

Stiffened rafts

Stiffened slabs are either reinforced or post-tensioned systems, the latter being common in countries like the US. Design procedures consist of determining bending moments, shear, and deflections, associated with structural and swell pressure loads. The general layout used is illustrated in **Figure 33.13**, which shows examples used commonly in the US. Similar approaches are used in the UK and are presented in NHBC (2011a; 2011b).

Designs are modelled on the soil–structure interaction at the base of the slab, by considering the slab as a loaded plate or beam resting on an elastic medium. Essentially, two extremes exist – the first where a ground profile develops assuming a

Foundation type	Design philosophy	Advantages	Disadvantages
Pier and beam; pile and beam	Isolate structure from expansive movement by counteracting swell with anchoring to stable strata	Can be used in a wide variety of soils; reliable for soils of high swell potential	Relatively complex design and construction processes requiring specialist contractors
Raft; stiffened raft	Provides a rigid foundation to protect structure from differential settlements	Reliable for soils of moderate swell potential; no specialist equipment needed in construction	Only works for relatively simple building layout; requires full construction quality control
Modified continuous perimeter footing; deep trench fill foundations	Same as raft or stiffened raft foundation – includes stiffened perimeter beams	Simple construction with no specialist equipment needed	Ineffective in highly expansive soils or within the zone of influence of trees

Table 33.4 Foundation types used in expansive soils
Data taken from Nelson and Miller (1992); NHBC (2011a)

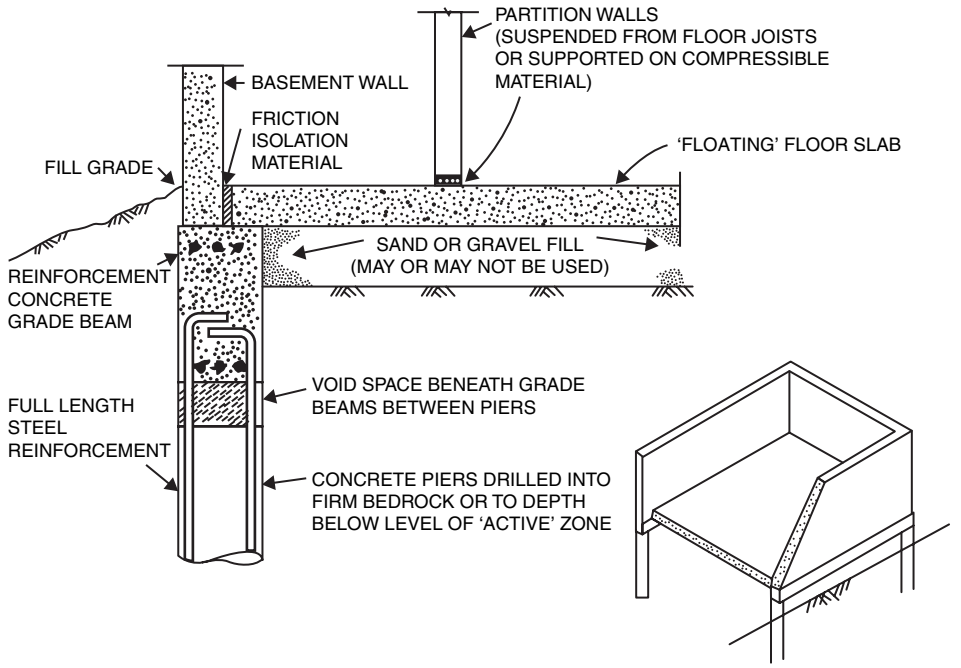


Figure 33.12 Illustration of a pier and beam foundations
Reproduced from Nelson and Miller (1992); John Wiley & Sons, Inc

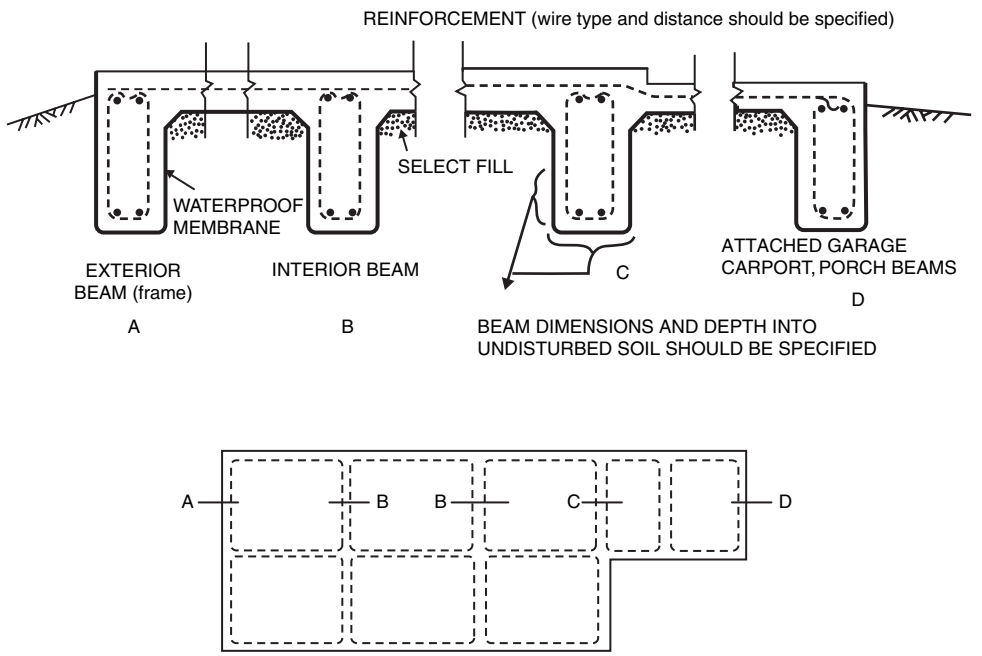


Figure 33.13 Typical detail of a stiffened raft
Reproduced from Nelson and Miller (1992); John Wiley & Sons, Inc

weightless slab, and the second where a slab of infinite stiffness is placed on the swelling soil. In reality, slabs exhibit some flexibility and so the actual heave produced by swelling soils lies somewhere between these two extremes. These modes of movement are illustrated in **Figure 33.14**.

Several design approaches have been developed, each using a range of different combinations of soil and structural design parameters. A detailed account of these is provided by Nelson and Miller (1992) with additional discussion provided by Houston *et al*. (2011).

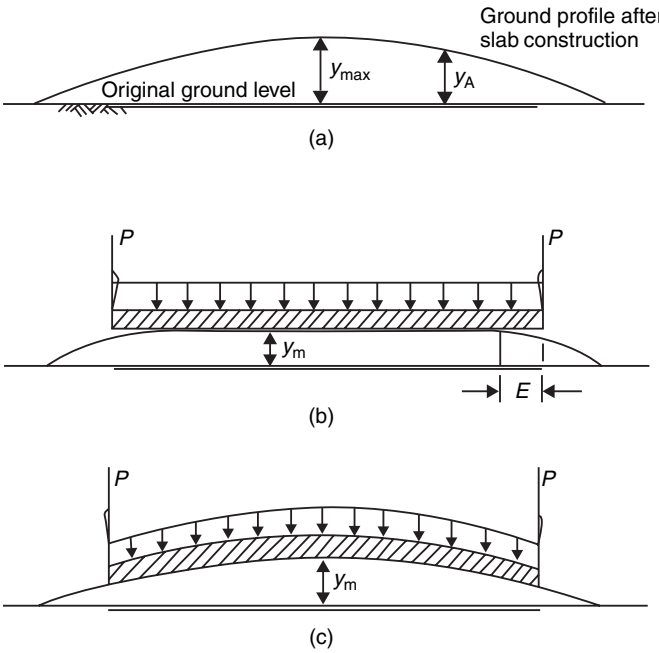


Figure 33.14 Profiles after construction for various stiffness of raft: (a) with no load applied; (b) with infinitely stiff slab; (c) with flexible slab. Notes: y_{max} = maximum heave, no foundation present – the free field heave; y_m = maximum differential heave; E = distance from outer edge to point where swelling soil contacts foundation; P = loading; y_A = height of free field heave along ground profile
Reproduced from Nelson and Miller (1992); John Wiley & Sons, Inc

The primarily geotechnical information required includes size, shape and properties of the distorted soil surface that develop below the slab. These depend on a number of factors including heave, soil stiffness, initial water content, water distribution, climate, post-construction time, loading, and slab rigidity. It should be noted that the slab, through its elimination of evapotranspiration (see **Figure 33.5**), promotes the greatest increase in water content near to the centre of the slab – and hence to where long-term distortion is most severe. However, the maximum differential heave (y_m in **Figure 33.14**) has been found to vary between 33 and 100% of total maximum heave (Nelson and Miller, 1992). On occasion, edge heave can occur when the exterior of a structure experiences increases in water content before the interior.

Modified continuous perimeter footing

Shallow footing should be avoided where expansive soils are found. However, where they are used, a number of approaches can be employed to minimise the effects of swelling/shrinkage. Modifications include:

- narrowing footing width;
- providing void spaces within support beam/wall to concentrate loads at isolated points;
- increasing perimeter reinforcement – taking this into the floor slab stiffening foundations.

The use of narrow spread footing in expansive soils should be restricted to soils exhibiting 1% swell potential and very low swell pressures (Nelson and Miller 1992).

NHBC (2011a) suggested that strip and trench fill foundations can be used when placed in a non-expansive layer that overlies expansive soils, provided that:

- soil is consistent across the site;
- the depth of non-expansive material is greater than ¾ of the equivalent foundation depth, assuming all soil is expansive (guidance provided within NHBC, 2011a);
- the thickness of the non-expansive soil below the foundation is at least equal to the foundation width.

Case studies

Chen (1988) provides a series of case study examples of foundations and problems that arise when dealing with expansive soils, including distress caused by the following: pier/pile uplift, improper pier/pile design and construction, heaving of a pad and floor slab, heaving of a continuous floor, and a rising water table. Further reviews of issues related to other foundation types, for example the use of post-tensioned stiffened raft foundations, are discussed by Houston *et al.* (2011). Other useful case studies are provided by Simmons (1991) and Kropp (2011). It is clear that a number of foundation failures occur and these can be summarised as follows:

1. *Changes in water content*
   - chiefly high water tables;
   - poor drainage under foundations;
   - leaks due to sewer failure or poorly managed runoff;
   - irrigation and garden watering.
2. *Poor construction practice*
   - insufficient edge beam stiffness;
   - inadequate slab thickness;
   - inadequate anchorage from piers;
   - pier length inadequate or ‘mushrooming’ of piers/piles resulting in uplift as swelling occurs;
   - lack of reinforcement making structure intolerant to movements;
   - inadequate void space.
3. *Lack of appreciation of soil profile*
   - underlying geology contains inclined bedding of bedrock, causing swell to be both vertical and horizontal;
   - uncontrolled fill placement;
   - areas of extensive depth of expansive soil, so drilled pier and beam foundation may not be practical and a more flexible system should be used.

When assessing failure from swell–shrink behaviour it is important to isolate structural defects from foundation movement, as both can cause cracking distress in buildings (Chen, 1988). Useful reviews of geotechnical practice in relation to expansive soils have been provided by Lawson (2006) for Texas, Kropp (2011) for the San Francisco Bay Area, and Houston *et al*. (2011) for Arizona. Although these are US-based, there are many lessons that geotechnical engineers can learn from these studies. Ewing (2011) provides an interesting case from Jackson, Mississippi, USA, of a series of repairs over a 30-year period to a house (on the US's register of historic places) built on 1.5 m of non-expansive soils overlying expansive clay some 8 m thick.

33.5.4.2 Pavement and expansive soils

Pavements are particularly vulnerable to expansive soil damage, with estimates suggesting that they are associated with approximately half of the overall costs from expansive soils (Chen 1988). Their inherent vulnerability stems from their reasonably lightweight nature, extended over a relatively large area. For example, Cameron (2006) describes problems with railways built on expansive soils where poor drainage exists, and Zheng *et al*. (2009) provide details (from China) of highway sub-grade construction on embankments and in slopes. Damage to pavements on expansive soils comes in four major forms:

- severe unevenness along significant lengths – cracks may or may not be visible (particularly important for airport runways);
- longitudinal cracking;
- lateral cracking, developed from significant localised deformations;
- localised pavement failure associated with disintegration of the surface.

Pavement design is essentially the same as that used for foundations. However a number of different approaches are required as pavements cannot be isolated from the soils and it is impractical to make pavements stiff enough to avoid differential movements. Therefore it is often more economic to treat sub-grade soils (see section 33.5.4.3 below for further details). Pavement designs are based on either flexible or rigid pavement systems; these procedures are discussed in Section 7 *Design of earthworks, slopes and pavements* and Chapter 76 *Issues for pavement design* of this manual. However, when dealing with expansive soils a number of approaches should be considered:

1. choose an alternative route and avoid expansive soil;
2. remove and replace expansive soil with a non-expansive alternative;
3. design for low strength and allow regular maintenance;
4. physically alter expansive soils through disturbance and re-compaction;
5. stabilise through chemical additives, such as lime treatment;
6. control water content changes – although very difficult over the life of a pavement. Techniques include pre-wetting, membranes, deep drains, slurry injection treatment.

Nelson and Miller (1992) provide further details on testing undertaken to mitigate expansive soil behaviour for pavement construction. Cameron (2006) has advocated the use of trees as they can be beneficial in semi-arid environments to manage poorly-drained areas under railways. However, this needs careful management and may require several years to be fully effective.

33.5.4.3 Treatment of expansive soils

Essentially, treatment of expansive soils can be grouped into two categories:

1. soil stabilisation – remove/replace; remould and compact; pre-wet, and chemical/cement stabilisation;
2. water content control methods – horizontal barriers (membranes, asphalt and rigid barriers); vertical barriers; electrochemical soil treatment, and heat treatment.

A detailed account of the various treatment approaches is provided by Chen (1988) and Nelson and Miller (1992), with a detailed review of stabilisation over the last 60 years provided by Petry and Little (2002). As with any treatment approach, it is essential to undertake appropriate site investigations and evaluations (see Section 6 *Design of retaining structures* and section 33.5.1 above). Special consideration should be given to the following: depth of the active zone, potential for volume change, soil chemistry, water variations within the soil, permeability, uniformity of the soils, and project requirements. An overview of each of the two categories of treatments applied to expansive soils is provided below, with **Table 33.5** providing brief details of soil stabilisation approaches.

In a recent survey, Houston *et al*. (2011) found that many geotechnical and structural engineers considered chemical stabilisation approaches, such as the use of lime, as ineffective for pre-treatment of expansive soils for foundations. Preference is typically given for use of either pier/pile and beam foundations, or stiffened raft foundations. This is not true for pavements, where lime and other chemical stabilisation approaches are commonly used worldwide. The various stabilisers can be grouped into three categories (Petry and Little, 2002):

- traditional stabilisers – lime and cement;
- by-product stabilisers – cement/lime kiln dust and fly ash;
- non-traditional stabilisers – e.g. sulfonated oils, potassium compounds, ammonium compounds and polymers.

Further details of these can be found in Petry and Little (2002). However, as with any soil treated with lime, care is needed to assess chemical as well as physical soil properties to prevent swelling from adverse chemical reactions (Petry and Little, 2002). For example, Madhyannapu *et al*. (2010) provide details

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of quality control when stabilising expansive sub-soils using deep soil mixing, demonstrating the use of non-destructive tests based on seismic methods.

Chemical stabilisation can be used to provide a cushion immediately below foundations placed on expansive soils, e.g. for pavements (Ramana and Praveen, 2008). Swell mitigation has also been achieved by mixing non-swelling material e.g. sand (Hudyma and Avar, 2006) or granulated tyre rubber (Patil *et al*., 2011) into expansive soils to dilute swell potential.

In some cases surcharging may be used, but this is only effective with soils of low to moderate swelling pressures. This requires enough surcharge load (see the first row in **Table 33.5**) to counteract expected swell pressures. This method is therefore only used for soil of low swell pressure and with structures that can tolerate heave. Examples include secondary highway systems, or where high foundation pressures occur. Pre-wetting – due to its uncertainties – can only be used with caution, with both Chen (1988) and Nelson and Miller (1992) indicating that it is unlikely to play an important role in the construction of foundations on expansive soils.

Fluctuations in water content are one of the primary causes of swell–shrink problems, with non-uniform heave occurring due to non-uniformity of water content, soil properties, or both. Thus, if water content fluctuations can be minimised over time, then swell–shrink problems can be mitigated. Moreover, if water content changes can be slowed down and water distributions in expansive soils made uniform, then differential movements can also be reduced. In essence, this is the aim of the introduction of moisture/water barriers. These act to:

1. move the edge effects away from the foundation/pavement and so minimise seasonal fluctuation effects;
2. lengthen the time for water content changes to occur – due to longer migration paths under foundations.

Barrier techniques comprise:

- horizontal barriers – using membranes, bituminous membranes or concrete;
- vertical barriers – polyethylene, concrete, impervious semi-hardening slurries.

Improvement approach	Outline of approach	Advantages	Disadvantages
Removal and replacement	Expansive soil removed and replaced by non-expansive fill to a depth necessary to prevent excessive heave. Depth governed by weight needed to prevent uplift and mitigate differential movement. Chen (1988) suggests a minimum of 1–1.3 m	Non-expansive fill can achieve increase bearing capacities; simple and easy to undertake; often quicker than alternatives	Preferable to use impervious fill to prevent water ingress which can be expensive; thickness required may be impractical; failure can occur during construction due to water ingress
Remoulding and compaction	Less expansion observed for soil compacted at low densities above OWC[1] than those at high densities and below OWC (see **Figure 31.15**). Standard compaction methods and control can be used to achieve target densities	Uses clay on site, eliminating cost of imported fill; can achieve a relatively impermeable fill, minimising water ingress; swell potential reduced without introducing excess water	Low density compaction may be detrimental to bearing capacity; may not be effective for soil of high swell potential; requires close and careful quality control
Pre-wetting or ponding	Water content increased to promote heave prior to construction. Dykes or berms used to impound water in flooded area. Alternatively, trenches and vertical drains can be used to speed infiltration of water into soil	Has been used successfully when soils have sufficiently high permeabilities to allow relatively quick water ingress, e.g. with fissure clays	May require several years to achieve adequate wetting; loss of strength and failure can occur; ingress limited to a depth less than the active zone; water redistribution can occur – causing heave after construction
Chemical stabilisation	Lime (3–8% by weight) common with cements (2–6% by weight) sometimes used, and salts, fly ash and organic compounds less commonly used. Generally lime mixed into surface (~300 mm), sealed, cured and then compacted. Lime may also be injected in slurry form. Lime generally best when dealing with highly plastic clays	All fine-grained soils can be treated by chemical stabilisers; effective in reducing plasticity and swell potential of an expansive soil	Soil chemistry may be detrimental to chemical treatment; health and safety need careful consideration as chemical stabilisers carry potential risks; environmental risks may also occur – e.g. quick lime is particularly reactive; curing inhibited in colder temperatures

[1] OWC – optimum water content, as determined by standard proctor test BS1377 (BSI, 1990).

Table 33.5 Soil stabilisation approaches applied to expansive soils
Data taken from Nelson and Miller (1992)

Detailed accounts of these are provided in both Chen (1988) and in Nelson and Miller (1992). In addition, electrochemical soil treatment approaches are being developed that utilise electrical current to inject stabilising agents into the soils. Further details are provided by Barker *et al*. (2004). As well as barrier methods, water management can be employed with restrictions applied to avoid irrigation within certain distances of the structure. However, monitoring is needed to ensure compliance with these restrictions.

33.5.4.4 Remedial options

Expansive soils cause significant damage to buildings, as discussed throughout this chapter, and so remedial action is required to repair any damage. However, it is important to establish a number of factors before embarking on a remedial plan. Key questions that should be considered are (after Nelson and Miller, 1992):

- Are remedial measures needed – is damage severe enough to warrant treatment?
- Is continued movement anticipated and so would it be better to wait?
- Who will pay?
- What criteria should be selected?
- How has the damage been caused and what is its extent?
- What remedial measures are applicable?
- Are there any residual risks post remediation?

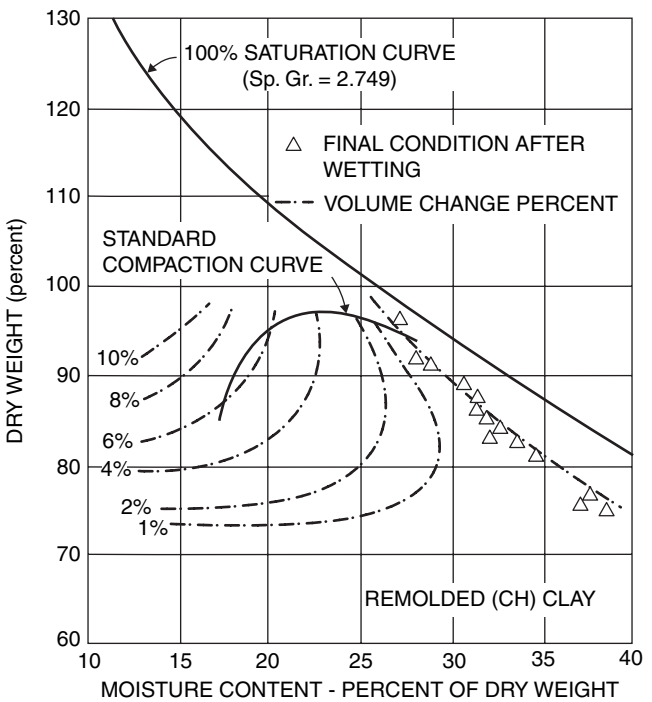


Figure 33.15 Percentage expansion for various placement conditions (c.f. Table 33.5)
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Clearly, to select an appropriate remedial measure, an adequate forensic site investigation is required. Key information required includes the cause and extent of the damage, the soil profile (as it is often difficult to determine whether settlement/heave is the cause of structural distress), and the soil's expansive potential. Other necessary information has already been discussed in section 33.5.1 above. Failure to carry out an adequate site investigation can lead to false diagnoses and inappropriate remedial measures employed. Further details are provided by Nelson and Miller (1992) as well as BRE Digests 251 (1995a), 298 (1999), 361 (1991), 412 (1996) and 471 (2002).

The following are examples of remedial measures employed for foundations:

- repair and replace structural elements or correct improper design features;
- underpin;
- provide structural adjustments of additional structural supports e.g. post tensioning;
- stiffen foundations;
- provide drainage control;
- stabilise water contents of foundation soils;
- install moisture barriers to control water content fluctuations.

Full underpinning of an operational structure is often impractical (and increasingly seen as unnecessary) and it is more common for underpinning work to be applied only to key parts of the foundations (Buzzi *et al*., 2010). Moreover, localised application of underpinning to deal with differential settlements may not improve the overall performance of the foundation (Walsh and Cameron, 1997). Thus any localised treatment must be designed to take account of all factors, otherwise there is a danger of exacerbating the problems due to the inherent natural spatial variability of expansive soils. Recently, underpinning using expanded polyurethane resin has met with some success, because resin can be injected using small diameter tubes directly where it is needed (Buzzi *et al*., 2010). However, due to concerns about its long-term stability and the possibility that swelling in injected soils could be exacerbated if all the cracks were filled, its adoption has been slow. However, a detailed experimental study (Buzzi *et al*., 2010) concluded that resin injected expansive soils did not exhibit enhanced swelling as a number of cracks remained unfilled, providing swell relief. Problems with lateral swelling can sometimes be accommodated by cracking within the soil matrix. However, if no cracks are present, problems can occur – particularly with retaining structures. Expanded polystyrene geofoam has demonstrated some success with dealing with lateral expansion, and has been shown to reduce the subsequent impact of vertical swelling (Ikizler *et al*., 2008).

With respect to pavements, distress can be considered as one of four possible types of damage, as highlighted in section 33.5.4.2 above. Most common remedial measures are

either removal and replacement, or construction of overlays. Whichever method is used, care is needed to ensure that the causes of the original distress are dealt with.

Many of the pre-construction approaches can also be used for post-construction treatments; for pavements these include moisture barriers, removal, replacement and compaction, and drainage control.

33.5.4.5 Domestic dwelling and vegetation

Tree roots will grow in the direction of least resistance and where they have the best access to water, air and nutrients (Roberts, 1976). The actual pattern of root growth depends upon, amongst other factors, the type of tree, the depth to the water table, and local ground conditions. Trees will tend to maintain a compact root system. However, when trees become very large, or where trees are under stress, they can send root systems far from the trunk. There is some published guidance on 'safe planting distances' that can be used by the insurance industry to inform householders of the potential impacts of different tree species on their properties. Further details are also given in NHBC (2011a).

Paving of previously open areas of land, such as the building of patios and driveways, can cause major disruption to the soil–water system. If the paving cuts off infiltration, many trees will send their roots deeper into the ground or further from the trunk in order to source water. The movement of these tree roots will cause disturbance of the ground and will lead to the removal of water from a larger area around the tree. Problems occur when houses are situated within the zone of influence of a tree (**Figure 33.16**).

If an impermeable method of paving is used, it may prevent water from penetrating into the ground. This can affect

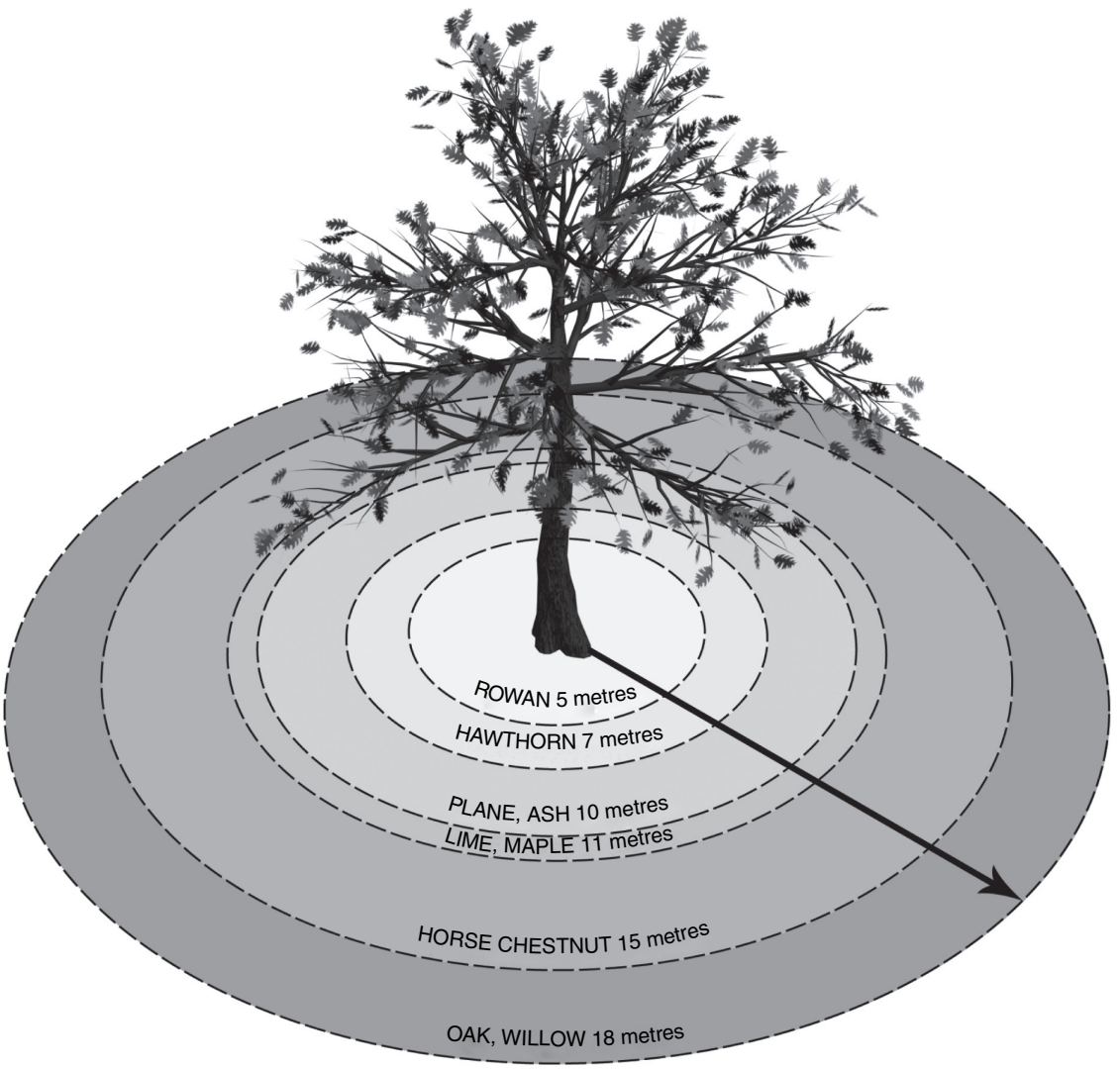


Figure 33.16 The zone of influence of some common UK trees
Reproduced from Jones *et al.* (2006) © NERC, with permission from The British Geological Survey

the shrink–swell behaviour of the ground and also the growing patterns of nearby trees. A well-designed impermeable paving system, in good condition, may actually reduce the amount of shrink–swell activity in the ground immediately below it. Paving moderates variations in water content of the soil and thus the range of shrink–swell behaviour. However, if the paving seal is broken, water can suddenly enter the system, causing swelling of the ground.

Different problems are faced when considering the distinctly separate areas of designing new build structures and remediating existing damaged buildings. New build guidelines for domestic dwellings recognise the need for thorough ground investigations to design systems to cope with the hazards presented by existing trees or their recent removal. Reference should be made to National House Building Council (NHBC) Standards Chapter 4.2 *Building Near Trees* (NHBC, 2011a) and the *Efficient Design of Piled Foundations for Low-Rise Housing – Design Guide* (NHBC, 2010). In the case of existing dwellings, a range of reports and digests are available (e.g. BRE Digests 298, 1999; 412, 1996) and *A Good Technical Practice Guide* provided by Driscoll and Skinner (2007).

Essentially, foundations should make allowances for trees in expansive (swell–shrink) soils and should take account of (NHBC, 2011a):

- shrinkage/heave linked to changes in water content;
- soil classification;
- water demand of trees (this is species-dependent);
- tree height;
- climate.

In the case of existing structures, the main cause of distress results from the effects of differential settlement, where different parts of the building move by varying amounts due to variations in the properties of the underlying soil. Equal or proportionate movements across the plan area of a building, though significant in terms of vertical movement, may result in little structural damage (IStructE, 1994). However, in the UK this is rare; by far the most overwhelming cause of damage to property results from the desiccation of clay subsoil which consequently causes differential settlements/movements, often stemming from the abstraction of water by the roots of nearby vegetation.

If vegetation is involved, it produces a characteristic seasonal pattern of foundation movement: subsidence in the summer, reaching a maximum around September, followed by upward recovery in the winter (see **Figure 33.17**). If subsidence followed by recovery is occurring, there is no need to try to demonstrate shrinkable clay or desiccation. No other cause produces a similar pattern – soil drying by vegetation must be involved (unless the foundations are less than 300 mm). Furthermore, there is no need to demonstrate the full cycle as it is sufficient to confirm movement is consistent with this pattern. Monitoring upward recovery in the winter is particularly valuable in this case. Further details are given by Crilly and Driscoll (2000) and Driscoll and Chown (2001), drawn from a test site in Chattenden, Kent, set in expansive London Clay (see **Figure 33.17**). In addition, both articles provide details of instrumented piles, discussing design implications.

Level monitoring can demonstrate this pattern. BRE Digest 344 (1995b) makes recommendations for the taking of measurements of the 'out-of-level' of a course of masonry or of the damp-proof course, which can be used to estimate the amount of differential settlement or heave that has already taken place. BRE Digest 386 (1993b) discusses precise levelling techniques and equipment which can monitor vertical movements with an accuracy consistently better than ±0.5 mm. Precise levelling can be conducted easily, quickly and accurately and so provides one of the most effective ways to distinguish between potential causes of foundation movement (Biddle, 2001).

The choice of mitigation should be proportionate to the problem and specific to the true area of the affected structure. It is important not to become distracted by extraneous but nevertheless interesting features.

Biddle (2001) suggests one of four remedial options to deal with the adverse actions of trees:

1. fell the offending tree to eliminate all future drying;
2. prune the tree to reduce drying and the amplitude of seasonal movement;
3. control the root spread to prevent drying under foundations;
4. provide supplementary watering to prevent soil from drying.

Biddle (2001) states that it is now recognised that in most situations, underpinning is unnecessary and that foundations can be stabilised by appropriate tree management – usually by felling the offending tree or by carrying out heavy crown reduction. Site investigations should reflect this change and be aimed at providing the information to allow appropriate decisions on tree management. In particular:

- confirmation that vegetation-related subsidence is involved;
- identification of which tree(s) or shrub(s) are involved;
- assessment of the risk of heave if a tree is felled or managed;
- identification of the need for any other site investigations;
- if the tree warrants retention, assessment of whether partial underpinning would be sufficient;
- confirmation that vegetation management has been effective in stabilising the foundations;
- provision of information within an acceptable timescale.

Trees are often pruned to reduce their water use and therefore their influence on the surrounding soil. However, unless the trees are thereafter subjected to a frequent and ongoing regime of management, the problems will very quickly return. Whilst

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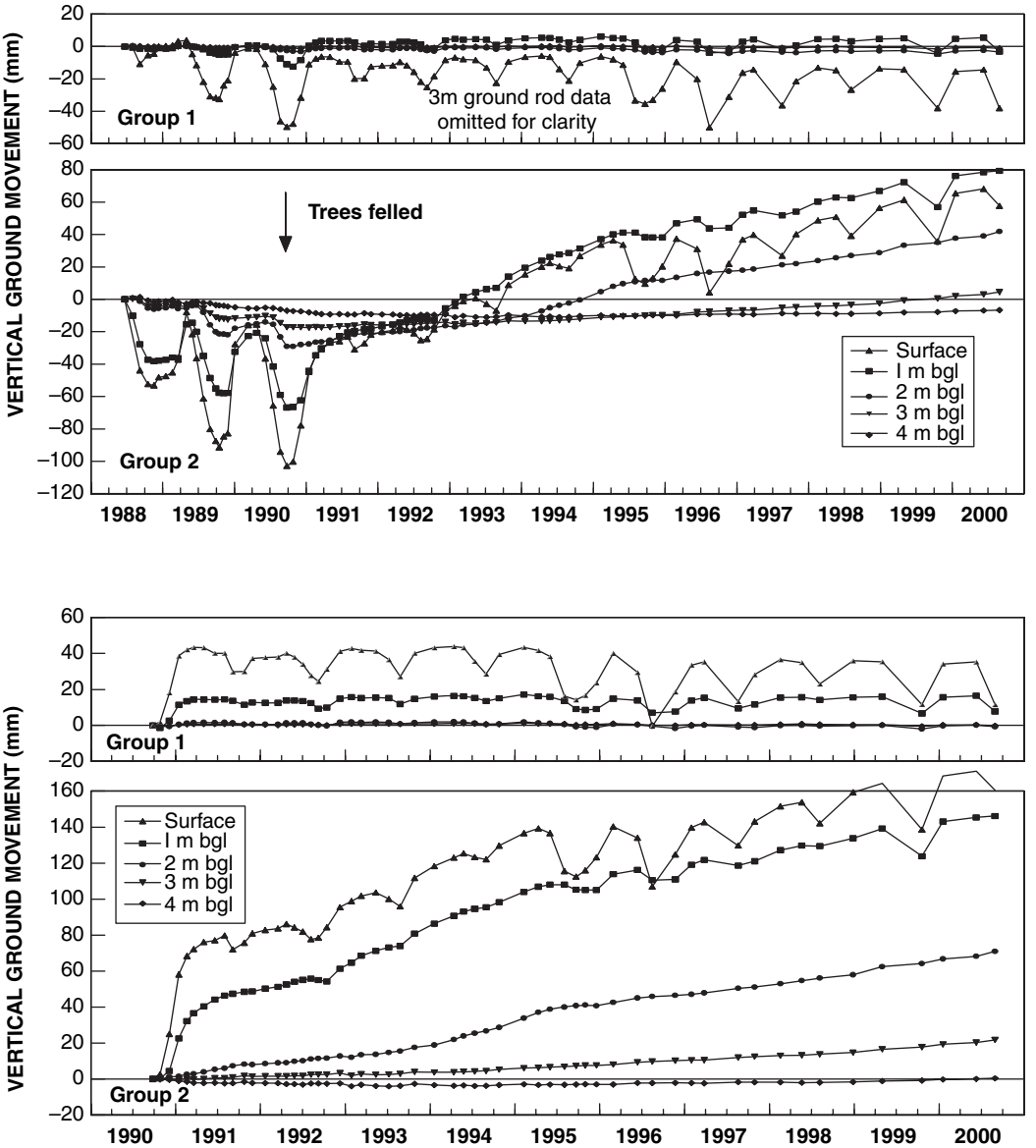


Figure 33.17 Examples of ground movements due to seasonal fluctuations at Chattenden. The upper plot shows results obtained since the first movements in June 1988. The lower plot shows an enlarged scale with results obtained since the trees were felled – group 1 is remote from tree and group 2 near to trees

Reproduced from Crilly and Driscoll (2000); Driscoll and Chown (2001); all rights reserved

tree removal will ultimately provide an absolute solution in the majority of cases, there are situations where this is not an option (e.g. protected trees, adverse risk of heave, incomplete evidence in contentious issues, and physical proximity of trees).

In the past, an obvious and often knee-jerk solution has been to provide significant and often disproportionate support to the structure through foundation strengthening schemes, incorporating various forms of underpinning. This approach is often ecologically, financially and technically incongruent with the problems faced. Alternatively, various forms of physical barriers can be used, constructed from, for example, *in situ* concrete. However, such barriers often prove ineffective over time. Barriers are currently being developed that incorporate a bioroot barrier, which is a mechanically-bonded geocomposite consisting of a copper-foil firmly embedded between two layers of geotextile. Such biobarriers are now being used specifically in arboriculture and for Japanese knotweed control where a permeable barrier is required. They act as signal barriers by diverting root growth (both biologically and physically) without making any attempt to physically restrain their progress.

Alternative remediation by supplementary watering is usually considered impractical due to the quantities required by the tree. This approach can suffer from the unavailability of water precisely when it is needed – due to prevailing drought conditions.

If a mature tree is felled, a building may incur heaving on a dry clay soil. Unfortunately, the evidence is rarely obvious; however, clues to look out for include:

- the house is new – less than 20 years old;
- there is expansive soil present;
- the crack pattern might appear a bit odd – wider at the bottom than at the top, with no obvious cause; and
- cracks continue to open, even in the wet months.

Heave problems can be costly and always require thorough investigation involving soil sampling, precise levels and aerial photographs. Heave is a threat but rarely a reality where established existing properties are involved, and the structure predates the planting of the tree.

Ultimately, if the offending tree can be accurately targeted and dealt with rapidly before the next growing season, the extent of any damage and need for remedial work will be kept to a minimum (Biddle, 2001).

33.6 Conclusions

Expansive soils are one of the most significant ground-related hazards found globally, costing billions of pounds annually. They are found throughout the world – commonly in arid/semi-arid regions – where their high suctions and potential for large water content changes can cause significant volume changes. In humid regions, such as the UK, problematic expansive behaviour generally occurs in clays of high plasticity index. Either way, expansive soils have the potential to demonstrate significant volume changes in direct response to changes in water content. This can be induced through water ingress, through modification to water conditions, or via the action of external influences such as trees.

To understand, and hence engineer expansive soils in an effective way, it is necessary to understand soil properties, suction/water conditions, water content variations (temporal and spatial), and the geometry/stiffness of foundations and associated structures. This chapter provides an overview of these features and includes methods to investigate expansive behaviour both in the field and in the laboratory, together with associated empirical and analytical tools to evaluate it. Following this design, options for pre- and post-construction are highlighted for both foundations and pavements, together with methods to ameliorate potentially damaging expansive behaviour, including dealing with the impact of trees.

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33.7.2 Useful websites

Association of British Insurers; www.abi.org.uk
British Geological Survey (BGS); www.bgs.ac.uk
International Society of Arboriculture, UK and Ireland Chapter; www.isa-arboriculture.org
Royal Institution of Chartered Surveyors; www.rics.org
Subsidence Claims Advisory Bureau; www.subsidencebureau.com
The Clay Research Group, UK; www.theclayresearchgroup.org
The Subsidence Forum; www.subsidenceforum.org
US Geological Survey (USGS); www.usgs.gov

It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 40 *The ground as a hazard*
- Chapter 76 *Issues for pavement design*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

Chapter 34

Non-engineered fills

Fred G. Bell British Geological Survey, UK
Martin G. Culshaw University of Birmingham and British Geological Survey, UK
Hilary D. Skinner Donaldson Associates Ltd, London, UK

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The term 'fill' (or 'made ground') is used to describe material that has been deposited by human processes. The material could be natural, or have been altered artificially prior to deposition. Fill is either engineered or non-engineered depending on whether any specific treatment during deposition takes place. As a result, a number of engineering challenges may exist and will require different remediation approaches to improve and mitigate problematical behaviour.

Non-engineered fill may consist of domestic waste, building waste, slag, mining and quarry waste, industrial waste and soil waste. However, methodologies for the classification and description of fills are not fully developed or standardised nationally or internationally. This affects the way in which fills are presented on geological maps, though standardisation in the UK is becoming established.

Non-engineered fills may settle variably, have poor bearing capacity and may suffer significant movements due to causes other than the imposed loading. Other problems associated with some wastes include contamination, spontaneous combustion and the emission of gas. The extent to which non-engineered fill will be suitable as a foundation material depends largely on its age, composition, uniformity, properties and the method by which the material was placed.

34.1 Introduction

The term 'fill' refers to material that has been deposited artificially as, for instance, in an excavation or as ground made by human activity. Collectively, these deposits, together with unfilled excavations, are sometimes termed 'artificial ground' with reference to their geographical distribution and method of placement, rather than their composition (for example, see Ford *et al.*, 2006). Non-engineered fill, as opposed to engineered fill, is material that has not been subject to some degree of controlled placement to ensure that the geotechnical properties of the finally placed material comply with a predetermined design. It is dumped, with little control, in deep lifts so that it is typically poorly compacted and, so, is likely to be in a loose state with varying geotechnical properties, both laterally and with depth. Terminology, in practice, can be confusing, as Norbury (2010) has pointed out, in that engineered fill is often referred to as 'fill' while non-engineered fill is referred to as 'made ground'. This usage does not coincide with that of the British Geological Survey (BGS) in mapping artificial ground (see section 34.3.1).

Due to the shortage of prime land for development in urban areas, artificial ground, of necessity, is being, and increasingly will be, used for building purposes (Charles, 2008). However, non-engineered fill is derived from a wide variety of materials including domestic refuse, ashes, slag, clinker, building waste, chemical waste, mining and quarry waste, as well as different types of soils. These materials, by their very nature, exhibit significant variability, both physically and chemically. Thus, the extent to which an existing non-engineered fill may be suitable as a foundation material depends largely on its composition and uniformity; its engineering behaviour is influenced by its method of placement and subsequent history. In the past, the control exercised in placing many fills was frequently insufficient to ensure appropriate engineering properties (hence the term 'non-engineered'). Consequently, non-engineered fills may have variable settlement characteristics (**Figure 34.1**) or poor bearing capacity, and may be compositionally variable over short distances and suffer significant movements due to causes other than the imposed loading. Other problems can include contamination, spontaneous combustion and the emission of gas, depending on the nature of the fill material. In addition, time-related changes in volume and composition are likely to take place or may be triggered by any stress or environmental changes.

This chapter first discusses the generic characteristics of non-engineered fills that cause their problematic properties and their often unpredictable engineering behaviour, which can yield significant problems for geotechnical engineers. An overview of how non-engineered fills are classified (as 'artificial ground'), mapped and described is followed by a discussion of the nature, properties and behaviour of a range of non-engineered fills. Chapter 58 *Building on fills* gives specific details of building and foundation aspects for fill materials.

34.2 Problematic characteristics

Fills exhibit a range of engineering properties quite as wide as that of natural soils. The behaviour of natural soils and rocks is strongly influenced by their structure and fabric. Although soil structure arises from many different causes, its effect on

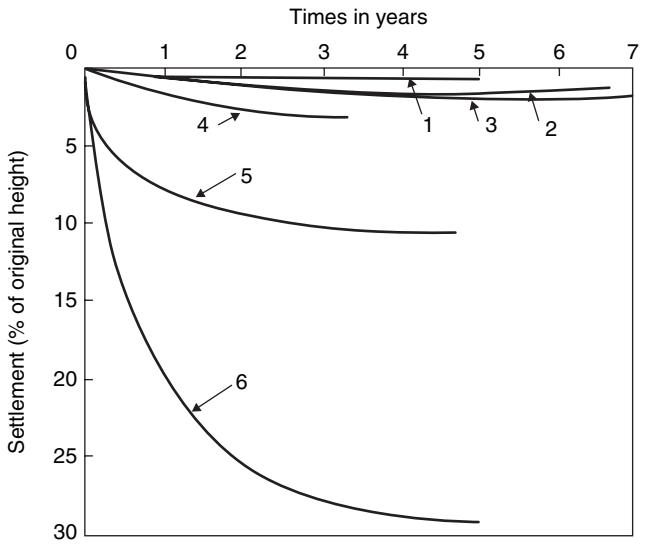


Figure 34.1 Observations of the settlement of various types of fill due to consolidation under its own weight. 1. Well-graded sand, well compacted. 2. Rockfill, medium state of compaction. 3. Clay or chalk, lightly compacted. 4. Sand, uncompacted. 5. Clay, uncompacted. 6. Mixed refuse, well compacted

Reproduced from Meyerhof (1951); Institution of Structural Engineers

behaviour is similar. As strength increases, the stress domain over which a soil exhibits stiffer behaviour is enlarged. In fills formed from natural soils, such structure will have been largely destroyed during excavation and placement, with little opportunity for it to be reinstated. Consequently, fill behaviour may be inferior to that of apparently similar natural soils.

Variability cannot be overemphasised. Whether between areas of infilled ground and natural ground, or between different areas of fill, variability can result in differential settlement or structural loading that is unacceptable. All analyses of fill must address the likely variations in properties and behaviour. Where fill density and history can be estimated, tests on representative, recompacted samples may give an indication of fill behaviour. The engineering behaviour of the material should be identified within the required context: for example, load-carrying capacity, volume changes in response to load or other factors, or strength. In addition, the potential for the generation of deleterious substances should be identified.

It is helpful to compare and contrast the properties of fills with the properties of natural soil, with the following factors in mind that will affect their engineering behaviour:

- Nature of the material: a fill may be composed of the same type of material as a natural soil (such as clay, sand or rock) or it could be composed of a range of other materials (such as wastes from chemical and industrial processes). The nature of the material will indicate the underlying best-case strength and stiffness behaviour and will affect the potential for any change in density, whether by mechanical, hydrological, biological or chemical causes.
- Method of deposition: a fill may have been deposited in a manner that is quite similar to the deposition of some natural soils (for example sedimentation under water) or it may have been deposited in a quite different manner (such as compaction). The method of placement and any subsequent activities will affect the density, water content and variability. Generally, like natural materials, a higher density at a lower moisture content is likely to demonstrate better engineering behaviour, although care should be taken where cementation or other 'metastable' conditions may occur.
- Age: many fills have been placed quite recently and the deposits are much younger than natural soils. Often, younger deposits are still undergoing movements arising from their placement or material type. In addition, changes in composition may be anticipated; for some fills that may mean poorer behaviour should be anticipated – for example, the ash content of more recent domestic refuse is lower, and the accompanying behaviour poorer, than older deposits.

See section 34.7.1, Further reading, and Chapter 58 *Building on fills* for more information.

34.3 Classification, mapping and description of artificial ground

34.3.1 Classification

Geotechnical engineers have typically divided 'fills' into those that are engineered and those that are not. However, when carrying out a site investigation on existing artificial ground it is not usually easy to make this distinction. Traditionally, geological surveyors have ignored artificial ground when creating a geological map. Engineering geologists, too, have paid little attention to the mapping of these deposits. For example, a report by the Geological Society Engineering Group Working Party on engineering geological mapping (Knill *et al*., 1970) only recognised 'made ground', 'spoil tip (made ground above natural surface level)' and 'backfilled opencast site or excavation'. However, the increased demand for land in urban areas has resulted in pressure to re-develop brownfield sites, including artificial ground. In turn, this has encouraged mapping geologists and engineering geologists to include these deposits on their maps. However, the simple mapping of artificial ground without systematically distinguishing between different categories does not meet the needs of developers and geotechnical engineers for more detailed information. In the last decade, a simple classification of artificial deposits has been used on geological maps in the UK. Ford *et al*. (2006) have extended the classification of Rosenbaum *et al*. (2003) and identified five principal classes of artificial ground (**Figure 34.2**):

1. *Made ground* is where material has been placed upon the pre-existing ground surface. Most engineered fill would fall into this class.
2. *Worked ground* is where there have been excavations into the pre-existing ground surface.
3. *Infilled ground* is a combination of one or more phases of excavation involving the extraction of material (worked ground) and one or more phases of deposition of material on the excavated surface or made ground (Price *et al*., 2011).

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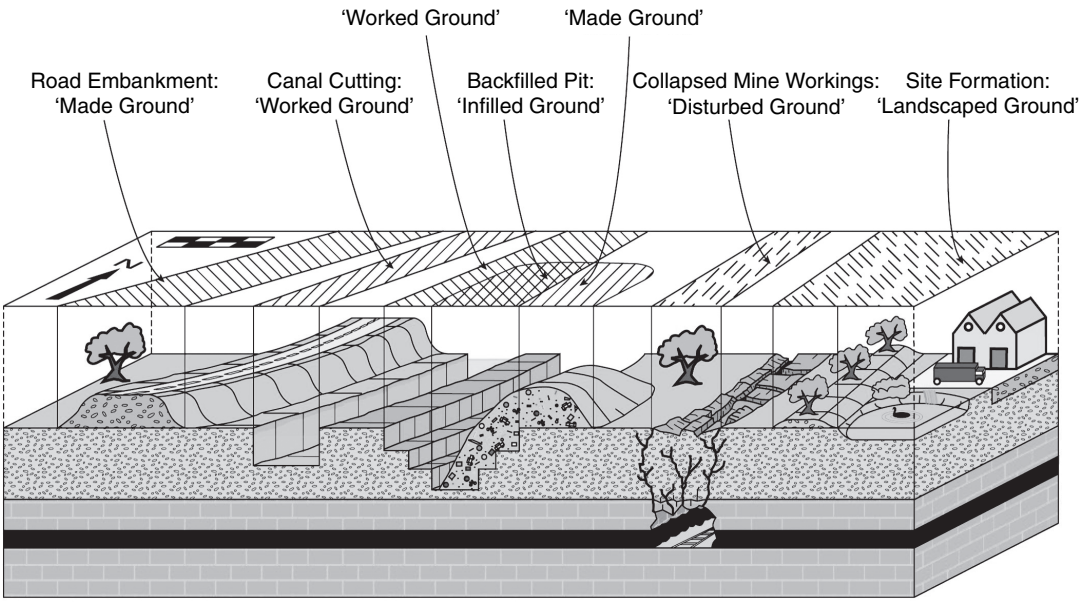


Figure 34.2 The main types of artificial ground and the way in which they are depicted on geological maps in the UK
Reproduced from Price *et al.* (2011); The Royal Society

4. *Disturbed ground* is where there has been surface or near-surface disturbance so that ill-defined excavations, areas of subsidence, spoil and other irregular features are associated in a complex way with each other: areas around mines often fall into this category.
5. *Landscaped ground* is where the original surface has been extensively remodelled, but where it is impractical or impossible to separately delineate areas of worked and made ground.

For each of the classes 1, 2 and 4, a three-tier hierarchy (class, type, unit) provides a flexible and logical sub-division into an increasingly descriptive set of lithostratigraphical categories for mapping purposes. The structure allows the user to select an appropriate level of detail according to available requirements, whilst providing a structure that is readily amenable to digital recording. Examples of the classification hierarchy are shown in **Figure 34.3**.

The typical usage is intended to be intuitive. The unit level gives the highest level of detail, allowing specific types of artificial ground to be recorded. This level of detail is suitable for mapping at 1:10 000 scale, which is the scale of most geological maps in the UK. Should a confident unit-level classification be impossible due to insufficient data or time, then a classification at the type level can be considered. The range of type-level categories provides a non-specific set of parent classes that retain a moderate level of detail. Class-level corresponds to the level of detail currently available on 1:50 000 scale geological maps published in the UK in the last 20 years or so. This is a very general classification level and specific observations and significant information may be lost through generalisation.

For landscaped ground, the classification is not sub-divided beyond type level. This is intended to avoid duplication of information that is available from other UK sources (for example, Ordnance Survey topographical maps and the National Land Use Database, which provides a comprehensive, consistent and up-to-date record of previously developed land in England that may be available for development).

34.3.2 Mapping

The mapping of artificial ground utilises the same principles as for superficial deposits and, to a lesser extent, bedrock. Mapping considers *morphology* (the shape of the ground), *origin* and *genesis* (the 'process' that has formed the deposit), *age* (important where several generations of deposits are present) and *lithology* (the composition of the materials themselves). This can be illustrated by comparing the way in which a railway embankment (class – made ground; type – embankment; unit – railway embankment) and an esker (an embankment-like natural deposit formed as a glacier retreats) are mapped. The two have similar morphologies, though an esker may well be more sinuous. Their genesis is clearly different: eskers are formed as glaciers retreat while railway embankments are formed during the construction of railway systems. Eskers have ages of thousands of years, while railway embankments are less than 200 years old. Eskers are composed largely of sand and gravel while railway embankments can be composed of a variety of materials, both natural and artificial, which have experienced varying degrees of engineering (usually compaction).

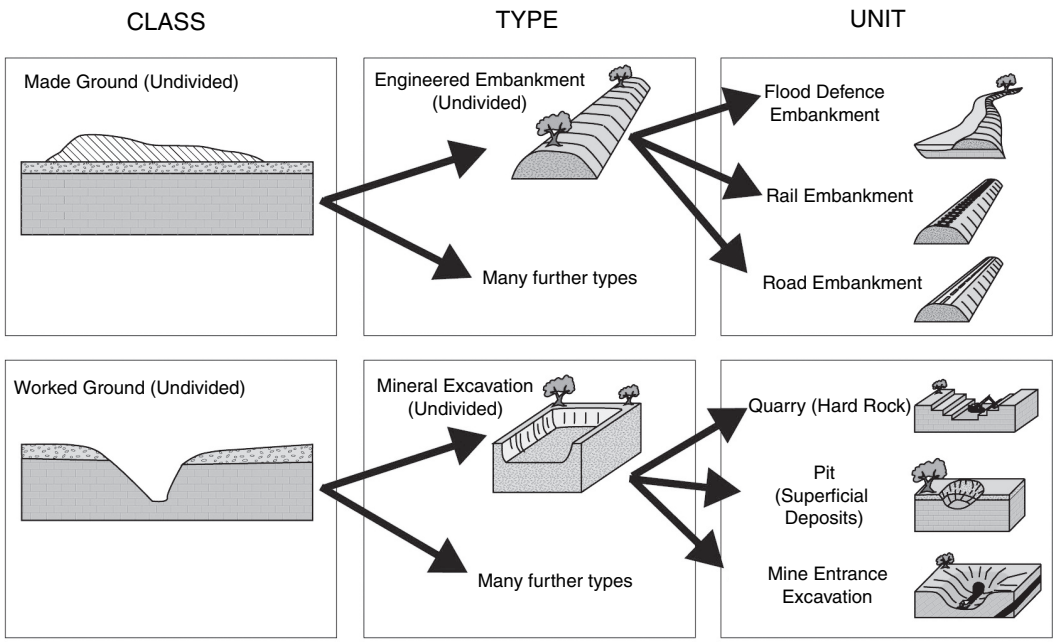


Figure 34.3 Examples of the artificial ground classification hierarchy
Reproduced from Price *et al.* (2011); The Royal Society

34.3.3 Description

The Geological Society Engineering Group Working Party reports on core logging and on engineering geological mapping (Knill *et al.*, 1970; Geological Society Engineering Group Working Party, 1972) do not refer to how fill materials should be described. However, the International Association of Engineering Geology's Commission on Engineering Geological Mapping has produced a brief guide to the mass characteristics of non-engineered fill that should be described (IAEG Commission, 1981):

- mode of origin, including method of placement;
- presence of larger objects;
- voids;
- chemical and organic matter;
- toxic materials including gases;
- age.

In terms of non-engineered fills, Norbury (2010) has suggested a more detailed approach. First, a sample of non-engineered fill material is divided into the very coarse, gravel and fine fractions. The proportion of each is determined and how the material is likely to behave in different situations (for example, in the sides of an excavation or in a spoil heap) is assessed. The material is then described in terms of its composition identifying the following:

a) natural soil materials;

b) artificial materials such as:

- asphalt;
- concrete;
- masonry, blockwork and brickwork;
- glass, metal, plastics;
- wood, paper;
- burnt products such as slag, clinker and ash including evidence by strong colours;
- large objects and obstructions, which may be critical to piling operations;
- hollow objects, compressible and collapsible objects or voids, which could be important for ground stability;
- chemical wastes and dangerous or hazardous materials;
- decomposable materials with a note of the degree of decomposition.

The proportions of each are also determined.

Norbury (2010) suggested that further information should be provided where possible covering:

- any dating for papers or other artefacts;
- signs of current or recent heat or combustion;
- smells or odours;
- layers and their inclination as a guide to the method of placement;
- origin of materials, if identifiable.

34.4 Types of non-engineered fill

Non-engineered fills generally are associated with the backfilling of opencast mineral extraction sites, disposal of waste and

with derelict land. Although a modern waste disposal site is engineered so that its construction should minimise the production of leachate and its infiltration into the ground, as well as dealing with any landfill gas produced, it is not necessarily designed to support the subsequent erection of buildings.

34.4.1 Filled opencast sites

34.4.1.1 Working of opencast sites

Stratified mineral deposits that occur at, or near, the surface, such as coal or sedimentary iron ore in relatively flat terrain, are generally mined by strip or opencast mining. A deposit will either be horizontal or gently dipping. In Britain, opencast coal mining involves the exploitation of shallow seams at sites normally from 10 to 800 ha in area. This usually involves creating a box-cut (an initial, trench-like excavation) to reveal the coal. The opencast working of coal involves its excavation to maximum depths of around 100 m below the surface with stripping ratios (the ratio of the amount of overburden or non-coal material to the amount of coal recovered) of up to 25:1. Hence, opencast mining has been a major producer of deep fills, the geotechnical properties of which are of notable importance if, after restoration, they are to be considered for building development. In this context, the major factor according to Charles (2008) is the possibility of long-term settlement of the backfill.

An opencast area will be worked on a broad front with face lengths of 3 to 5 km not being uncommon. Once the coal has been extracted, the box-cut is filled with overburden from the next cut. Draglines may be used for overburden (primarily sandstone and mudrock) removal along with face shovels and dump trucks, and scrapers. The topsoil is removed separately by scraper and placed in dumps around the site for use in subsequent restoration. Similarly, the subsoil is stripped and stored temporarily in separate dumps about the site, again for restoration purposes. Face shovels are used to excavate the initial box-cut and for subsequent forward reduction of overburden. The material from the box-cut is placed above ground in a suitable position in relation to filling the excavation so that later re-handling is minimised. Draglines excavate the lower seams in a progressive strip cut behind the face shovels. Drilling and blasting is carried out where necessary. Because of the short lifespan of spoil heaps, they can often be maintained at their natural angle of repose.

Restoration can begin before a site is closed; indeed, this usually is the more convenient method. Hence, worked-out areas behind the excavation front are filled with rock waste. This means that the final contours can be designed with less spoil movement than if the two operations were undertaken separately. Furthermore, more soil for spreading can be conserved when restoration and coal working are carried out simultaneously. Because of high stripping ratios (often 15:1 to 25:1), there is usually enough spoil to more or less fill an excavation. Once the spoil has been re-graded, it is covered with topsoil. The restored land is generally used for agriculture or forestry but it can be used for country parks, golf courses, housing or light industrial estates with suitable engineering.

34.4.1.2 Properties and engineering behaviour

The properties of some backfills of opencast mines were given by Charles and Watts (2001) and have been summarised by Charles (2008); see **Table 34.1**. However, many fills are heterogeneous and may contain large boulders of mudstone, siltstone or sandstone so the geotechnical properties quoted should be regarded as indicative values, at best. Hence, field measurements taken at opencast sites are of special value in understanding the geotechnical behaviour of a backfill. Instrumentation and monitoring of a fill is carried out once filling is finished (see Chapter 58 *Building on fills* for further details).

The water table at many opencast sites is lowered by pumping to provide dry working conditions in the pit. If, when worked out, a site is to be restored for agricultural use or for

(a) Coarse backfills

Location	Fill type	Silt and clay %	Dry density (ρd) Mg/m³	Particle density (ρs) Mg/m³	Moisture content (w) %	Porosity (n) %	Air voids (V_a) %
Horsley	Mudstone and sandstone	10	1.70	2.54	7	33	21
Blindwells	Mudstone and sandstone	20	1.56	2.45	7	38	23
Tamworth	Clay with shale fragments	45	1.78	2.62	9	32	16

(b) Clay backfills

Location	Moisture content (w) %		Plastic limit (w_P) %	Liquid limit (w_L) %	Undrained shear strength (w_u) kPa
	Mean	Range			
Ilkeston	19	12–25	23	41	150
Corby	18	7–28	17	28	100

Table 34.1 Some properties of opencast mining backfills

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forestry, then this takes place without control of compaction. However, if a site is to be built over, then settlement is likely to be a problem without proper compaction. In particular, significant settlements of opencast backfill can occur when the partially saturated material becomes saturated by rising groundwater after pumping has ceased. For example, Charles *et al.* (1993) referred to an opencast site in Northumberland, where the waste was backfilled without any systematic compaction. They recorded that when the water table rose some 0.33 m into the fill, settlement occurred where the fill was 63 m in depth.

Settlement due to wetting collapse is more significant than that due to the self-weight of the backfill, which can give rise to difficulties in settlement prediction (Blanchfield and Anderson, 2000). Therefore, if a site is to be built on after restoration, then ideally the waste should be compacted properly. However, the subsequent development of a site will frequently be some time after an opencast area has been worked out and backfilled. As a consequence, such areas require investigation prior to any building development to determine the moisture condition of the backfill material and to determine how much it has been compacted (if at all) (Occupational Health and Safety Information Service, 1983). Consequently, Charles (1993) recommended that if an opencast site is to be built over but the backfill was not properly compacted, then collapse compression could be realised prior to development by inundation. Indeed, Charles and Skinner (2001) maintained that the susceptibility to collapse compression on inundation often represents a notable hazard when construction is to take place on fill. Most types of partially saturated fill, which are in a sufficiently loose or dry state, are susceptible to collapse compression over a wide range of applied loads when first inundated. Inundation can result from the downward percolation of water infiltrating from the surface or can be due to rising groundwater levels after pumping has ceased. Collapse compression can cause serious damage to any buildings placed upon such fills. However, in many instances flooding is difficult to carry out in a controlled and effective way. Alternatively, the ground could be pre-loaded or subjected to deep compaction by either vibrocompaction or dynamic compaction, or the structures could be designed to accommodate any subsequent ground movements. However, placing vibro stone columns in opencast backfill, according to Charles and Watts (2001), might provide a means of allowing surface water into the fill and thereby cause collapse compression.

Alternatively, where backfilled opencast coal workings exceed 30 m in depth, because greater settlements may occur, Kilkenny (1968) recommended that the minimum time that should elapse before development takes place should be 12 years after restoration is complete. He noted that the settlement of opencast backfill appeared to be complete within 5 to 10 years after the operation. For example, comprehensive observations of the opencast restored area at Chibburn, Northumberland, which was 23 to 38 m in depth, revealed that the ultimate settlement amounted to approximately 1.2% of the fill thickness and that some 50% of the settlement was complete after 2 years and 75% within 5 years. In shallow opencast fills, that is, up to 20 m in depth, settlements of up to 75 mm have been observed. Greater settlement may occur if there is movement of groundwater through a permeable fill, including a rising water table as noted above, as a result of the breakdown of point-to-point contacts in the shale or sandstone components of the fill. In such circumstances settlement may continue over a greater length of time than noted by Kilkenny.

Restoration of other opencast mineral workings can be dealt with in a similar way to those for coal. Another mineral that has been extracted on a large scale by opencast mining was sedimentary iron ore; for example, this was worked near Scunthorpe, Lincolnshire, and near Corby in Northamptonshire. In the latter area Penman and Godwin (1975) noted that the maximum rates of settlement occurred immediately after the construction of two storey semi-detached houses on an old opencast site that had been backfilled. Settlement decreased to small rates after about 4 years. They suggested that two of the causes of settlement in this fill were creep, which is proportional to log time, and partial inundation. Similar conclusions were reached by Sowers *et al.* (1965). The houses at Corby were constructed 12 years after the fill was placed. The amount of damage that they suffered was relatively small and was attributable to differential settlement; it was not related to the type of foundation structure used. These were either strip footings or reinforced-concrete rafts with edge beams.

34.4.2 Domestic waste disposal or sanitary landfill

With increasing industrialisation, technical development and economic growth, the quantity of waste produced by society has increased immensely. In addition, in developed countries the nature and composition of waste has evolved over the decades, reflecting industrial and domestic practices. For example, in Britain, domestic waste has changed significantly since the 1950s, from largely ashes with little putrescible content of relatively high density to low density, highly putrescible waste. Although domestic waste is disposed of in a number of ways, quantitatively the most important method is placement in a landfill. In England in 2003–2004, for example, of approximately 25.2 million tonnes of domestic solid waste produced about 72% was disposed of in landfills (Parliamentary Office of Science and Technology, 2005). However, by 2007–2008, the amount recycled had increased to 35% (Office of National Statistics, 2009). Landfill is controlled through the European Landfill Directive (Council of the European Union, 1999), implemented in the UK through the Landfill (England and Wales) Regulations 2002 and the Landfill (Scotland) Regulations 2003. Since the introduction of these regulations, the amount of waste sent to landfill has reduced significantly. Increasingly, waste materials from different sectors (e.g. construction or household) are recycled. However, geotechnical engineers commonly encounter historical landfill sites, which, depending on their time of operation, will exhibit significantly different physical and chemical characteristics. Moreover, landfills are active sites with a

number of time-dependent changes occurring throughout its life. The Environment Agency has published a series of guides covering waste acceptance; monitoring of landfill leachate, groundwater and surface water; landfill engineering; landfill gas; and landfill permitting and surrender (www.environment-agency.gov.uk/business/sectors/108918.aspx).

34.4.2.1 Composition

Waste disposal or sanitary landfills are usually very mixed in composition (**Table 34.2**) and suffer from continuing organic decomposition and physicochemical breakdown. They can consist of a heterogeneous collection of almost anything: waste food, garden rubbish, paper, plastic, glass, rubber, cloth, ashes, building waste, metals, etc. In future, recycling is likely to change the character of waste. Nonetheless, matter exists in the gaseous, liquid and solid states in landfills and all landfills comprise a delicate and shifting balance between the three states. Any assessment of the state of a landfill and its environment must take into consideration the substances present in it, and their mobility now and in the future. Much of the material of which a landfill is composed is capable of reacting with water to give a liquid rich in organic matter, mineral salts and bacteria, commonly known as leachate. Leachate is formed when rainfall infiltrates into a landfill and dissolves the soluble fraction of the waste and from soluble products formed as a result of chemical and biochemical processes occurring within the decaying waste. Methane and carbon dioxide are the primary gases produced in landfills together with traces of organic gases, carbon monoxide and hydrogen sulfide. The concentration of methane can range from 20% to 65%, carbon dioxide from 15% to 40%, while other gases typically make up <1% (Wilson *et al.*, 2007). In very general terms, gas production is likely to have reduced to a very low level after about 50–60 years (Nastev 1998). Gases are produced in the breakdown process and accumulations of gas in pockets in fills have led to explosions, for example, following the escape of methane at Loscoe in Derbyshire (Williams and Aitkenhead, 1991). Some materials such as ashes and industrial waste may contain sulfate and other products that are potentially deleterious to concrete.

Material	Characteristics as fill
Garbage: food, waste	Wet. Ferments and decays readily. Compressible, weak.
Paper, cloth	Dry to damp. Decays and burns. Compressible.
Garden refuse	Damp. Ferments, decays, burns. Compressible.
Plastic	Dry. Decay resistant, may burn. Compressible.
Hollow metal, e.g. drums	Dry. Corrodible and crushable.
Massive metal	Dry. Slightly corrodible. Rigid.
Rubber, e.g. tyres	Dry. Resilient, burns, decay resistant. Compressible.
Glass	Dry. Decay resistant. Crushable and compressible.
Demolition timber	Dry. Decays and burns. Crushable.
Building rubble	Damp. Decay resistant. Crushable and erodible.
Ashes, clinker and chemical wastes	Damp. Compressible, active chemically and partially soluble.

Table 34.2 Municipal waste materials incorporated in fills

34.4.2.2 Landfill design

The design of a landfill site is influenced by the physical and biochemical properties of the waste and the need to control leachate production. Post-closure settlement of sites is taken into account at a very early stage in the design of a landfill. Indeed, modern landfill disposal facilities require detailed investigations to ensure that appropriate design and safety precautions are undertaken. Furthermore, legislation generally requires that those responsible for waste disposal facilities guarantee that sites are suitably contained so as to prevent harm to the environment (for further details see the Environment Agency's website). Geological and hydrogeological conditions must be taken into account when selecting a landfill site (Proske *et al.*, 2005). However, the selection also involves economic and social factors. It should be noted, also, that the methods used to contain landfill have changed considerably. It was not until the late 1970s and early 1980s that landfill liners were used; prior to this leachate and gas control was through dilute-and-disperse techniques. Since then, landfill liner and capping construction have become ever more sophisticated, with double lining systems now being used, combining clay (or a suitable substitute using bentonite enriched sands or clays, or engineered spoil material, such as colliery spoil) and plastic HDPE (high-density polyethylene) liners. Included in a lining system there will also be a series of leachate and gas collection layers and protection media, so that liners are complex engineered inclusions within a landfill matrix. Great care is needed to avoid compromising the lining system during investigation or ground treatment (for example, through piling), otherwise pollution linkages will be generated allowing possible migration to a number of sensitive receptors. **Figure 34.4** shows the early stages of construction of a cell within a landfill site.

However, it is vital that monitoring continues during and after the construction of a landfill, particularly with regard to leachate and gas emissions. Boreholes through the base of a leachate-containing landfill to check what is happening should only be used if precautions are taken to avoid leakage. Geophysical methods are being used increasingly for this purpose (for example, see Kuras *et al.*, 2006).

At the present time there is no standard rule as to how waste should be dumped or compacted. Nonetheless, compaction is important since it reduces settlement and hydraulic conductivity, while increasing shear strength and bearing capacity. Moreover, the smaller the quantity of air trapped within landfill waste, the lower the potential for spontaneous combustion. Modern well-designed landfills usually possess a cellular structure, as well



Figure 34.4 Early stages in the construction of a landfill cell in north-east England. The liner and a high permeability drainage layer are being placed
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Figure 34.5 Cellular structure of landfill near Kansas City, USA

as a lining and a cover, that is, the waste is contained within a series of cells formed of clay (see **Figure 34.5**). However, certain leachates have been troublesome for some clay liners. Those consisting of organic solvents and those containing high levels of dissolved salts can give rise to cracking and the development of pipes in clay. However, clays can be treated with special polymers to reduce their sensitivity to potential contaminants. Uneven settlement of a landfill can cause fractures within the cover, which allow water to percolate into the fill. If conventional compaction techniques are used they do not always achieve effective results, especially with non-uniform waste. An additional complication is the time dependency of waste degradation, resulting in further settlement and the potential for cracking of capping layers.

34.4.2.3 Geotechnical properties

Waste materials in landfills have very variable properties with dry densities varying from 0.16 to 0.35 Mg/m^3 when tipped; after compaction the density may exceed 0.60 Mg/m^3. The moisture content ranges from 10% to 50%, the average specific gravity of the solids varies from 1.7 to 2.5, and the low bearing capacities are 19 to 34 kPa. Baling or shredding the waste materials before deposition improves their *in situ* properties but at greatly increased cost. Further details of typical engineering properties of landfill are given by Dixon *et al.* (1998).

34.4.2.4 Decomposition

Initially, the decomposition of organic waste is aerobic. Once decomposition starts, the oxygen in the waste is rapidly exhausted and so the waste becomes anaerobic. There are basically two processes for the anaerobic decomposition of organic waste. Initially, complex organic materials are broken down into simpler organic substances, which are typified by various acids and alcohols. Any nitrogen present in the original organic material tends to be converted into ammonium ions that are readily soluble and may give rise to significant quantities of ammonia in the leachate. The reducing environment converts oxidised ions such as those in ferric salts to the ferrous state. Ferrous salts are more soluble and, therefore, iron is leached from the landfill. The sulfate in the landfill may be reduced biochemically to sulfide. Although this may lead to the production of small quantities of hydrogen sulfide, the sulfide tends to remain in the landfill as highly insoluble metal sulfides. In a young landfill the dissolved salt content may exceed 10 000 mg per litre, with relatively high concentrations of sodium, calcium, chloride, sulfate and iron, whereas as the landfill ages the concentration of such inorganic materials usually decreases. Also, suspended particles may be present in the leachate due to the washout of fine material from the landfill.

The second stage of anaerobic decomposition involves the formation of methane. In other words, methanogenic bacteria use the end-products of the first stage of anaerobic decomposition to produce methane and carbon dioxide. Methanogenic bacteria prefer neutral conditions, so that excessive acid formation in the first stage will inhibit their activity.

As a landfill ages, because much of the readily biodegradable material has been broken down, the organic content of the leachate decreases. The acids associated with older landfills are not readily biodegradable. Consequently, the biochemical oxygen demand (BOD) in the leachate changes with time, increasing to a peak as the microbial activity increases. This peak is reached between six months and 2.5 years after tipping.

Methane production can constitute a dangerous hazard because methane is combustible and in certain concentrations in air is explosive (5–15% by volume in air), as well as asphyxiating. In many instances, landfill gas is able to disperse safely into the atmosphere from the surface of a landfill. However, when a landfill is completely covered with a soil capping of low permeability to limit leachate generation, the potential for the

gas to migrate along unknown pathways increases and there are cases of hazards arising from methane migration. Furthermore, there are unfortunately cases on record of explosions occurring in buildings due to the ignition of accumulated methane derived from landfills near to, or on, which they were built (Williams and Aitkenhead, 1991).

The source of gas should be identified so that remedial action can be taken. Generally, the connection between a source of methane and the location where it is detected can be verified by detecting a component of the gas that is specific to the source or by establishing the existence of a migration pathway from the source to the location where the gas is detected. Identification of the source can involve a drilling programme and analysis of the gas recovered. For instance, in a case referred to by Raybould and Anderson (1987), distinction had to be made between household gas and gas from sewers, old coal mines and a landfill before remedial action could be taken to eliminate the gas hazard affecting several houses. Proper closure of a landfill site can require gas management by passive venting, power operated venting or the use of an impermeable barrier. Measures to prevent migration of the gas include impermeable barriers (clay, bentonite, geomembranes or cement) and gas venting. Venting either wastes the gas into the atmosphere or facilitates its collection for utilisation. Where atmospheric venting is insufficient to control the discharge of gas, an active or forced venting system is used. Further details on the investigation of gas hazard in landfills and of remedial measures are given by Wilson *et al.* (2007) and references therein.

Other hazardous situations can occur due to microbial activity in a landfill, which generates heat. Hence, the temperature in a landfill rises during biodegradation to between 24 °C and 45 °C, although temperatures of up to 70 °C have been recorded. In some cases landfills have caught fire, increasing the risk still further, though the risk is relatively low; Roche (1996) reported fire problems in around 1 in 600 landfill sites in England and Wales compared with surface water pollution, groundwater pollution and gas migration, each affecting about 1 in 60 sites. A fire at the Lean Quarry landfill site near Liskeard in southeast Cornwall in June 2010 took over 12 hours to extinguish. A fire in January 2011 at the Kerdiffstown landfill near Naas in Ireland burned for several weeks.

34.4.2.5 Settlement

McDougall *et al*. (2004) noted that settlements associated with landfills are likely to be large and irregular. According to Sowers (1973) the mechanisms responsible for the settlement of waste disposal fills include mechanical distortion, bending, crushing and reorientation of materials, which cause a reduction in the void ratio; ravelling (that is, the transfer of fines into the voids); physicochemical and biochemical changes such as corrosion, combustion and fermentation; and the interactions of these various mechanisms. The initial mechanical settlement of waste disposal fills is rapid and is due to a reduction in the initial void ratio. It takes place with no build-up of pore water pressure. This primary compression takes place within a few days of further material being placed on top (Powrie *et al.*, 1998). Settlement continues due to a combination of secondary compression (that is, material disturbance) and physicochemical and biochemical action, and Sowers showed that the settlement log-time relationship is more or less linear. The rate of settlement produced by ravelling and combustion is erratic. Hence, the ultimate settlement is related to the initial void ratio and the environmental conditions favourable to deterioration, decay, ravelling and combustion.

The prediction of the settlement at a landfill site is a vital element for effective design, particularly in relation to post-closure settlement during site redevelopment. However, a determination of the amount and rate of settlement of a landfill is not a simple task. Secondary settlement of between 15% and 50% of the original fill thickness has been suggested (Powrie *et al.*, 1998) while Ling *et al.* (1998) gave figures of 30–40% for overall settlement. The heterogeneous nature of the materials involved, the fact that different materials may have been disposed of in different places and at different times, means that traditional soil mechanics for settlement prediction is generally unsatisfactory. Not only would settlement, especially large differential settlement, adversely affect any future construction on such a site but it may damage the cover system or lining, for example, geomembrane linings may be torn. Accordingly, Ling *et al.* (1998) reviewed the various empirical methods of estimating the settlement of landfills and then proposed a further method for evaluating the amount and rate of settlement. Watts and Charles (1999) also discussed the settlement characteristics of landfills and suggested various ground improvement techniques that could be used; these are discussed in Chapter 84 G*round improvement*.

34.4.2.6 Stabilisation

The stabilisation of soft landfills can be realised by mixing with soil, fly ash, incinerator residue, lime or cement. When a minimum unconfined compressive strength of 24 kPa occurs due, for example, to irregularities in mixing at the site, a geosynthetic material (for example, a geoweb) should be used to span such areas and should be securely anchored in a trench at the perimeter of the site. A geomembrane should be placed over the geoweb to provide a secondary seal protecting against the infiltration of surface liquids. Clay soil above the geomembrane acts as a seal but it must be as thin as possible so as not to overload the geoweb and geomembrane.

34.4.3 Coarse colliery discard

Spoil heaps associated with coal mines are ugly blemishes on the landscape and have a blighting effect on the environment (see **Figure 34.6**). In the past, these had the potential for catastrophic failure with devastating consequences (for example, the Aberfan failure in October 1966). Spoil heaps consist of coarse discard, that is, run-of-mine material that reflects the various rock types (mudrock, siltstone and sandstone) that are extracted during mining operations. Coarse discard also contains varying



Figure 34.6 (a) Spoil tips (one of which has been 'landscaped') at International Colliery, Blaengarw, Garw Valley, South Wales; (b) Spoil tip at the abandoned Greenside lead mine tip in the Glenridding Valley, Cumbria, England

(a) © NERC, with permission from the British Geological Survey; (b) © Paul Nathanail, with permission

amounts of coaly material that has not been separated by the preparation process. Obviously, the characteristics of coarse colliery discard differ according to the nature of the spoil. The method of tipping also influences the character of a coarse discard. In addition, some spoil heaps, particularly those with relatively high coal contents, may be burnt, or still be burning, and this affects their mineralogical composition.

34.4.3.1 Composition and properties

In Britain illites and mixed-layer clays are the principal components of unburned spoil in English and Welsh tips (Taylor, 1975). Although kaolinite is a common constituent in Northumberland and Durham, it averages only 10.5% of the discard of other areas. Quartz exceeds the organic carbon or coal content, but the latter is significant in that it acts as the major diluent, that is, the clay mineral content tends to decrease as the carbon content increases. Sulfates, feldspars, calcite and other materials average less than 2%.

The chemical composition of spoil material reflects the mineralogical composition. Free silica may be present in concentrations up to 80% and above, and combined silica in the form of clay minerals may range up to 60%. Concentrations of aluminium oxide may be between a few per cent and 40% or so. Calcium, magnesium, iron, sodium, potassium and titanium oxides may be present in concentrations of a few per cent. Lower amounts of manganese and phosphorus also may be present, with copper, nickel, lead and zinc in trace amounts. The sulfur content of fresh spoils is often less than 1% and occurs as organic sulfur in coal, and in pyrite. Pyrite is an unstable mineral, breaking down quickly under the influence of weathering (see also Chapter 36 *Mudrocks, clays and pyrite*). The primary oxidation products of pyrite are ferrous and ferric sulfates and sulfuric acid. A chemical attack by sulfate on concrete can lead to its deterioration. The oxidation of pyrite within spoil heap waste is governed by the access of air, which, in turn, depends upon the particle-size distribution, water saturation, and the degree of compaction. However, any highly acidic oxidation products, which may form, will be neutralised by alkaline materials in the waste material. Charles and Watts (2001) noted that pyrite in some shales has led to expansion through slow oxidation.

The moisture content of spoil increases with increasing content of fines. It also is influenced by the permeability of the material, the topography and climatic conditions. Generally, it falls within the range 5% to 15% (**Table 34.3**).

The range of specific gravity values depends on the relative proportions of coal, shale, mudstone and sandstone in the waste and tends to vary between 1.7 and 2.7. The proportion of coal is of particular importance: the higher the coal content, the lower the specific gravity. The bulk density of the material in spoil heaps shows a wide variation, with most material falling within the range 1.5 to 2.5 Mg/m^3. Low densities are mainly a function of low specific gravity. The bulk density tends to increase with increasing clay content.

The argillaceous content influences the grading of the spoil, although most spoil material is essentially granular (**Table 34.3**). In fact, as far as the particle-size distribution of a coarse discard is concerned there is a wide variation; often most material may fall within the sand range but significant proportions in the gravel and cobble ranges may also be present. Indeed, at placement a coarse discard very often consists mainly of gravel to cobble-sized particles but subsequent breakdown on weathering reduces the particle size. Once buried within a spoil heap, particles of a coarse discard undergo little further reduction in size. Hence, older and surface samples of spoil contain a higher proportion of fines than those obtained from depth.

The liquid and plastic limits can provide a rough guide to the engineering characteristics of a soil. However, for a coarse discard they are only representative of that fraction passing the 425-μm-BS sieve, which frequently is less than 40% of the sample concerned. Nevertheless, the results of consistency tests indicate a low to medium plasticity, whilst in certain

	Yorkshire Main	Brancepath	Wharncliffe
Moisture content, %	8.0–13.6	5.3–11.9	6–13
Bulk density, Mg/m³	1.67–2.19	1.27–1.88	1.58–2.21
Dry density, Mg/m³	1.51–1.94	1.06–1.68	1.39–1.91
Specific gravity	2.04–2.63	1.81–2.54	2.16–2.61
Plastic limit, %	16–25	Non-plastic–35	14–21
Liquid limit, %	23–44	23–42	25–46
Permeability, m/s	(1.42–9.78) × 10–6	?	?
Size, < 0.002 mm, %	0–17	Most material of sand-sized range	2–20
Size, > 2.0 mm, %	30–57		38–67
Friction angle, ϕ	31.5–35.0°	27.5–39.5°	29–37°
Cohesion intercept, c, kPa	19–21	4–39	16–40

Table 34.3 Examples of soil properties of coarse discard
Data taken from Bell (1996)

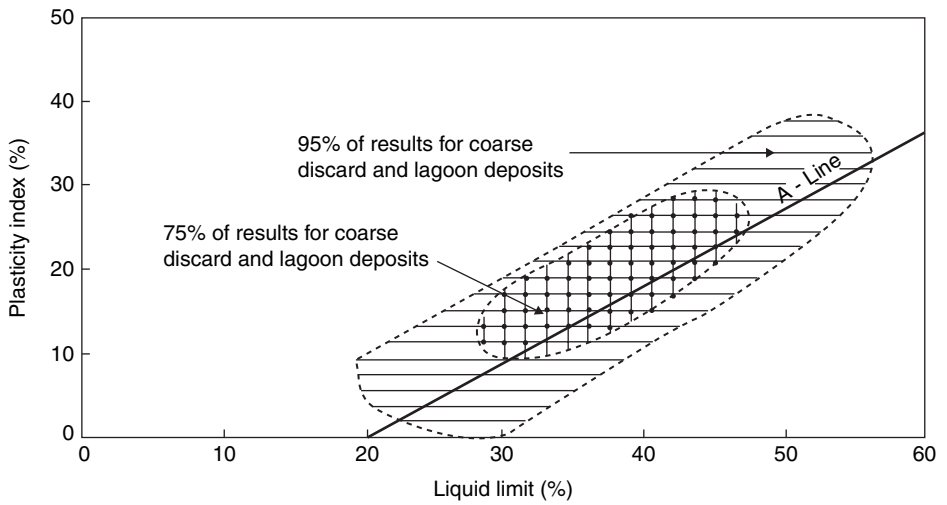


Figure 34.7 General range of plasticity characteristics of the coarse discard found in spoil heaps in the UK
Reproduced from Thomson (1973); National Coal Board

instances spoil has proved to be virtually non-plastic (**Figure 34.7**). Plasticity increases with increasing clay content.

34.4.3.2 Weathering

The most significant change in the character of a coarse colliery discard due to weathering is the reduction of particle size. The extent to which breakdown occurs depends upon the type of parent material involved and the effects of air, water and handling between mining and placing on the spoil heap. After a few months of weathering the debris from sandstones and siltstones usually consists of larger than cobble-sized particles. After that, the degradation to component grains takes place at a slow rate. However, mudstones, shales and seatearth exhibit rapid disintegration to gravel-sized particles. Although a coarse discard may reach its final level of degradation within a matter of months, with the degradation of many mudstones and shales taking place within days, once it is buried within a spoil heap it suffers little change. When spoil material is burnt, it becomes much more stable with respect to weathering.

Taylor and Smith (1986) stated that mudrock breakdown resulted from either mechanical (air breakage) or physicochemical mechanisms. Seedsman (1993) divided physicochemical swelling into crystalline and osmotic swelling. The former involves an increase in the volume of the clay mineral by adsorption of water at its surface. The latter involves the addition of water into the interlayer space of the clay mineral in response to chemical conditions and a consequent increase in volume. Taylor (1988) maintained that the disintegration of British Coal Measures mudrocks (which form a significant part of colliery waste) takes place as a consequence of air breakage after a sufficient number of cycles of wetting and drying.

In other words, if mudrocks undergo desiccation, then air is drawn into the outer pores and capillaries as high suction pressure develops. Then, on saturation, the entrapped air is pressurised as water is drawn into the rock by capillarity. Such slaking, therefore, causes the fabric of the rock to be stressed. The size of the pores is more important than the volume of the pores, for the development of capillary pressure. Capillary pressure is proportional to surface tension. Previously, however, Badger *et al.* (1956) had contended that air breakage occurred only in Coal Measures shales that were mechanically weak, whilst the presence of dispersed colloidal material appeared to be a general cause of disintegration. They also found that the variation in the disintegration of different shales in water was usually not connected with the total amount of clay colloid or the variation in the types of clay mineral present. Rather, it was controlled by the type of exchangeable cations attached to the clay particles and the accessibility of the latter to attack by water, which, in turn, depended on the porosity of the shale. Air breakage could assist this process by presenting new surfaces of shale to water. Subsequently, Dick and Shakoor (1992) maintained that the influence of clay minerals on slaking diminishes in those mudrocks that contain less than 50% of clay-sized particles and that micro-fractures are the dominant lithological characteristic controlling durability. In such cases, slaking is initiated along micro-fractures. Moon and Beattie (1995), in a study of Coal Measures mudrocks from Waikato, New Zealand, found that the proportion of clay mineral present in the mudrock had the greatest influence on durability. Further details of mudrocks are given in Chapter 36 *Mudrocks, clays and pyrite*.

Intra-particle swelling was considered by Taylor and Spears (1970) to have a major effect on the durability of mudrocks from Coal Measures in Britain that contained significant amounts of expandable mixed-layer clay minerals. Moreover, they suggested that those mudrocks that exhibited the greater breakdown were found to contain a higher exchangeable Na ion content in the mixed-layer clay. In fact, the presence of expandable mixed-layer clay is thought to be one of the factors promoting the breakdown of argillaceous rocks, particularly if Na^+ is a prominent interlayer cation. Present evidence nevertheless suggests that in England, colliery spoils with high exchangeable Na^+ levels are uncommon. Sodium may be replaced by Ca^{2+} and Mg^{2+} cations originating from sulfates and carbonates in the waste but dilution by more inert rock types is possibly the most usual reason for the generally low levels of sodium. Moreover, after going through the preparation process and after exposure as a new surface layer of a spoil heap prior to burial, the disintegration of the mudrock and shale material will have been largely achieved. However, it appears that the low shear strength of certain fine-grained English spoils is associated with comparatively high exchangeable Na^+ levels.

34.4.3.3 Spontaneous combustion

Spontaneous combustion of carbonaceous material, frequently aggravated by the oxidation of pyrite, is the most common cause of burning spoil. It can be regarded as an atmospheric oxidation (exothermic) process in which self-heating occurs. Coal and carbonaceous materials may be oxidised in the presence of air at ordinary temperatures, below their ignition point. Generally, lower rank coals (that is, coals with a lower carbon content) are more reactive and accordingly more susceptible to self-heating than coals of higher rank. Spoil heaps that have been subject to spontaneous combustion are made up of a mixture of burnt, partially burnt and unburned material. Therefore, the characteristics of the spoil are more variable.

The oxidation of pyrite at ambient temperature in moist air leads to the formation of ferric and ferrous sulfate, and sulfuric acid (see Chapter 37 *Sulfate acid soils*). This reaction also is exothermic. When present in sufficient amounts, and especially when finely divided, pyrite associated with 'coaly' material increases the likelihood of spontaneous combustion (Michalski *et al.*, 1990). When heated, the oxidation of pyrite and organic sulfur in the coal generates sulfur dioxide. If there is not enough air for complete oxidation, then hydrogen sulfide is formed, which is a highly toxic gas.

The moisture content and grading of spoil are also important factors in spontaneous combustion. At relatively low temperatures an increase in free moisture increases the rate of spontaneous heating. Oxidation generally takes place very slowly at ambient temperatures but as the temperature rises, oxidation increases rapidly. In material of large size the movement of air can cause heat to be dissipated whilst in fine material the air remains stagnant, which means that burning ceases when the oxygen is consumed. Accordingly, ideal conditions for spontaneous combustion exist when the grading is intermediate between these two extremes and hot spots may develop under such conditions. These hot spots may have temperatures around 600 °C or occasionally up to 900 °C (Bell, 1996). Furthermore, the rate of oxidation generally increases as the specific surface of the particles increases.

Spontaneous combustion may give rise to subsurface cavities in spoil heaps, the roofs of which may be incapable of supporting a person. Burnt ashes may also cover zones that are red hot to appreciable depths. Thomson (1973) recommended that during restoration of a spoil heap, a probe or crane with a drop-weight could be used to prove areas of doubtful safety that are burning or underlain by cavities. Badly fissured areas should be avoided and workers should wear lifelines if they walk over areas not proved safe. Any area that is suspected of having cavities should be excavated by a dragline or drag scraper rather than allowing plant to move over suspect ground. An example of an exposed cavity in an Indian spoil tip is shown in **Figure 34.8**.

When steam comes in contact with red-hot carbonaceous material, water gas (carbon monoxide and hydrogen) is formed, and when the latter is mixed with air, over a wide range of concentrations, it is potentially explosive. If a cloud of coal dust forms near burning spoil when reworking a heap, then it can ignite and explode. Damping with a spray may prove useful in the latter case.

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Noxious gases are emitted from burning spoil. These include carbon monoxide, carbon dioxide, sulfur dioxide and, less frequently, hydrogen sulfide. Each may be dangerous if inhaled at sufficiently high concentrations, which may be present at fires on spoil heaps (**Table 34.4**). The rate of production of these gases may be accelerated by disturbing burning spoil through excavating or reshaping. Carbon monoxide is the most dangerous gas since it cannot be detected by taste, smell or irritation and may be present in potentially lethal concentrations. By contrast, sulfur gases are readily detectable and are usually not present in high concentrations. Even so, when diluted they may still cause distress to someone with a respiratory ailment. Nonetheless, sulfur gases are mainly a nuisance rather than a threat to life. In certain situations a gas monitoring programme may be required. Where danger areas are identified, personnel should wear breathing apparatus.



Figure 34.8 Cavity exposed in a burning coal spoil tip in India
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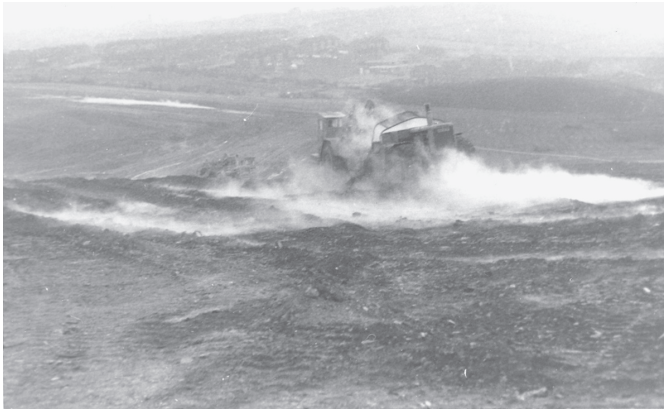
Burning spoil material may represent a notable problem when reclaiming old tips (**Figure 34.9**). Spontaneous combustion of coal in colliery spoil can be averted if the coal occurs in an oxygen-deficient atmosphere that is humid enough with excess moisture to dissipate any heating that develops. Cook (1990), for example, described shrouding a burning spoil heap with a cover of compacted discard to smother the existing burning material and prevent further spontaneous combustion. Thomson (1973) also recommended blanketing and compaction, as well as digging out, trenching, injection with non-combustible material and water, and water spraying as methods

Gas	Concentration by volume in air (parts per million)	Effect
Carbon monoxide	100	Threshold Limit Value (TLV) under which it is believed nearly all workers may be repeatedly exposed day after day without adverse effect
	200	Headache after about 7 hours if resting or after 2 hours if working
	400	Headache and discomfort, with possibility of collapse, after 2 hours at rest or 45 minutes exertion
	1 200	Palpitation after 30 minutes at rest or 10 minutes exertion
	2 000	Unconsciousness after 30 minutes at rest or 10 minutes exertion
Carbon dioxide	5 000	TLV. Lung ventilation slightly increased
	50 000	Breathing is laboured
	90 000	Depression of breathing commences
Hydrogen sulfide	10	TLV
	100	Irritation to eyes and throat: headache
	200	Maximum concentration tolerable for 1 hour
	1 000	Immediate unconsciousness
Sulfur dioxide	1–5	Can be detected by taste at the lower level and by smell at the upper level
	5	TLV. Onset of irritation to the nose and throat
	20	Irritation to the eyes
	400	Immediately dangerous to life

Notes
1. Some gases have a synergistic effect, that is, they augment the effects of others and cause a lowering of the concentration at which the symptoms shown in the above table occur. Further, a gas which is not, itself, toxic may increase the toxicity of a toxic gas, for example, by increasing the rate of respiration; strenuous work will have a similar effect.
2. Of the gases listed, carbon monoxide is the only one likely to prove a danger to life, as it is the commonest. The others become intolerably unpleasant at concentrations far below the danger level.

Table 34.4 Effects of noxious gases
Reproduced from Thomson (1973); National Coal Board

(a)



(b)



Figure 34.9 (a) Burning spoil in an old coal tip, Wharncliffe, near Barnsley; (b) burning coal spoil tip, India
Courtesy of International Mining Consultants Limited

by which spontaneous combustion in spoil material may be controlled. Bell (1996) described the injection of a curtain wall of pulverised fuel ash, extending to the original ground level, as a means of dealing with hot spots.

34.4.3.4 Restoration

The configuration of a spoil heap depends upon the type of equipment used in its construction and the sequence of tipping the waste. The shape, aspect and height of a spoil heap affect the intensity of exposure, the amount of surface erosion that occurs, the moisture content in its surface layers and its stability. Although the mineralogical composition of coarse discards from different mines varies, pyrite frequently occurs in shales and coaly material is present in spoil heaps. When pyrite weathers it promotes acidic conditions in the weathered material. Such conditions do not aid the growth of vegetation. Indeed, some spoils may contain elements that are toxic to plant life. Furthermore, the uppermost slopes of a spoil heap are frequently devoid of near-surface moisture. To support vegetation a spoil heap should have a stable surface in which roots can become established, must be non-toxic and contain an adequate and available supply of nutrients. Hence, spoil heaps are often barren of vegetation.

The restoration of a spoil heap will require large-scale earthmoving. The restored land may be used for agriculture, for public amenities such as golf courses, or for buildings. Since restoration invariably involves spreading the waste over a larger area, this may mean that additional land beyond the site boundary has to be purchased. Where a spoil heap is very close to a disused colliery, spoil may be spread over the latter area. This involves the burial or removal of derelict colliery buildings and sometimes old mine shafts or shallow old workings may have to be treated (Johnson and James, 1990). Watercourses may have to be diverted, as may services, notably roads.

The landscaping of spoil heaps is frequently carried out to allow them to be used for agriculture or forestry. This type of restoration is generally less critical than when structures are to be erected on the site, since bearing capacities are not as important and steeper surface gradients are acceptable. Most spoil heaps offer no special handling problems other than the cost of regrading and possibly the provision of adjacent land, so that the gradients on the existing site can be reduced by the transfer of spoil to the adjacent land. However, as noted, some spoil heaps present problems due to spontaneous combustion.

The surface treatments of spoil heaps vary according to the chemical and physical nature of the spoil, and the climatic conditions. The preparation of the surface of a spoil heap also depends upon the use to which it is to be put subsequently. Where it is intended to sow grass or plant trees, the surface layer should not be compacted. Drainage plays an important part in the restoration of a spoil heap and during landscaping there should be erosion control. If the spoil is acidic, then it can be neutralised by liming. The chemical composition influences the choice of fertiliser used and, in some instances, spoil can be seeded without the addition of top soil, if suitably fertilised. More commonly top soil is added prior to seeding or planting.

34.4.3.5 Acid mine drainage

Acid mine drainage (AMD) or acid rock drainage (ARD) may be associated with a coarse colliery discard since the coaly material and mudrock are likely to contain sulfide minerals, notably pyrite (Geldenhuis and Bell, 1998). Such drainage results from the natural oxidation of sulfide minerals in the waste when it is exposed to air and water. It is a consequence of the oxidation of sulfur in the mineral to a higher state of oxidation and, if aqueous iron is present and unstable, the precipitation of ferric iron with hydroxide occurs. Acid mine drainage can also be associated with opencast workings as well as underground workings, tailings ponds or mineral stockpiles (Brodie *et al.*, 1989). If acid mine drainage is not controlled it can pose a threat to the environment since acid generation can lead to elevated levels of heavy metals and sulfate in surface water or groundwater, which will obviously have a detrimental effect on water quality (see Coal Authority website: http://coal.decc.gov.uk/en/coal/cms/environment/about_m_water/about_m_water.aspx). Acid generation tends to occur in the surface layers of spoil heaps where air and water come into contact with sulfide minerals. Methods for remediating AMD have been reviewed by Johnson and Hallberg (2005). Broadly, there are three main types. Active

methods require the pumping of the mine water and treatment with chemical reagents such as hydrogen peroxide or caustic soda. Passive methods use natural systems such as reed beds to improve water quality and do not use pumping; the water from the mine is allowed to flow under gravity. Hybrid active and passive schemes may utilise pumping and some chemical treatment at the start of the process before water flows into reed bed systems (Coal Authority website: http://coal.decc.gov.uk/en/coal/cms/environment/treat_types/treat_types.aspx). Around 50 abandoned coal mines have been treated in Britain, mostly using passive systems. For the Wheal Jane tin mine in Cornwall, Whitehead and Prior (2005) described a passive treatment system developed as part of a research project. Three separate systems were installed, each with aerobic reed beds, anaerobic cells and rock filters. The three systems differed in the method used to raise the pH of the mine water entering them. The Coal Authority website (see above) briefly describes the treatment approaches used at a number of former coal mining sites.

34.4.4 Other types of fill

34.4.4.1 Pulverised fuel ash (PFA)

Pulverised fuel ash (PFA) is waste that is produced by coal-fired power stations and is defined as the solid material extracted by electrostatic or mechanical means from the furnace flue gases (Winter and Clarke, 2002). Charles and Watts (2001) pointed out that most of the PFA is metal oxides along with some trace constituents and pozzolanic materials. About 80% of the ash from a coal-fired power station is PFA and the remaining 20% is furnace bottom ash. After extraction, the PFA is referred to as fresh PFA and can be conditioned with an optimum amount of water to produce conditioned ash. Conditioning PFA allows it to be transported more easily and reduces the problem of dust. When PFA is stored as conditioned ash it is called stockpiled ash.

When PFA is disposed of as waste, it is usually mixed with water so that it can be transported as slurry to a lagoon, for example. It can be disposed of in this manner in old clay pits. This ash can be subsequently retrieved as stockpiled ash. The water in the lagoon can be decanted when the ash has settled out of the slurry. Developments on PFA lagoons are rare but some are mentioned by Charles and Watts (2001). One is the Metro Centre at Gateshead and another is housing in Peterborough. They also referred to an oil tank being constructed on a 5 m thickness of lagoon PFA in Denmark.

According to Yang *et al.* (1993), the properties of fresh pulverised fuel ash are governed by the type and chemical composition of the coal used, the source of the coal, the degree of pulverisation, the efficiency of combustion, the variation in firing conditions and the extraction process. Consequently, the character of PFA does vary. In fact, it is not only variable between sources but also on a day-to-day basis from a single source. Nonetheless, the particles of PFA are primarily of coarse silt to fine-sand size and are more or less spherical in shape. Their specific gravity ranges from around 1.90 to 2.72, depending on their source and they are non-plastic. Ash may exhibit 'cohesive' properties although Yang *et al.* (1993) noted that such properties might be attributable to the suction forces that develop during compaction of the ash. Hence, they may dissipate with time or when inundated, so leading to loss of strength. Some ashes are self-hardening because of the presence of cementitious material such as free lime or calcium sulfate compounds. In addition to Yang *et al.* (1993), reviews of the engineering properties of PFA can be found in Sutherland *et al.* (1968), Gray and Kin (1972) and Leonards and Bailey (1982). PFA has a number of other civil engineering applications, for example as a cement replacement or additive.

Most PFA is used for land-reclamation projects or for general and structural fills (for example, embankments, foundation fills or fills behind retaining walls). Pulverised fuel ash has frequently been used for the construction of road embankments. For instance, Ekins *et al.* (1993) mentioned the use of lightweight PFA in the construction of the Western Approach Road, Southampton. Pulverised fuel ash is also used as fill behind the abutments of road bridges. The use of PFA as a structural fill for the foundations of a precipitator at a power generating station in Indianapolis, Indiana, was described by Leonards and Bailey (1982). In addition, a notable quantity of PFA is used in the manufacture of cement as some types of PFA are pozzolanic. The pozzolanic activity is attributable to the reaction of the fine glass (mullite) particles with free lime in the presence of water.

Conditioned PFA can be used as high-quality selected fill, either as fill to structures and reinforced earth or for stabilisation with cement to form capping. Enough water must be added to the PFA so that it is suitable for compaction. Winter and Clarke (2002) indicated that the optimum moisture content of conditioned ash is around 25% but that it may be as high as 35% or more. They further noted that the optimum moisture content of some conditioned ash from Scotland may be as low as 18% whereas that of lagoon ash may be as high as 28%. They quoted maximum dry densities varying from 1.0 to 1.65 Mg/m^3.

34.4.4.2 Industrial waste

There are many types of industrial waste, such as chemical wastes and wastes from ore smelting. Not only may industrial fills be very heterogeneous and poorly compacted but they may be chemically problematic in that the ground may be aggressive. Industrial wastes can take a number of forms such as smelter slag, various types of residues and sludge. Furthermore, industrial wastes are commonly associated with derelict sites. Such sites represent a wasted resource and are normally located within urban areas so that their rehabilitation is highly desirable. Mabey (1991) reviewed the position of derelict land in Britain, noting that in 1988 there were some 40 500 ha of derelict land. Of the land attributable to industrial dereliction, 94% was considered to need remediation. More recently, there has been great emphasis on the reclamation of derelict land but national statistics seem

to show that there may be nearly 80 000 ha in this condition in Britain. According to the National Land Use Database, the amount of derelict land in England has remained fairly constant in recent years falling from about 66 000 ha in 2002 to nearly 62 000 ha in 2009. In Scotland, there were just over 8 000 ha of derelict land in 2010 (Anon., 2011).

Unfortunately, many industrial wastes have been contaminated to a greater or lesser extent (Bell *et al.*, 2000). Contaminated land may also emit gases or may be a fire hazard. Details of such hazards should be determined during a site investigation (British Standards Institution, 2011). Site hazards result in constraints on the freedom of action, necessitate stringent safety requirements, may involve time-consuming and costly working procedures, and affect the type of development. For instance, Leach and Goodyear (1991) noted that constraints might mean that the development plan has to be changed so that the more sensitive land uses are located in areas of reduced hazard. Alternatively, where notable hazards exist, then a change to a less sensitive end use may be advisable. There are a number of general references that provide further information on how to deal with and remediate hazards from contaminated land; for example, Nathanail and Bardos (2004) and Bardos *et al.* (2007). Bunce and Braithwaite (2001) described the remediation of an 80 ha site at Pride Park, Derby. The land had been used for gravel extraction, waste disposal, railway works and a gas and coke plant. The site was heavily contaminated and for the worst areas a bentonite containment wall was constructed to prevent contaminant migration. Water collecting within the wall was pumped out using 18 abstraction wells, treated and finally discharged into the River Derwent. About 75% of the site was usable as a result of the remedial work.

Some industrial wastes, notably those that are non-contaminated, can be used as bulk fill for land-reclamation schemes. Indeed, derelict sites often require varying amounts of filling, levelling and regrading. Fill should be obtained, if possible, from the site. Furthermore, some industrial wastes can be used as raw materials; for example, blast furnace slag can be used for road aggregate.

34.4.4.3 Building waste

Building waste is associated with demolition work and mainly includes concrete and brick rubble. In addition, timber, steel reinforcement, glass, plaster, stone, slate and tiles are likely to be included in building waste to a greater or lesser extent so that it is essentially heterogeneous in composition. Some of these materials are unsuitable for use as fill. For example, Reid and Buchanan (1987) mentioned that the derelict Queen's Dock in Glasgow had been filled with 1.3 million m^3 of demolition rubble from old buildings in the vicinity, degradable material such as timber having been removed. Similarly, Hartley (1991) referred to the use of old foundation material that was crushed and used as fill at a site in Middlesbrough. Building wastes are usually poorly compacted and may contain gypsum products. The latter could be a source of sulfate, which could attack concrete. Again building wastes like industrial waste may be contaminated and, therefore, appropriate assessment will be needed (see discussion above). However, it is increasingly the case that building waste following demolition is separated, crushed and graded (where appropriate) and recycled.

34.5 Conclusions

The reclamation of derelict land is an important aspect of government policy. More recently there has been an emphasis on siting new building developments on brownfield rather than greenfield sites and many of these brownfield sites are covered in substantial depths of fill. Thus, the subject of building on fill has acquired considerable prominence in recent years. The geotechnical problems in achieving safe and economic developments on filled ground are substantial.

Therefore, it is important to understand both the distribution of different types of fill and their likely behaviour. It has been shown that the engineering behaviour of a fill is strongly influenced by its composition, method of deposition and subsequent stress history. These factors control the key aspects in successfully reusing areas underlain by non-engineered fill. It is important, therefore, to assess the variability of a fill (in terms of composition and physical, mechanical and chemical properties), and to assess how the behaviour of a fill may change over time. In particular, a fill may be subject to significant volume change on loading. This can be caused by a number of factors including the original loading as the fill was deposited (often over wide areas), changes in water content influenced by variable permeability, swelling and shrinkage of clay fills, collapse compression, biodegradation and chemical reactions.

Past (largely) industrial land use indicates the likely nature of non-engineered fills encountered. The principal types of fill include filled opencast sites (widely distributed in coalfield areas), domestic waste and sanitary landfill (widely distributed in former quarries and other excavations in urban, peri-urban and rural areas), colliery discard (found close to former coal mines), pulverised fuel ash (both as a waste and used for land reclamation), industrial wastes (which can be both physically and chemically variable) and building wastes (often found in central urban areas).

Chapter 54 *Single piles* describes the requirements for building on fills in further detail, with reference to the investigation and prediction of behaviour.

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34.7.2 Useful websites

British Geological Survey (BGS); www.bgs.ac.uk
BRE Group (formerly Building Research Establishment); www.bre.co.uk
Coal Authority; www.coal.decc.gov.uk
Construction Industry Research and Information Association (CIRIA); www.ciria.org
Environment Agency; www.environment-agency.gov.uk
National Land Use Database; www.homesandcommunities.co.uk/ourwork/national-land-use-database
Northern Ireland Environment Agency; www.ni-environment.gov.uk
Ordnance Survey; www.ordnancesurvey.co.uk
Scottish Environment Protection Agency; www.sepa.org.uk
Transport Research Laboratory (TRL); www.trl.co.uk

It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 58 *Building on fills*
- Chapter 69 *Earthworks design principles*
- Chapter 75 *Earthworks material specification, compaction and control*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 35
Organics/peat soils

Eric R. Farrell AGL Consulting, Dublin, Ireland, and Department of Civil, Structural and Environmental Engineering, Trinity College, Dublin, Republic of Ireland

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Extensive deposits of peat and organic soils can be found in Great Britain, Ireland and worldwide. Their engineering behaviour depends on their morphology, which must be reflected in the description and classification of soil and in turn must be included in the interpretation of geotechnical parameters. Water content is the most important index test and can be used to estimate engineering properties of virgin peat. Other indices are loss-on-ignition and the degree of decomposition scale. Secondary compression dominates when estimating settlements of peat; generally this occurs during the primary consolidation (hydrodynamic) phase and after dissipation of excess pore pressure. Sophisticated soil models reflect this continuous secondary compression, although these are not readily used for routine design. Surcharging of peat is effective in reducing long-term creep movements under embankments. Permeability of peat decreases with compression and affects consolidation behaviour.

Determining effective stress parameters of fibrous peats depends on the type of test and the orientation of the sample. Fibres affect *in situ* test methods to determine undrained shear strength, particularly the *in situ* vane which gives s_u values that depend on the size of the vane used. Laboratory tests also recorded relatively high s_u/σ_v ratios.

Design issues include design and construction approaches for roads, structures, slopes, canals and dykes founded in or with peats and organic soils.

35.1 Introduction

Peats and organic soils would generally be considered 'problematic' in a geotechnical context because of their high compressibility, creep behaviour, low bulk density and generally low undrained shear strength. The high compressibility and creep behaviour have given rise to large settlements under structures and to undulating roads that become a hazard to traffic. Regionally, for example, changes in the hydrogeological regime have resulted in settlements of several metres and large bogslides have wiped out villages in Ireland with many fatalities. Historically, dykes and canals have been constructed on or with peat, which leaves a legacy of high-risk structures within our society. The engineering behaviour of peat and organic soils is therefore an important aspect of geotechnical engineering, both in the design of structures on peat and organic soils and in the assessment of existing structures on these soils.

The percentage of land covered by peat is about 33.5% in Finland, 18% in Canada, about 17% in the Republic of Ireland and Sweden, 13.7% in Indonesia, 12.4% in Northern Ireland, 10.4% in Scotland and 6.3% across the United Kingdom as a whole (Hobbs, 1986). Large tracts of peatlands are also to be found in Russia and the USA. A variety of terms in the English language are used to describe the areas within which peats and organic soils are formed, such as swamp, boglands, moor, fen and muskeg. However, 'mire' is the internationally accepted term to cover all of these ecosystems (Gore, 1983). This chapter mainly discusses the particular properties of peats; however, there is obviously a transition in the behaviour from highly organic soils, which would have many similar properties to peats, to soils with low organic content, which would have similar behaviour to mineral soils.

35.2 Nature of peats and organic soils

Peat and organic soils include a wide range of soils with different morphologies and characteristics that impact on their engineering behaviour. The organic content can come from plants or animals, but most of this chapter will relate to the organics from plant debris. Peat itself is formed from the accumulation of the remains of dead vegetation in various stages of decomposition, and this accumulation can build up where there is a reservoir of water to promote growth and to preserve the remains. However, there are a variety of conditions where this can occur. For example, raised bogs obtain their nutrients from rainwater, which results in a particular type of vegetation growth, and are made up almost entirely of organic material, whereas other bogs are fed from surface and groundwater flow that inevitably includes some inorganic particles, which are generally referred to as mineral particles in this context.

The remains of plants that make up peats and organic soils have an open cellular structure which gives rise to their high compressibility and high moisture contents. The open structure of a peat fibre can be appreciated from the electron microscope image of a typical stem shown in **Figure 35.1**. The cell structure can thus deform when transferring inter-particle forces. Water is held in three forms: (i) free water in large cavities; (ii) capillary water in the narrower cavities; (iii) water bound physically, colloidally and osmotically (MacFarland and Radforth, 1964). There is also the reinforcing effect of the fibres, which can significantly affect the ability of peats to resist load. These



Figure 35.1 Electron microscope image of Balltdermot peat

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differences from clays and sands should, in theory, make many of the traditional methods of analysis inapplicable. However, many of the correlations between various properties and behavioural parameters are little different from those of clay soils except in regard to its strength (Hobbs, 1986).

The decomposition of the plant remains can have a significant effect on the characteristics of a peat, and this is normally termed the degree of humification in geotechnical engineering. Plant remains are broken down by soil microflora, bacteria and by fungi and in non-acidic soils earthworms can contribute to the breakdown (Hobbs, 1986). The initial process is aerobic where the plant matter is broken down to give off gas and water. Immersion in water dramatically reduces the availability of oxygen, thus reducing not only the aerobic microflora but encouraging the anaerobic species with different and less rapid metabolic activity. The partially decayed vegetation matter can thus accumulate as peat. The upper active layer with undecayed fibrous plants is called the acrotelm (generally100–600 mm thick), with the lower layer being called the catotelm. Initially the decomposition of the cellulose structure affects the plant leaves but this process can develop to the stems and roots of the plants, and eventually to completed decomposition into an amorphous-granular material consisting of mainly gelatinous organic acids, which have a sponge-like structure (Landva and Pheeney, 1980). However, the complete breakdown of the plant remains is rare in the UK and Ireland. The rate of decay generally increases with an increase in temperature and with pH. This process of plant degradation is not uniform and therefore can exacerbate the natural non-uniformity of peatlands. The nature of peat can also be affected by historic fires, anthropogenic effects, previous landslides and other external factors.

Mires can be broadly separated into fen and bog, with bog being sub-divided into raised bog (over fen bog) and blanket bog. Raised bogs are formed over what was originally a fen bog and include a transitional stage, which must be considered when making an engineering assessment. The separation between fen and bogs lies in the origin and chemistry of their water supplies. Moore and Bellamy (1974) and Hobbs (1986) illustrate the different formation environments in which mires are formed.

Fens are predominantly fed by groundwater or flowing water sources, and generally are neutral to alkaline, although there are also acidic fens. The alkaline environment increases the rate of decay and hence can result in a greater degree of humification than in raised bogs. Fens form in a variety of wetlands, from water-filled depressions to river banks and lake shores, and can support a variety of vegetation growth, depending on their water sources and genesis. Floating bogs can form by vegetation growth from the side of a lake, leaving clear water between the base of the peat and the lake bottom.

Bogs are formed in ombrotropic (rain-fed) conditions and are acidic. Raised bogs start off by organic growth on or from the banks of lakes, i.e. fen peat, which expands to encompass the lake, at which stage the landscape would commonly be referred to as a fen (Hobbs, 1986). These fen peats would frequently be formed over the original very soft lake-bed deposits, where these exist, which can be the critical feature in the selection of appropriate design/construction options on these deposits. Where the conditions are appropriate, there is a transitional stage where the upward growth is such that the plants come to rely more on rainfall for nutrients rather than on the natural groundwater and eventually a mounded mass of organic material is formed, which obtains its entire nutrient from rainwater; at this stage it is called a raised bog. Sphagnum moss and cotton grass tussocks are common forms of vegetation.

Blanket bogs are a form of raised bog with rainwater as the principal source of nutrients but which form directly on the original soil rather than on a lake. They typically form on the side of mountains in the colder and temperate regions of the Earth where there is an excess of precipitation over evaporation. Given that these are generally on sloping ground, the nature of the original ground on which they are formed and the hydrogeological conditions can be important considerations when assessing the slope stability of the mass, together with the characteristics of the peat itself. Features of some of these blanket bogs are the presence of subterranean pipes or channels which have developed over time and also, in some locations, the presence of a hardpan layer in mineral soils close to the base of the peat, which was formed by the accumulation of leached minerals from the peat. Donnelly (2008) documents the presence of soil pipes of up to 1.5 m in diameter on parts of the Pennine moors and of pseudo-sinkholes caused by their collapse.

Layers of marl can be associated with peat deposits in calcareous regions. These layers form due to photo-reduction of dissolved carbon dioxide by submerged green plants.

Peats have also been deposited in estuarine and coastal environments and are sometimes buried under mineral soils

due to change in the depositional conditions. For example, Kidson and Heyworth (1976) have shown that large areas of Somerset, known as the Levels, are underlain by considerable thickness of Quaternary sands, gravels, clays and peats which were formed within the contemporary tidal-range, the sedimentation being influenced by rise in sea level over the last 9000 years.

35.3 Characterisation of peats and organic soils

35.3.1 Classification systems

There are a number of classification systems for peat deposits in existence, most of which have a botanical basis suitable for agriculture and ecology but these are generally too elaborate for engineering applications. An example of a comprehensive classification system is given in Clymo (1983). Engineering classification systems have been proposed by von Post (1922), Radforth (1969) in Canada, Landva *et al*. (1983), Hobbs (1986, 1987), and by others. The von Post system, which was developed for horticultural, agricultural and forestry use, includes a simple 'squeeze' test as a means of assessing the degree of decomposition of peats, which has been incorporated into many of the other classification systems, albeit with some modification.

The important characteristics to be identified in any engineering classification system are the type of peat, the structure, the degree of decomposition and the amount of mineral particles present. The *in situ* colour description can be useful; however, a characteristic of organic soils is the rapid change in colour on exposure to air due to oxidation which can make colour descriptions from recovered samples problematical.

The Eurocode system references two standards which cover the identification and classification of soils, namely EN ISO 14688-1:2002 (*Identification and Description*) and EN ISO 14688-2:2004 (*Principles of Classification*). EN ISO 14688-1:2002 defines organic matter as consisting of plant and/or animal organic materials, and the conversion of products of those materials, e.g. humus (see **Table 35.1**). BS 5930:1999 (BSI, 1999) has description categories of fibrous, pseudo-fibrous and amorphous peat. Gygttja is a particular Scandinavian term which is applied to organic bottom deposits in lakes or the sea. The degree of decomposition of a peat in EN ISO 14688–1:2002 is assessed on the basis of a simple squeeze test (see **Table 35.2**), which is a simplification of the von Post system where the degree of decomposition (H_n) is measured on a scale from one to ten, a modification of which has been incorporated into the ASTM standards (see **Table 35.3**). The von Post system is commonly used where a detailed description of peats is required.

Term	Description
Fibrous peat	Fibrous structure, easily recognisable plant structure, retains some strength
Pseudo-fibrous peat	Recognisable plant structure, no strength of apparent plant material
Amorphous peat	No visible plant structure, mushy consistency
Gyttja	Decomposed plant and animal remains; may contain inorganic constituents
Humus	Plant remains, living organisms and their excretions, together with inorganic constituents, form the topsoil

Table 35.1 Identification and description of organic soil
Data taken from EN ISO 14688-1:2002

Term	Decomposition	Remains	Squeeze
Fibrous	None	Clearly recognisable	Only water No solids
Pseudo-fibrous	Moderate	Recognisable	Turbid water <50% solids
Amorphous	Full	Not recognisable	Paste >50% solids

Table 35.2 Degree of decomposition of wet peat as determined by squeezing
Data taken from EN ISO 14688-1:2002

Degree of humification	Nature of material extruded on squeezing	Nature of plant structure in residue
H1	Clear, colourless water; no organic solids squeezed out	Unaltered, fibrous, undecomposed
H2	Yellowish water; no organic solids squeezed out	Almost unaltered, fibrous
H3	Brown, turbid water; no organic solids squeezed out	Easily identifiable
H4	Dark brown, turbid water; no organic solids squeezed out	Visibly altered but identifiable
H5	Turbid water and some organic solids squeezed out	Recognisable but vague, difficult to identify
H6	Turbid water; ½ of sample squeezed out	Indistinct, pasty
H7	Very turbid water, ½ of sample squeezed out	Faintly recognisable; few remains identifiable, mostly amorphous
H8	Thick and pasty; 2/3 of sample squeezed out	Very indistinct
H9	No free water; nearly all of sample squeezed out	No identifiable remains
H10	No free water; all of sample squeezed out	Completely amorphous

Table 35.3 Degree of humification or decomposition, ASTM D5715-00
Based on von Post (1922)

Soil	Organic content (≤2mm)% dry mass
Low organic	2 to 6
Medium organic	6 to 20
High organic	>20

Table 35.4 Classification of soils with organic constituents
Data taken from EN ISO 14688-2:2004

EN ISO 14688-2:2004 (*Principles of Classification*) classifies a soil with more than 20% organics as high-organic soil (see **Table 35.4**); however, there is no particular definition of when an organic soil becomes a peat or when a clayey peat becomes a peaty clay or how the organic content is determined. BS 5930:1999 (BSI, 1999) gives a classification system for soils where the organic content is the secondary constituent but does not classify a peat. ASTM classifies a peat as having less than 25% ash content, which is equivalent to an organic content in excess of 75%.

35.3.2 Index tests

The water content (w) is the most important property when dealing with peat and organic soils, as it is a simple test to which many of the engineering properties can be related. Consequently, w can be used to quantify variations in the peat and organic soil deposits. There is evidence that there may be charring of certain types of peat when the water content is determined at the normal 105–110°C and MacFarlane and Allen (1964) suggest that a temperature of 85°C may be appropriate if combustion of material or oxidation of the peat is to be avoided. However, Hobbs (1986) considers that the normal 105°C is acceptable for engineering purposes and this would be consistent with the findings of O'Kelly (2005) and with general practice.

The organic content is a very useful index test and is normally taken as being equal to the loss-on-ignition (N). This index test quantifies the amount of organics present and also has the very useful relationship between the average specific gravity of the particles (G_{Save}) given as Equation (35.1) (Skempton and Petley, 1970). This equation is based on the specific gravity of the cellulose being about 1.4 and of the mineral particles being about 2.7. This specific gravity can be used to estimate the bulk density using Equation (35.2); however, it should be noted that the degree of saturation S_r can be less than 100% in some peats due to its gas content.

$$\frac{1}{G_{s_{ave}}} = \frac{N}{1.4} + \frac{(1-N)}{2.7} \tag{35.1}$$

$$\rho = \frac{G_{save}\rho_w(1+w)}{1+\frac{wG_{save}}{S_r}} \tag{35.2}$$

The loss-on-ignition (N) is determined on a specimen initially dried at 50°C and fired in a muffle furnace at 440°C ± 40°C as recommended in BS 1377:Pt3:1990 (BSI, 1990). Clay minerals present lose fixed water at temperatures in excess of 450°C causing an error in the ignition loss. The ASTM uses the ash content (D), rather than the organic content, where D = 1 – N, and includes two tests for its determination, one for a firing temperature of 440°C (initially dried at 105°C) until it is completely ashed and the other for 750°C. Some researchers have found that fresh plant roots are not removed at temperatures below 900°C. Hobbs (1987) summarises the correction factors to be applied to the loss-on-ignition value determined at temperatures above 450°C to account for the water loss from minerals. BS 1377:Pt.3:1990 (BSI, 1990) also includes a chemical titration test (Walkley and Black method), which is generally used only for soils with low organic content.

The liquid limit (w_L), which can be a useful indicator to the morphology of the peat and its degree of decomposition, can generally be carried out on peats with a degree of decomposition greater than H_3 on the von Post scale (von Post scale given in **Table 35.3**). The structure of the peat/organic soil generally has to be broken down to a suitable size using a mechanical device, thus not only breaking down the structure but also some of the decomposed material (Hobbs, 1986). Hanrahan *et al.* (1967) found the w_L value to be sensitive to the chemistry of the water, therefore water from its natural source should be used. Bog peats (w_l = 800 to 1500%) typically have higher w_L than fen peats (w_L = 200 to 600%). Hobbs (1986) showed, by observing the decomposition of a cauliflower, that both the liquid limit and the water content decreased with the degree of decomposition, and considered that the water content (natural) was equal to about $1.38w_L$ and $0.875w_L$ for bog and fen peat, respectively. The plastic limit can only be carried out on peats with some clay particles, and consequently is generally of little interest to bog peats. The addition of organic content to a mineral soil can alter its liquid and plastic limit significantly, not only because it introduces an additional material, but also because the organic particles, which are negatively charged, may be strongly adsorbed on the mineral surfaces, with the effect that the properties of both materials are altered (Mitchell and Soga, 2005). It also means that the classification of an organic soil based on organic content alone would not be expected to capture all the significant geotechnical features of a soil.

Bulk density is also a useful index and there is generally a good agreement between measured bulk densities and those determined from Equation (35.2), using G_{save} estimated from loss on ignition tests. Hobbs (1986) plotted the bulk density measured on five UK mires and showed that the degree of saturation/gas content was the main influence on bulk density when the water content was in excess of about 600%. About half the samples had bulk densities of less than 1 mg/m^3 when the water content was greater than 500%, indicating that the peat would be buoyant in water. For water contents in excess of about 200%, the volume of gas as a percentage of the total

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volume is approximately 1-S_r, where S_r is the degree of saturation. The average gas volume indicated by the results of Hobbs (1986) is about 7.5%.

Although not normally considered an index test, the pH can be a useful indicator of the nature of peats and organic soils (Hobbs, 1986). Metabolic activity is influenced by temperature and acidity: the higher the temperature and pH (lower acidity) the faster the decomposition. There is a considerable degree of overlap, but bog peats have a pH in the range 4.5 to 3.3, and transitional peats from 6 to 4. The pH of a fen peat will depend on the water source; however, most tend to have pH greater than 5, with the acid fens being less than 5.

35.3.3 Sampling methods

It can be difficult to obtain 'undisturbed' samples of highly organic soils, particularly fibrous peats. Simple sampling methods which can be used to get continuous or almost continuous disturbed samples, such as gouge augers, can be used to determine a water content/depth profile and thus to identify important stratigraphic variations. Sharp-edged piston samples have been used successfully; however, the samples generally compress and it is important to record the percentage recovered in order to assess the degree of compression due to sampling. Undisturbed samples can be cut from the *in situ* material; however, this becomes impractical with depth. Special samplers, such as Sherbrooke sampler (Terzaghi *et al*., 1996) which essentially allows a block sample to be cut from the base of a borehole using a system of blades, can be used on special projects. Oxidation generally changes the colour of recovered samples when exposed to the atmosphere and biodegradation can occur within the samples. The colour changes are typically limited to the exposed surfaces; however, microbial processes can lead to gas formation which may contribute to excess pore water pressures (Kellner *et al*., 2005). The effect of such microbial process were investigated by Fox *et al*. (1999) by comparing the long-term settlements of an oedometer sample which had been sterilised using radiation to kill bacteria and fungi with untreated samples. This work suggests that biodegradation may be an issue in long-term creep tests, but additional research is required before this can be confirmed.

35.4 Compressibility of peats and organic soils

35.4.1 Introduction

The large compressions that can occur in virgin peats are clearly illustrated in **Figure 35.2**, which presents the vertical strain plotted against the logarithm of the effective stress, as recorded in a number of case histories (Landva and La Rochelle, 1983; additional cases by O'Loughlin, 2007), although some shear deformation and secondary compression is included in some of these data. These results show that a peat stratum can reduce in thickness by more than 50% under a relatively modest increase in effective stress of the order of 50 kPa. Furthermore, secondary compression can be the dominant process, and not 'a rather tiresome appendage' to the consolidation process (hydrodynamic) as is generally assumed (Hobbs, 1986). The large strains will necessarily distort the structure of peats and organic soils

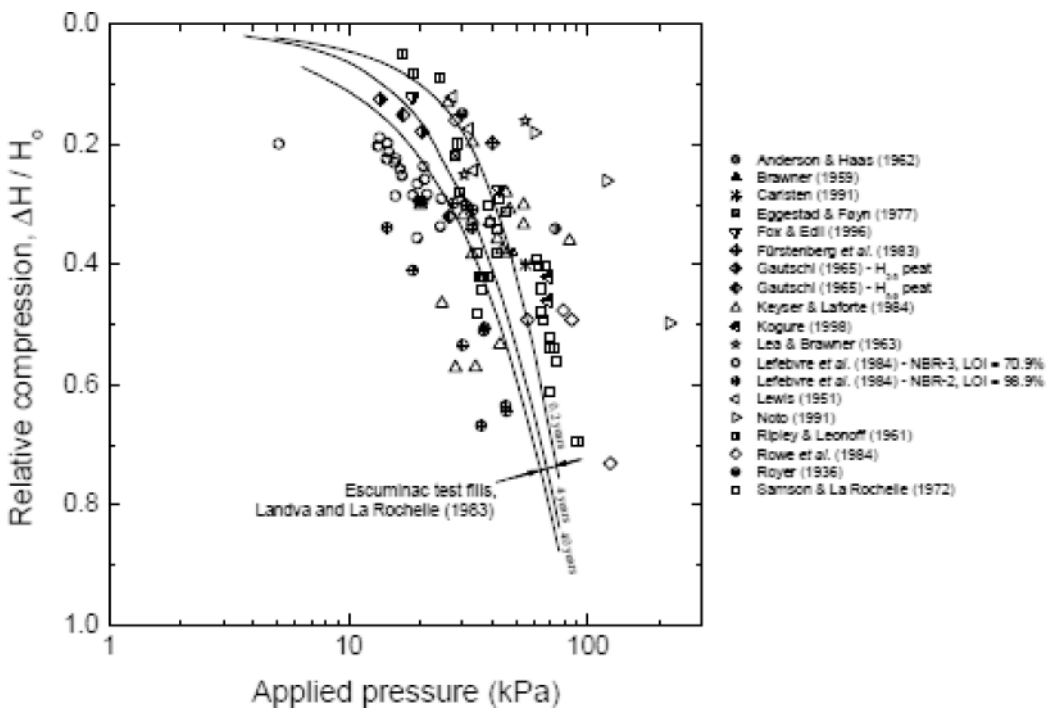


Figure 35.2 Relative compression versus applied pressure (Landva and La Rochelle, 1983, updated by O'Loughlin, 2007)
Reproduced from O'Loughlin (2007)

and hence the permeability and compressibility characteristics change as the soil compresses. Given the genesis of raised bogs with vegetation growth patterns, the properties are not uniform in either the vertical or horizontal direction. There are therefore many complexities in peats compared with mineral soils which must be considered when selecting an appropriate model to estimate settlements.

The terms 'primary' and 'secondary' compression can lead to confusion and to controversy (den Haan, 1996) and many researchers consider that it is better to consider 'primary consolidation' as a hydrodynamic phase in the settlement process that is controlled by the dissipation of excess pore water pressures, and that secondary compression is a continuous process that occurs during both the hydrodynamic phase and continues when the excess pore pressures have essentially dissipated. This introduces some interesting concepts, for example, that secondary compression movements should be estimated from the start of loading rather than from the end of primary consolidation (EOP), as is assumed in the EOP approach, which also affects the interpretation of 'pre-consolidation' or critical pressure.

35.4.2 Magnitude of settlement (constrained deformation)

35.4.2.1 Different philosophies

Secondary compression can be a dominant process in organic soils and there are two fundamentally different schools of thought on the way it should be modelled: one which considers that it begins at the end of primary consolidation (EOP method) and the other which considers that secondary compression occurs both during and after the primary consolidation phase, the primary consolidation being considered as the hydrodynamic phase. The significance of these two approaches for samples of different heights, H_1 and H_2, is illustrated in **Figure 35.3**, where t_p is the time to the completion of primary consolidation. The EOP assumption simplifies the estimation of settlements in that the primary settlements can be estimated in the traditional fashion and the secondary compression included as a separate calculation. The EOP approach would not include the additional settlement that would arise from secondary compression that occurs during the primary phase for a thick layer. The standard 24 hour consolidation test on peats would include some secondary compression, which may reduce the error in practice (Hobbs, 1986).

The EOP approach is frequently used in practice as the calculation models for the continuous approach to date are complex and the additional complexity is generally not justified when other uncertainties are considered, except for very large projects.

35.4.2.2 EOP method

The advantage of the EOP approach is that the primary settlement can be estimated from an e-log σ'_v plot in the normal fashion but necessarily requires some assumption/knowledge of a 'pre-consolidation/yield' pressure (σ'_c). Field and laboratory tests have shown that virgin peats do have an 'apparent' yield point, possibly due to capillary suction forces when in the acrotelm or due to the weight of snow (Hobbs, 1986; Mesri, 2007). This may be estimated from the e-log σ' curve in the normal Casagrande (1936) construction, by using Janbu's method (1963) or from relationships such as that given in Mesri (2007).

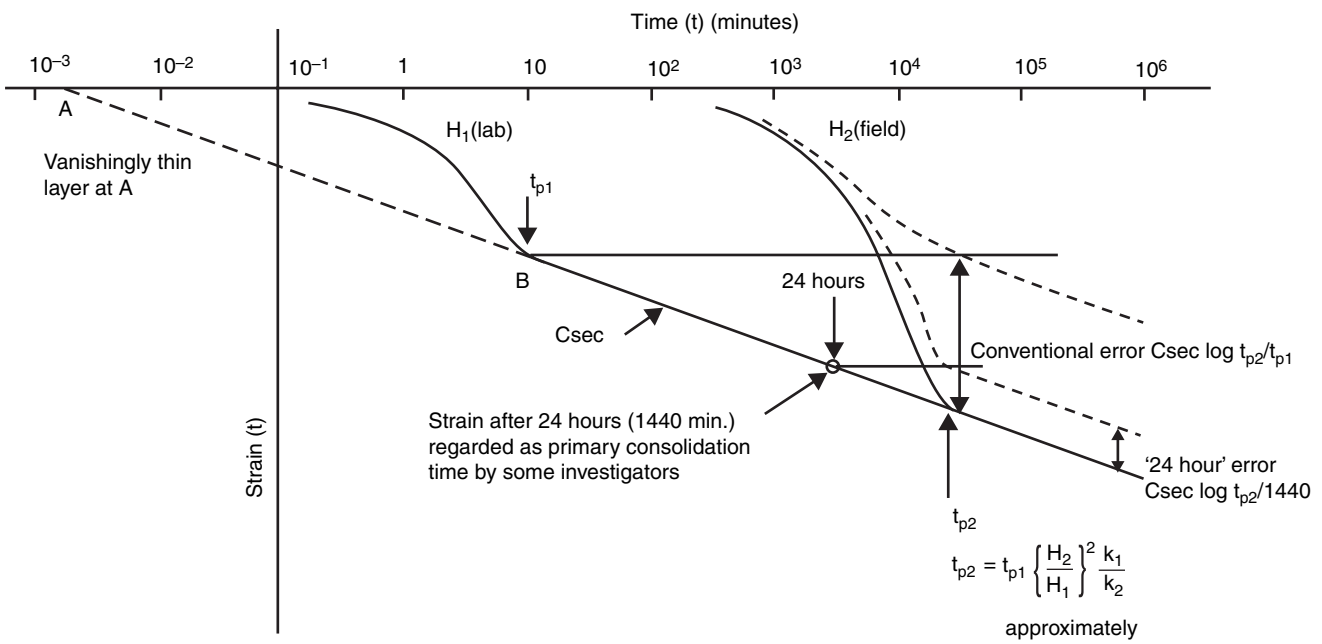


Figure 35.3 Comparison between EOP and continuous assumptions
Reproduced from Hobbs (1986) © The Geological Society

For those soils which have an essentially bi-linear e-log σ' relationship, which is not true for highly organic soils and peats as will be discussed later in this section, the compression and swelling indices, C_c and C_s, can be used to determine the change in void ratio due to a change in effective stress. Hobbs (1986) developed useful relationships for UK peats of $C_c = 0.0065w$ for bog and $C_c = 0.008w$ for fen peat. Mesri *et al*. (1997) suggest that $C_c = 0.01w$. Hobbs (1986) showed that there was a relationship between C_c (from 24 hour incrementally load consolidation tests) and initial void ratio which had a limiting value for peats of $C_c \approx 0.45e_o$.

It is generally more useful to consider the settlements in terms of strain (ε) resulting from an effective stress increase from σ'_{vo} to $\sigma'_{vo} + \Delta\sigma'_{vo}$ which, for increases in vertical effective stress from below to above the pre-consolidation pressure, is:

$$\varepsilon = \frac{\Delta e}{1+e_o} = \frac{C_s}{1+e_o}\mathrm{Log}\left(\frac{\sigma_c'}{\sigma_{vo}'}\right) + \frac{C_c}{1+e_o}\mathrm{Log}\left(\frac{\sigma_{vo}' + \Delta\sigma_v'}{\sigma_c'}\right) \quad (35.3)$$

$$\varepsilon = \frac{\Delta e}{1+e_o} = CR\mathrm{Log}\left(\frac{\sigma_c'}{\sigma_{vo}'}\right) + RR\mathrm{Log}\left(\frac{\sigma_{vo}' + \Delta\sigma_v'}{\sigma_c'}\right) \quad (35.4)$$

where CR and RR are the compression ratio and recompression ratios respectively.

Given that $e_o >> 1$ for peats, this implies that $C_c/(1+e_o) \approx 0.45$ is a limiting value. It also means that for peats, the settlement is virtually independent of the void ratio or water content. The use of the compression and recompression ratios, $CR = C_c/(1+e_o)$ and $RR = C_s/(1+e_o)$ respectively, requires an assumption regarding the initial void ratio and is therefore not a fundamental soil parameter. However, it can be useful in practice to give an indication of the variation in compressibility of normally or lightly consolidated soils. The CR values determined from 24 hour standard incremental consolidation tests are essentially independent of water contents for liquid limits in excess of about 200% (Farrell and O'Donnell, 2007), which supports the contention of limiting values of this parameter (see **Figure 35.4**). As stated previously, these 24 hour results inevitable include some secondary consolidation.

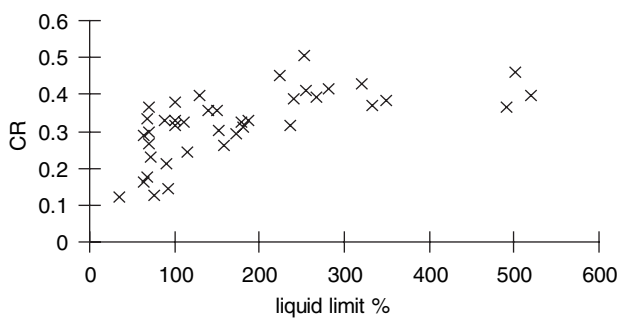


Figure 35.4 Variation of CR with w_L
Reproduced from Farrell *et al*. (2007)

Laboratory tests on highly compressible peat samples have shown that C_c is not constant at large strains (see **Figure 35.5**). This nonlinearity can be considered in the EOP method by using the actual curve e-logσ' (Mesri, 2007) or alternatively the plot can normally be linearised by using natural strain, e^H, in place of linear strain, e^c where $e^H = -\mathrm{Ln}(1-e^c)$. The superscript H refers to natural strains which are attributed to Hencky.

The void ratio and strain due to soil secondary compression under a constant effective stress generally increase linearly with log time; however, there are two parameters used to express this linearity, namely C_{sec}, which is the strain $\{\Delta e/(1+e_o)\}$ per log cycle of time, and C_α, which is Δe per log cycle of time. The strain is normally referenced to the initial height, rather than from the height at the end of primary consolidation. There is some evidence from laboratory tests that these coefficients may increase in time, called tertiary creep (Dhowain and Edil, 1980). The values of C_{sec} or C_α are generally considered to be independent of the applied stress for those stresses above the pre-consolidation/yield stress and for those soils which can be modelled by a constant C_c. There is a stress dependency to C_{sec} or C_α for stresses around the pre-consolidation/yield stress, and for those highly organic soils which cannot be modelled by a constant C_c. Mesri and Godlewiski (1977) consider that similar processes are involved in C_c and C_α, and by defining $C_c = \Delta e/\mathrm{Log}\sigma$ (which can be used above and below the pre-consolidation/yield stress) have shown that the relationship C_α/C_c is relatively constant. Mesri (2007) gives $C_\alpha/Cc = 0.06 \pm 0.01$ for a fibrous peat. Some researchers consider that the laboratory determined values of C_{sec} or C_α tend to underestimate

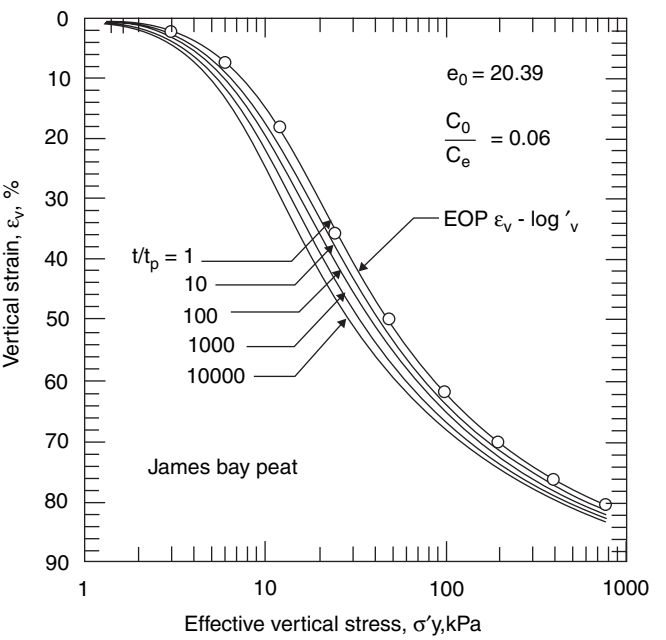


Figure 35.5 Vertical strain versus σ_v'
Reproduced from Mesri (2007), with permission from ASCE

the field creep by a factor of the order of two (Hobbs, 1986), whereas others consider that the values from field observations and laboratory tests are comparable (Mesri, 2007). The change in strain ($\Delta\varepsilon$) at constant σ'_v can be estimated from the relationship $\Delta\varepsilon = C_{sec}\text{Log}\frac{t}{t_P}$, where t is the time from the initial loading and t_p is the time to the end of primary consolidation. Thus the strain at different time increments can be drawn.

35.4.2.3 Continuous secondary compression approach

Several researchers have developed settlement models which consider that secondary consolidation is a continuous process, with that by den Haan (1996), called the abc model, being regarded as particularly appropriate but difficult to apply. The abc model incorporates the continuous creep, as well as other features such as the natural strain (e^H) and varying permeability with void ratio, which will be discussed below. The volume changes in the abc model are expressed in terms of specific volume ($\upsilon = 1 + e$) rather than in terms of void ratio and assume that the creep strain rate is uniquely defined by present stress and strain. The normally consolidated stress–strain relationship is $\text{Ln}v = \text{Ln}v_1 - b\text{Ln}\sigma'$, where v_1 is a reference specific volume at an effective stress of unity. Creep is defined by lines of constant creep rate $\left(\frac{d\varepsilon_c^H}{dt}\right)$ called isotaches, which have a slope b on an $\text{Ln}\upsilon - \text{Ln}\sigma$ plot as shown in **Figure 35.6**. The parameters used in this model are $b = \frac{\delta Lnv}{\delta Ln\sigma'_v}$; $c = \frac{\delta Lnv}{\delta Lnt}$ which are analogous to C_c and C_α respectively; and 'a', which is equivalent to a reload compression index and is defined as $a = \frac{\delta\varepsilon^H}{\delta Ln\sigma_v}$. The model also introduces the parameter called the intrinsic time (τ). Creep at constant effective stress, ε^H, is assumed to be linear when plotted against the logarithm of intrinsic time τ. The use of intrinsic time is necessary because of the complications that arise with a logarithmic scale and may be considered as the time which would be necessary to achieve the present volume if the present stress had been applied immediately to the soil in its freshly sedimented state. The intrinsic time of the base line is given the symbol τ_o. The isotaches are given by:

$$-Ln\frac{v}{v_1} = bLn\sigma'_v + cLn\frac{\tau}{\tau_0} \tag{35.5}$$

The total strain rate $\left(\frac{d\varepsilon^H}{dt}\right)$ is the sum of direct strain rate $\frac{d\varepsilon_d^H}{dt} = a\frac{dLn\sigma_v}{dt}$ and the creep strain rate $\frac{d\varepsilon_c^H}{dt}$ which is related to intrinsic time by $\frac{d\varepsilon_c^H}{dt} = \frac{c}{\tau}$. Thus the total strain rate is:

$$\frac{d\varepsilon^H}{dt} = \frac{d\varepsilon_d^H}{dt} + \frac{d\varepsilon_c^H}{dt} = \frac{a}{\sigma_v}\frac{d\sigma_v}{dt} + \frac{c}{\tau}. \tag{35.6}$$

These relationships define the behaviour of the soil and can be incorporated into a finite difference solution based on continuity of mass to model the hydrodynamic stage of the settlement process (den Haan, 1996). The use of this model is also discussed in O'Loughlin and Lehane (2001). The use of the isotache concept is discussed in detail in Degago *et al.* (2011).

Berry and Poskitt (1972) have used rheological models for peats with some success, but these have not been adopted for general design.

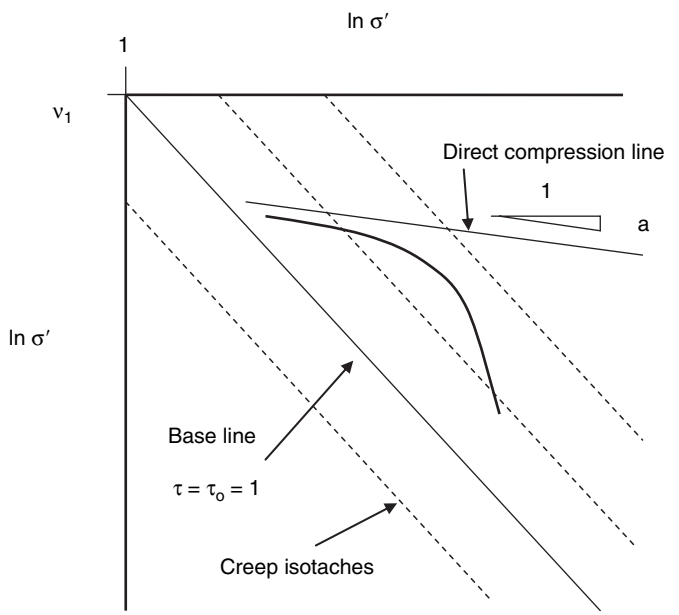


Figure 35.6 Stress vs creep relationship for abc model

35.4.2.4 Numerical modelling

Computer codes are being developed which include many of the features of the settlement soil models discussed above (see section 35.4.2.3) (Blommaart *et al.*, 2000) and these could be of particular benefit in giving confidence in analysing design situations with peat when they have been verified by case studies, such as that by Tan (2008).

35.4.3 Surcharging

Secondary compression settlements, which can give rise to significant serviceability issues for roads and structures founded on peats and organic soils, can be reduced by surcharging, particularly as it can benefit from the high initial permeability of fibrous peats. Care must be taken, however, that the increased load does not give rise to instability. The effectiveness of this method

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depends on the surcharge ratio $R_s' = \frac{\sigma_{vs}' - \sigma_{vf}'}{\sigma_{vf}'}$ where σ'_{vf} is the post-construction effective vertical stress and σ'_{vs} is the effective vertical stress reached during surcharging (Hanrahan, 1952; Mesri, 2007). The design criterion for a surcharge is usually to build out sufficient settlement during construction that would include both the primary and secondary settlement to be expected over the design life of the structure if a surcharge were not applied. The benefits of a surcharge are illustrated in **Figure 35.7**, where Curve A is the estimated settlement/time using, for example, equation (35.4), with an embankment at the design height with no surcharge, with the total estimated settlement after the design period (t_{design}) to be X, together with an allowance for rebound and swelling. A surcharge is selected to give the settlement X within the construction period as shown in **Figure 35.7**. The soil is allowed to consolidate under a surcharge $\Delta\sigma$; there is a rebound on removal, computed using $C_s \approx 0.1C_c$, long-term rebound (Samson and La Rochelle, 1972), then slight swelling until the creep line for that effective load is reached.

The longer the maximum load is maintained, the greater the delay in recurrence of the secondary compression. In both field and laboratory, the rebound shows a marked increase with surcharge ratios exceeding about three (Hobbs, 1986). The abc model of den Haan (1996) can also be used to estimate the effectiveness of a surcharge.

35.4.4 Rate of settlement

The general experience is that the field permeability of fibrous peat is significantly greater than that recorded in the laboratory consolidation tests (Hobbs, 1986). This is considered to be due to the limited size of the samples tested in the laboratory, to gas, and to the natural heterogeneity of peat to be expected from the plant growth patterns. The relatively high permeability of the acrotelm and the anisotropy of the peat deposits are also considered to contribute to increased field consolidation rates. The reported comparison between the time for primary consolidation of laboratory samples (t_s) and field consolidation times (t_f), and the respective sample height (H_s) and field thickness (H_f) using

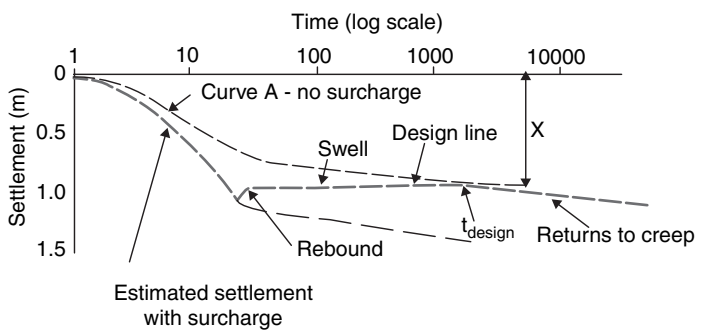


Figure 35.7 Simplified method of assessing effectiveness of surcharging (t_p = time to primary consolidation)

Equation (35.7) give i values from 0 to 2 for peats. Thus i = 0 implies that the time for primary consolidation is independent of the thickness of the peat, whereas if the peat complied with Terzaghi's theory of consolidation, i = 2.

$$\frac{t_f}{t_s} = \left\{\frac{H_f}{H_s}\right\}^i. \qquad (35.7)$$

Hobbs (1986) considers that the time for primary consolidation for peats can only be determined from field trials, which would require the installation of piezometers. Generally researchers have found that the time for primary consolidation in the laboratory, inferred in the normal way using the semi-log plot or Taylor's method, was approximately consistent with the end of the dissipation of excess pore pressures (Lefebvre *et al*., 1984; Mesri, 2007). O'Loughlin (2007), however, found that Taylor's method, and to a lesser extent Casagrande's method, consistently underestimated the time for primary consolidation.

The variation in permeability with effective stress would be expected to vary significantly with the type of peat and organic soil, which depends on the macro- and microstructure of the peat as well as on its degree of decomposition. It is common to express the change in permeability with respect to void ratio as $C_k = \Delta e/\Delta \log k_v$ (where Δe is the change in void ratio and k_v is the permeability in the vertical direction) and Mesri (2007) considers that $C_k = 0.25e_o$ for fibrous peat, where e_o is the *in situ* void ratio. Hanrahan (1954) considered the change in permeability to be linear on a log e–log k plot and derived $k = k_{vo}\left\{\frac{e}{e_o}\right\}^n$, where k_{vo} and e_o are the *in situ* permeability and void ratio respectively. Hobbs (1986) interpreted $k_{vo} = 4 \times 10^{-6}$ m/s, $e_o = 12$, $n = 11.03$ for a fibrous peat, where k_{vo} is the initial coefficient of permeability in the vertical direction. Hogan *et al*. (2006) reported k values of 2×10^{-2} m/s near the surface of a fen peat, which reduced to 10^{-5} to 10^{-6} m/s at 2–3 m depth in virgin peat. Hanrahan (1954) reported a decrease in k_v from an initial value of 4×10^{-6} m/s to 2×10^{-8} m/s under a pressure of 55 kPa for two days which decreased further to 8×10^{-11} m/s after seven months. Generally the horizontal permeability has been found to be significantly greater than the vertical by a factor of between three and ten (Mesri, 2007). The *in situ* horizontal permeability of peat can be estimated in the field by carefully executed variable head permeability tests; however, the reduction in this value with the decrease in void ratio that results from an increase in effective stress would be an important consideration in the interpretation of the test results.

35.5 Shear strength of peats and organic soils

35.5.1 Introduction

Generally the shear strength of peats and organic soils is determined in the traditional manner with s_u for undrained loading

and c′, ϕ' for drained conditions, without special consideration being given to the fibre content, high compressibility or to the relatively high permeability and gas content of some of the fibrous peats. The high compressibility and fibre content do affect the behaviour of fibrous peats in the laboratory and field tests and are poorly considered in current methods used to interpret these tests. In laboratory tests, for example, the high compressibility of peat samples results in necking and dimensional non-uniformities of samples during the consolidation phase of triaxial tests; also the fibre effects mean that drained triaxial tests result in an essentially one-dimensional behaviour, with the result that the soil cannot be taken to 'failure' as defined by Mohr Coulomb criteria. The fibres also affect the interpretation of effective stress parameters in undrained triaxial tests as the low Poisson's ratio of the fibre content of peat means that the pore water pressures rapidly build up to equal the cell pressure, thus making the effective lateral stress equal to zero. Determining the appropriate field strength also has its difficulties as the fibre content affects field vane tests and typically the recorded s_u decreases with an increase in the size of the vane (Landva, 1980).

In practice, the effect of fibres is eliminated in ring shear tests and these give the lower values of ϕ' for conservative estimates. Recently more attention is being given to direct simple shear (DSS) tests, particularly for determining undrained shear strength parameters (Farrell *et al*., 1999; Boylan *et al*., 2008).

35.5.1.1 Effective stress parameters

Peats and organic soils are predominantly frictional materials with values of ϕ' which are very much dependent on the method of test. For example, ϕ_{peak}' values of the order 50–60° have typically been recorded in consolidated undrained triaxial tests on fibrous peats, compared with values of the order of 32–40° for direct shear tests and for ring shear tests on the same material. The accuracy of the interpretation of ϕ' on fibrous peats from undrained triaxial tests is affected by the low lateral effective stresses recorded at failure in these tests, which frequently are essentially zero. Similarly, it is generally not possible to bring a sample of fibrous peat to failure in a drained triaxial test due to the continual compression of the fibres, which can be compared to the compression of a mattress. Direct simple shear tests (DSS) have been used to estimate the effective stress parameters; however, it is necessary to make an assumption regarding the effective horizontal stress at failure. Farrell *et al*. (1999) considered, on the basis of a finite element analysis of the simple shear test, that ϕ' could be estimated from $\phi' = Sin^{-1}\left(\frac{\tau_f}{\sigma'_{vf}}\right)$, where τ_f and σ'_{vf} are the shear stress and vertical effective stress at failure. Tests by Yamaguchi *et al*. (1985a, 1985b) recorded ϕ' of 35° on triaxial compression tests on horizontal samples compared with 51°–55° on vertical samples, with even higher values of 62° being recorded in triaxial

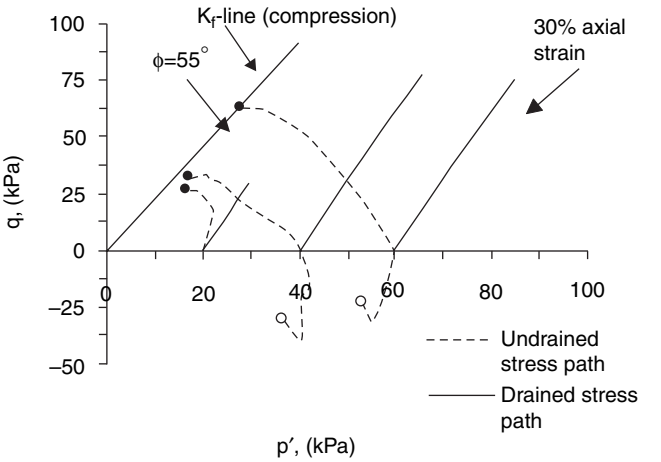


Figure 35.8 Triaxial tests on a fibrous peat
Reproduced from Farrell and Hebib (1998)

extension on vertical samples, which illustrates the inherent anisotropy. It is considered that ring shear tests, and possibly direct shear tests, give the intrinsic ϕ' of the constituents and that the higher values recorded in undrained compression and extension tests on vertical samples include the effects of the fibre structure. The relevance of these higher values must therefore be related to the actual failure mechanism being considered. There is little information on the effect of the possible presence of gas on the effective stress parameters. The results of a series of undrained and drained compression and extension triaxial tests on a fibrous peat reported by Farrell and Hebib (1998) are shown in **Figure 35.8**. Note that the drained tests, which were taken to 30% strain, did not reach the failure line and the ϕ' in compression of 55° was considerably greater than the ϕ' of 39° and 18° measured in the extension tests at initial consolidation pressures of 40 kPa and 60 kPa respectively. The direct shear and ring shear tests on this peat recorded ϕ' of 38°.

The effective stress parameters for organic soils which do not have a fibre structure can be obtained using the normal methods.

The at-rest pressure (K_o), which is normally related to ϕ' by Jaky's equation $K_o = 1 - \sin\phi'$, has generally been measured at between 0.3 and 0.35 on organic soils with water contents in the 330–850% range, which would suggest ϕ' values of the order of 40° to 44° (Mesri, 2007). Edil and Wang (2000), on the basis of laboratory tests, considered that K_o was of the order of 0.49 for amorphous peats and 0.33 for fibrous peats.

35.5.1.2 Undrained shear strength.

The undrained shear strength of organic soils (s_u) can be determined using the traditional methods of consolidated triaxial tests, *in situ* vanes, CPTu, etc. However, the results of these tests are affected by the fibres in the more fibrous peats. Furthermore, the undrained shear strength of some peats is

very low, of the order of 2–4 kPa, which is at the limit of the accuracy of normal testing arrangements. The gas content of peats can also affect their *in situ* behaviour, and this is frequently eliminated in laboratory tests by using a backpressure. This gas content can be lost when sampling the fibrous peat unless special measures are taken, as is evident from the length of samples recovered compared to the length pushed into the ground.

Laboratory tests have shown that peats have a high s_u / σ'_{vc} ratios in compression of about 0.5–0.6, which compares with ratios of about 0.3 for inorganic soils (Mesri, 2007), where σ'_{vc} is the vertical consolidation pressure. Similar high ratios have been recorded in extension. The direct simple shear test is receiving more attention in recent years as the failure mechanism is considered to be relevant to peat slides. Ratios of s_{uDSS} / σ'_{vc} of 0.45 (Porbaha *et al*., 2000), 0.46 (Foott and Ladd, 1981) and 0.5 (Farrell and Hebib, 1998) were recorded on fibrous peats, considerably higher than the ratio of about 0.22 expected for inorganic clays.

The field measurement of the *in situ* undrained shear strength of peats is problematic due to the influence of fibres on the results and also because of the low strengths measured. Boylan *et al*. (2008) show that temperature variations themselves can give rise to significant inaccuracies in CPTu probes, which often result in the interpretation of negative strength values, and advocate the use of full-flow penetrometers such as the T-bar and ball penetrometer for peats (Boylan and Long, 2006). The methods of determining the undrained shear strength of fibrous peats by field tests is an area that requires further research.

In the absence of an alternative field test, the vane is frequently used to determine an s_u of peat despite its significant limitations which have been discussed in Landva (1980). For example, the vane tended to pull the fibres along a surface outside the vane itself, there was compression of the peat in front of the vane and a gap frequently opened at the back of the vane, conditions far removed from a cylindrical shearing surface assumed when interpreting the results. Furthermore, the interpreted s_u values are generally found to decrease with an increase in the size of the vane, as is illustrated on a comparison of s_u values measured in a fibrous blanket bog and shown in **Figure 35.9**. These results are consistent with those reported by Landva (1980) where the s_u interpreted from a 55 mm × 110 mm vane were about twice that of a 75 mm × 150 mm vane and the interpreted undrained strength of this fibrous peat was as low as 2 kPa. Notwithstanding these limitations, the *in situ* vane is frequently used to assess the *in situ* strength of fibrous peats, as well as the increase in strength with consolidation, with the 55 mm diameter vane becoming the preferred size (Noto, 1991) and the test being considered to be an index test only. Mesri (2007) tentatively suggests that s_{uvane} be reduced by 0.5 to get the operational s_u. However, Noto (1991) estimated that the ratio of s_{uvane} / s_{ufield}, with s_{ufield} values being back-calculated from failure experiments on a testing embankment, to vary from 0.38 to 1.24 with an average of 0.74.

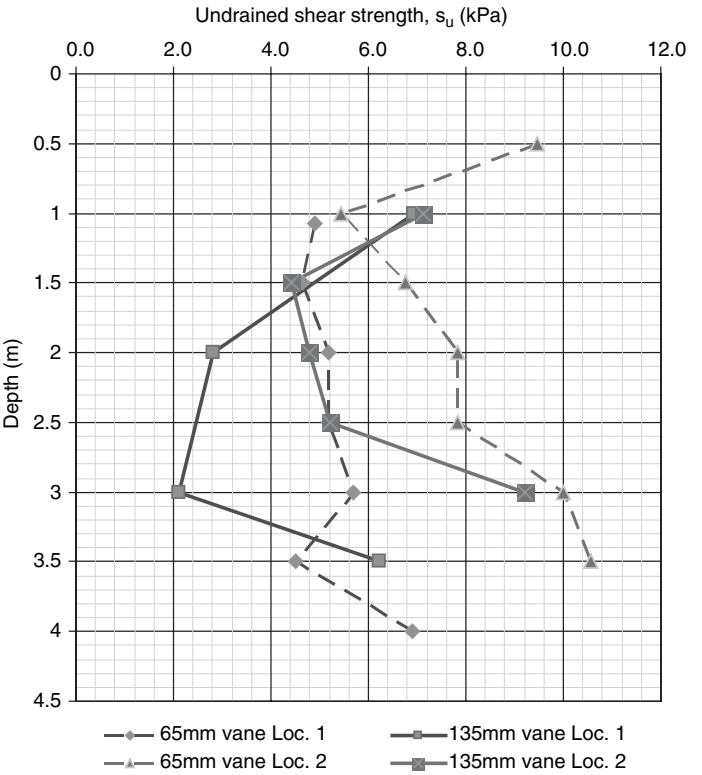


Figure 35.9 Comparison of s_u measured with 65 mm and 135 mm diameter vanes in a fibrous peat

35.6 Critical design issues in peats and organic soils

35.6.1 Roads

Most of the design/construction options for construction on soft inorganic soils can be used for roads on peats and organic soils, but these must be modified to consider the high compressibility/large deformations, the significant secondary compression, the low density of peats, and also the local geology within which the peat was formed. Munro (1991) gives a useful overview of some of the construction methods adopted by road authorities in Northern Europe.

The excavate/replace option is widely used for major roads on relatively thin layers of peats, generally up to about 3–4 m in thickness, as it avoids the uncertainties of long term secondary compression/creep movement which may arise even with surcharging. However, it becomes uneconomical for significant depths of peats due to cost and the environmental considerations of disposal of the excavated material. Additionally, the requirement for the replacement material to extend out from the footprint of the road to avoid surface cracks in the road pavement due to lateral consolidation (see **Figure 35.10**) can also significantly affect costs and land-take requirements. Lateral failure can occur if the granular replacement material is placed against a vertical or near-vertical peat face, due to the low density of the peat compared with a granular material.

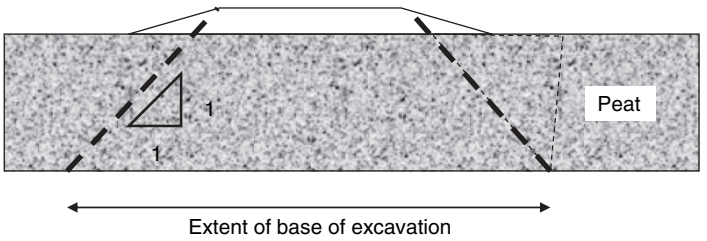


Figure 35.10 Extent of excavate/replace required

The replacement of peat layers may make it difficult to install vertical drains should these be required to speed up consolidation in underlying soft clay layers.

Historically, there is a legacy of old roads that have been constructed directly on top of the peats that currently serve in the minor road networks in the UK and Ireland. These roads, which generally started as gravel roads but were subsequently paved, develop an undulating profile under traffic and require regular maintenance and rehabilitation. The increased deformation under modern traffic also gives rise to transverse cracking, which can be mitigated to some extent by including geosynthetics to reinforce the bituminous layers or sometimes the sub-grade. Raised bogs frequently start their life over very soft lake-clay deposits; consequently the practice of placing additional surface material to periodically regulate the profile has resulted in past failures of the entire road due to shear failure through the soft clays. This problem can be avoided, and the secondary compression settlements reduced, by the use of lightweight fill in road improvement works.

'Floating' roads are a common method of forming access and low-traffic-volume roads on peats. Traditionally these were constructed using tree trunks as a base, but current practice is to use geosynthetics as basal reinforcement to a gravel layer where trees are not available. Such roads, which can carry heavy loads in fibrous peats due to the reinforcing effects of the fibres, are frequently used in forestry and wind farm sites in mountain areas; consequently their effect on the overall stability must be assessed in any design. Scottish Natural Heritage (2010) gives some guidance on the design and construction of floating roads on peat.

Lightweight fill, together with surcharge, is an acceptable construction option on peats and organic soils. Large-scale use of these materials tends to be limited by their costs. Lightweight fill is very efficient when used in combination with surcharge as the effectiveness of surcharge depends on the surcharge ratio, as discussed in section 35.4.3.

Stage construction with surcharging is the most common method for constructing on organic soils, and is sometimes used for peats. The terms 'pre-load' and 'surcharge' are considered to differ in that a pre-load is taken up to the design load whereas a surcharge is taken beyond the design load and, in the case of peats and organic soils, is designed to reduce secondary compression settlements. The principles of the design of surcharges are discussed in section 35.4.3. With reference to **Figure 35.7**, if the expected primary settlements and secondary settlements under a design load are estimated to be X, then the design load and a surcharge are left in place until this amount of settlement has occurred. Secondary compression settlements will occur after that design period, but at a lesser rate than would arise at the end of primary. In some situations, the settlements can exceed the thickness of fill placed, and it is necessary to apply more fill or to use lightweight fill for the initial loadings. The reduction in the effective stress due to submergence of the fill must be considered when estimating the settlements. Where the surcharge is not left in place for a sufficient time, care must be taken to ensure that there are no residual excess pore water pressures within low permeability layers after its removal.

The applicability of vertical drains to speed up the consolidation process depends on the type of peat or organic soil. Their usefulness for low embankments on fibrous peat which have high initial permeability is questionable, particularly as the drains themselves may be affected by the high compressions; however, the permeability decreases significantly at high effective stresses, thus the drains may be required where higher effective stresses are applied. General experience is that sand drains can act to reinforce peat, thus increasing its stability and reducing vibrations under load. Vacuum consolidation has benefits in speeding up consolidation but requires further research for use with the highly organic peats.

Stabilisation of peats and organic soils by mixing in a binder is a common practice in Sweden and Finland and in some other countries but has not been widely used in the UK or Ireland to date. The binder can be mixed in place using an attachment to an excavator as shown in **Figure 35.11** to form a stabilised mass, or alternatively a mixing tool can be used to form stabilised columns as shown in **Figure 35.12**, or a combination of both. Two types of mixing method can be used: the dry method where the binder is injected pneumatically, and the wet process where the binders are mixed in a slurry form before being injected from the mixing tool. The design and construction aspects using these methods are set out in the *EuroSoilStab* (BRE, 2002) and some experiences with Irish raised bogs are given in Hebib and Farrell (2003). The efficiency of this process is very dependent on the mineral content of the peat/organic soil and on the humic acid in the peat, the process being more cost-effective in organic clays than in the high organic peats. Stabilisation of the high organic peats in Ireland has not proven to be cost-effective to date as about 150–200 kg of cement per cubic metre of peat can be required to achieve the required engineering properties (Hebib and Farrell, 2003).

The efficiency of stone columns will depend on the organic content of the soil due to the inevitable lateral consolidation of the columns, and also due to the low strength of the peats. The actual feasibility of constructing *in situ* concrete columns in very soft ground could limit the use of that method.

Piled supported basal reinforced embankments have been constructed on very compressible and very soft peats (Orsmond,

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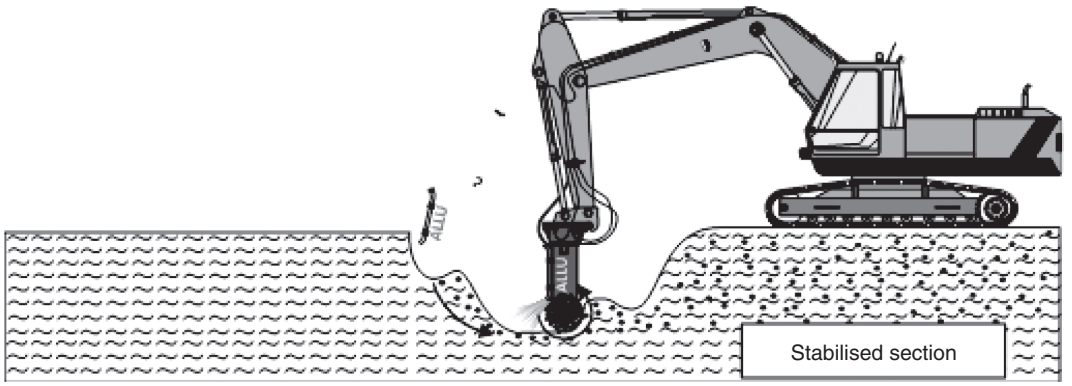


Figure 35.11 Mass stabilisation
Reproduced from BRE (2002)

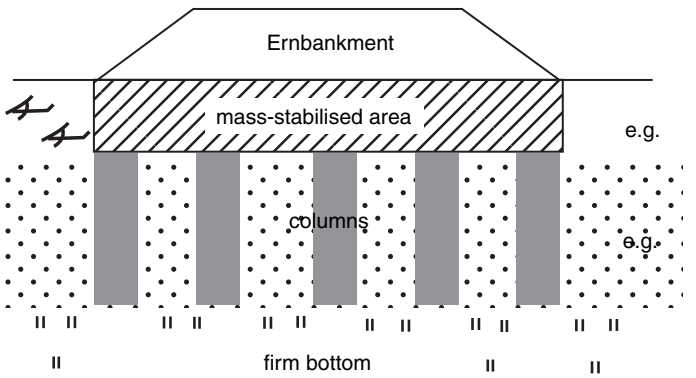


Figure 35.12 Column and mass stabilisation
Reproduced from BRE (2002)

2008). Issues that must be considered include the lateral stability of the entire embankment, the difficulties in maintaining the position of piles, the type of pile and the construction platform required to access the locations. Basally reinforced granular layers have generally provided a suitable platform for light piling rigs, even on very soft peats. Driven piles are commonly used. The feasibility of using cast-in-place or CFA piles will depend on the ability to get access for the rigs and also on the ability of the very soft material to support liquid concrete without radial failure. Although rarely reported in professional journals for legal reasons, failures have occurred in basally reinforced piled embankments due to stockpiling of material adjacent to the structure, giving rise to failure of the piles due to the lateral loadings. This failure mechanism also has implications for the type of structures that can be constructed on land adjacent to such structures.

35.6.2 Structures

The design of structures on peats and organic soil follows the normal procedure for building on soft soils, taking into account the high compressibility of peats. Structures are normally piled, although surcharging (see section 35.6.1 above) has been used in some situations with a rafted structure. It is normally necessary to construct a gravel-on-geogrid working platform to allow access for an appropriate piling rig and this platform will inevitably settle away from any piled structure, leaving a gap in the long term and also giving rise to downdrag forces on the piles. The risk of buckling of piles is assessed in the normal manner.

35.6.3 Slopes

Bog bursts and peat slides are major geohazards, particularly in Scotland and Ireland, and can be caused by environmental or anthropological factors. Major slides have occurred on peat slopes as shallow as 3° and have involved a mass of up to 100 000 tonnes of material covering about 0.5 km in width and over 1.5 km in length. Villages have been destroyed in the past, with one particularly serious incident in Castlegarde, County Limerick, Ireland in 1708 in which 21 people died. Recent research for the development of a landslide database has shown a proliferation of landslides in upland peat areas of Ireland. The Geological Survey of Ireland produced a booklet on landslides (Creighton, 2006), including bogslides, and the Scottish Government has published a document on peat landslide hazard and risk assessment (2006). Given that a peat bog sitting on the side of a hill is made up of over 90% water, it is not surprising that slope stability is a major issue.

Bog failures are generally thin relative to the length of material involved and therefore resemble a planar-type slide as discussed in Chapter 23 *Slope stability*. Typically the upper layers of bogs are fibrous and the reinforcing effects of these fibres prevent a local failure and thus contribute to the formation of a large mass movement with a failure surface at or close to the base of the slide. The determination of the disturbing force ($W\sin\alpha$, where α is the angle of the base to the horizontal) is relatively straightforward; however, the estimation of the resisting force depends on whether the failure is considered in terms of effective or total stress. With reference to section 23.3.2 of Chapter 23 *Slope stability*, and using the terms defined in that chapter, the resistance (R) is:

$$R = (W\cos\alpha - uA)\tan\varphi' + c'A \qquad (35.8)$$

Given the high water tables that normally are present in these bogs, and the fact that the bulk density of peat can be close to or less than that of water, the ($W\cos\alpha - uA$) term can be negligible and the resistance is then $c'A$. Considering the slope in terms of total stress, the disturbing force is the same as for the effective stress analysis, namely $W\sin\alpha$, and the resistance is s_uA; thus the difference in the analyses is whether c' or s_u should be used.

Some blanket bogs are known to have 'pipes' at their base through which water flows during heavy rainfalls (Donnelly, 2008). Water entry to the base of these bogs, with a consequential build-up of water pressures, is also facilitated by tension cracks forming in the peat during periods of dry weather. Given that the bulk density of peat is very close to or even less than that of water, the effective stress can be very small or negligible and the resisting force therefore depends on the apparent cohesion, if this is present. The failure surface of a well-documented peat slide in Pollatomish, County Mayo (Long and Jennings, 2005), which is shown in **Figure 35.13**, was at the interface of the peat and an underlying sand layer following a period of heavy rain, confirming that slide was caused by a loss in effective stress. The presence or otherwise of c' in such an analysis is debatable. The risk of such failures is increased by a prolonged period of dry weather which can dry out the peat, thus reducing the bulk density and which can also give rise to tension cracks which facilitate the access of water. The build-up of water pressures at the base of the peat can also be exacerbated by the presence of an iron pan layer which is sometimes noted at the base of blanket bogs.

When considering the stability of a peat layer of thickness D against planar failure in terms of total stress, with an s_u assigned to the peat, the disturbing force per unit basal area is $\gamma D\sin\alpha$ and the resisting force is a function of s_u, assuming there are no external loads and that the tension capacity of the peat is negligible compared with other forces. The ratio of resisting to disturbing force is then $s_u/\gamma D\sin\alpha$. This ratio therefore decreases with increased thickness, increased slope angle and increased bulk density. Some of the bog failures have occurred on very flat slopes (as low as 3º, but frequently on slopes of about 4–7º). Using this total stress approach, back-analysis of some of the bogslides would indicate s_u values at failure of the order 2 kPa, which is very low and difficult to measure in the field. Farrell (2010) proposed a mechanism to explain how a major bogslide could progress downhill when a local failure is initiated at the higher elevations if the material at the base of the slide is sensitive.

The stability of bogs can be affected by forestry drainage ditches and other anthropogenic factors.

35.6.4 Retaining structures, canals and dams

The special issues arising from the use of peats and organic soil in relation to retaining structures, canals and dams are their low density, high compressibility, high creep rate and generally low strength. Water-retaining structures require particular attention because small changes in the hydrogeology can result in very low effective stresses. Major failures have occurred in the past, for example the Edenderry canal failure of 1989 (Piggott *et al*., 1992), shown in **Figure 35.14**, which resulted in the horizontal movement of a 225 m × 105 m block of peat by a distance of about 60 m.

35.7 Conclusions

Peats and organic soils, which are to be found in many countries and are very extensive in some, present particular challenges to the geotechnical engineer because of their high compressibility, creep, low density and generally very low shear strength. Despite the presence of the cellular structure of organic remains, many of the correlations between the various



Figure 35.13 Peat slide at Pollatomish, 2003



Figure 35.14 Edenderry canal failure, Ireland, 1989
Reproduced from Piggott *et al*. (1992)

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properties and their behavioural patterns are little different from those of a clay, other than their shear strength.

Various processes can be involved in the formation of organic soils and the nature of the process can be important when interpreting the relevant geotechnical parameter and soil stratification appropriate to a particular design/construction situation. Classification systems have been developed to assist in the characterisation of the materials, the von Post (1922) or a variant being the system most generally adopted. The water content is the most important index test as it is a simple parameter to which many engineering properties can be related. However, the organic content, determined by the loss on ignition test, and the pH are also useful, as is the liquid limit which can generally be carried out on peats with a degree of decomposition greater than H_3 on the von Post scale.

The recovery of 'undisturbed' samples of highly organic peats, particularly fibrous peats, is difficult due to the compression of the material during sampling, even with sharp-edged piston samplers. Furthermore, the samples may undergo oxidation when exposed which can lead to decomposition and gas generation, although the effect of this is generally not considered significant except possibly for long-term testing.

The methods of estimating settlements and rates of settlements used for clays are also used for organic soils, although the effect of secondary consolidation/creep on these estimates increases with the organic content. Because of the large settlements involved in the highly organic soils and the natural variability within these deposits, simple behavioural models are usually used which consider secondary consolidation/creep to start at the end of the primary phase. Most sophisticated models which consider that secondary consolidation occurs continuously are difficult to apply but are being incorporated into some software programs. Surcharging of the peat is an effective method of reducing secondary compression/creep.

The undrained shear strength of organic soils and peats is normally determined using *in situ* vanes, cone penetration tests and laboratory triaxial or direct simple shear tests, although the most appropriate test for a particular failure mechanism in fibrous peats requires careful consideration. The very low undrained shear strength of some peats, sometimes of the order of 2 kPa, is below the accuracy of many of the current measurement methods. Similarly, the effective stress parameters determined in triaxial tests can be affected by fibres and by the method of test.

The unique properties of peats and organic soils have to be considered when designing roads and foundations or when assessing the stability of slopes in these soils. Some of the methods that have been adopted in practice have been presented within this chapter.

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35.8.1 Further reading

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It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 40 *The ground as a hazard*
- Chapter 48 *Geo-environmental testing*
- Chapter 49 *Sampling and laboratory testing*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 36

Mudrocks, clays and pyrite

Mourice A. Czerewko URS (formerly Scott Wilson Ltd), Chesterfield, UK
John C. Cripps University of Sheffield, UK

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Mudrock is the general name given to a large variety of fine-grained, clay-rich sedimentary rocks. As these materials occur frequently in the sedimentary rock sequence, they commonly form the natural ground on construction sites and they may also be encountered as Made Ground, fill or construction materials. Depending on compositional and structural factors, they can display a range of engineering behaviours – some of them problematic, including low strength, low durability and susceptibility to volume changes. In addition, it can be difficult to assess their mass properties. This chapter reviews the present state of knowledge concerning the performance of mudrocks in engineering situations. The intention is to provide the reader with a framework for understanding the geotechnical properties of mudrocks, especially with regard to effects induced by construction activities.

36.1 Introduction

The term mudrock is used here to denote a diverse group of fine-grained, clay-rich sedimentary rocks. Since such materials occur widely in the geological sequence, they are very frequently encountered in civil and environmental engineering projects. They are important because they can be the cause of a range of problems both during and after construction. A generally accepted geological definition of mudrocks that is suitable for engineering purposes is proposed by Stow (1981), as follows: mudrocks have a dominant grain-size of <63 μm of more than 50% of the constituents; and more than 50% the mineral constituents are composed of siliciclastic mineral fragments (including clay minerals, quartz and feldspar). Other fine-grained sediments with different compositions are designated accordingly as carbonate mudrocks, silica mudrocks, etc. As explained in section 36.1, there are discrepancies in terminology resulting in disagreement amongst different standards.

Mudrocks typically range from soft soil-like clay deposits to very stiff/hard soils and medium strong rocks. Due to the low strength and high compressibility of the former types, coupled with shrink–swell behaviour and, in the case of harder types, rapid breakdown in response to changes in moisture content and weathering processes, mudrocks present various challenges in civil engineering design and construction work. The controls on the performance of mudrocks and several forms of potentially problematic behaviour in different engineering situations are illustrated in **Figure 36.1**. Such behaviour often arises as a consequence of the combined effects of natural weathering processes and the impact of engineering works, but it is also strongly influenced by composition and structure of the material.

Although **Figure 36.1** is an extensive list of potentially problematic characteristics, mudrocks do have redeeming features. For example, their low permeability makes them ideal for the construction of low-permeability barriers and, as most types also possess high cation exchange capacity, they can remove potentially harmful dissolved species (for example heavy metals from groundwater). Mudrocks are also used as fills and in the construction of earthworks. Because most mudrocks have low strength they are relatively easy to excavate, but nonetheless most types perform satisfactorily as a bearing medium for moderately to highly loaded, low-sensitivity foundations.

Unfortunately, the reasons listed in **Figure 36.1** can hamper attempts to predict problematic forms of behaviour in mudrocks. However, an understanding of the composition and mode of formation of these materials, the selection of suitable methods of investigation and the correct interpretation of test results can all assist with this. Attempts are made to avoid excessive jargon but, where necessary, the reader should refer to the glossary at the head of this chapter for definitions of the relevant terms.

36.1.1 Definition and classification of mudrocks

The purpose of this section is to define the materials being described in this chapter. Besides the different uses of the terminology by geologists and engineers, and by workers from different countries, there is unfortunately a lack of agreement in the literature (including British and Eurocode standards) as to what constitutes mudrock.

Figure 36.2 shows the classification for mudrocks given in the current Eurocode 7 guidance documents. The terms mudstone and shale are shown to consist of a grain size of <63 μm, further defining siltstone as 50% fine-grained particles (2–63 μm) and claystone as 50% very fine-grained particles (<2 μm). As such these definitions are not clear. There are separate schemes described in BS EN ISO 14688–1: 2002 and –2: 2004 (for soils) and BS EN ISO 14689–1:2003 (for rocks), where soils are defined as materials with an undrained shear strength lower than 300 kPa (uniaxial compressive strength of 600kPa), and rocks are materials which are stronger than this. As shown in **Table 36.6**, this corresponds with a material that can be broken by hand pressure. As **Figure 36.2** shows, BS

EN ISO 14689–1:2003 defines mudstones as clastic sedimentary rocks composed of grains of quartz, feldspars and clay minerals in which the particles are predominantly smaller than 0.063 mm. The term argillaceous means that they are clay-rich and lutaceous, which is not a commonly used term meaning that they consist of fine-grained material or mud. Unfortunately the distinction between mudstone and siltstone or claystone is not explained, and although it is implied that mudstone comprises both silt- and clay-sized particles, the relative proportions of these is not stated.

Shale is also referred to in this table, to denote a fissile mudstone, in other words one that will split into thin sheets (see section 36.2.4.1). The term 'fissile mudstone' is preferred over the term shale, except where it is used as part of a formation name, for example Edale Shale. This is because in some mudrocks fissility only becomes manifested as a result of weathering action, so the same material could be called shale or mudstone depending on its weathering grade. In BS EN ISO 14689–1:2003, a division is made between siltstone, in which the particles are mostly between 0.002 and 0.063 mm in size, and claystone, in which most are finer than 0.002 mm. This follows the same approach as in IAEG (1979).

The guidance in BS 5930:1999 + A2:2010 *Code of Practice for Site Investigations* defines mudstone as rocks finer than 0.002 mm, thus using the same definition as for claystone in BS EN ISO 14689–1:2003. With such lack of agreement between standards, it is therefore vital to specify the scheme being used for classification. Further discussion of this issue is presented below and in section 36.1.2, which includes details of Grainger's (1984) classification of mudrocks.

BS EN ISO 14688–2:2004 specification for the classification of soils defines clays as possessing an average grain size of less than 0.002 mm, and advice is offered in section 36.3.2.2 of this chapter concerning the identification and description of such soils using simple physical tests. BS 5930:1999 + A2:2010 also provides guidance on the identification and classification of soils, including fine-grained ones such as clays. It accordingly indicates that the predominant grain size of clays is less than 0.002 mm and suggests that classification should be based on consideration of plasticity rather than grain size alone. In view of the average strength criterion mentioned above, most over-consolidated clays fall within the definition of clay soils.

BS 5930:1999 + A2:2010 and BS EN ISO 14689–1:2003 respectively define calcareous mudstone and marlstone as rock consisting of grains less than 0.063 mm in size with some carbonate. Although both standards indicate that there should be more than 50% carbonate present, it is likely that these rocks will contain less than this. A number of other types of rocks, for example chalk, rocks of chemical origin such as chert, and some rocks of volcanic origin, contain grains of clay size less than 0.002 mm – but as these do not contain clay minerals and detrital quartz grains, they are not regarded as mudrocks.

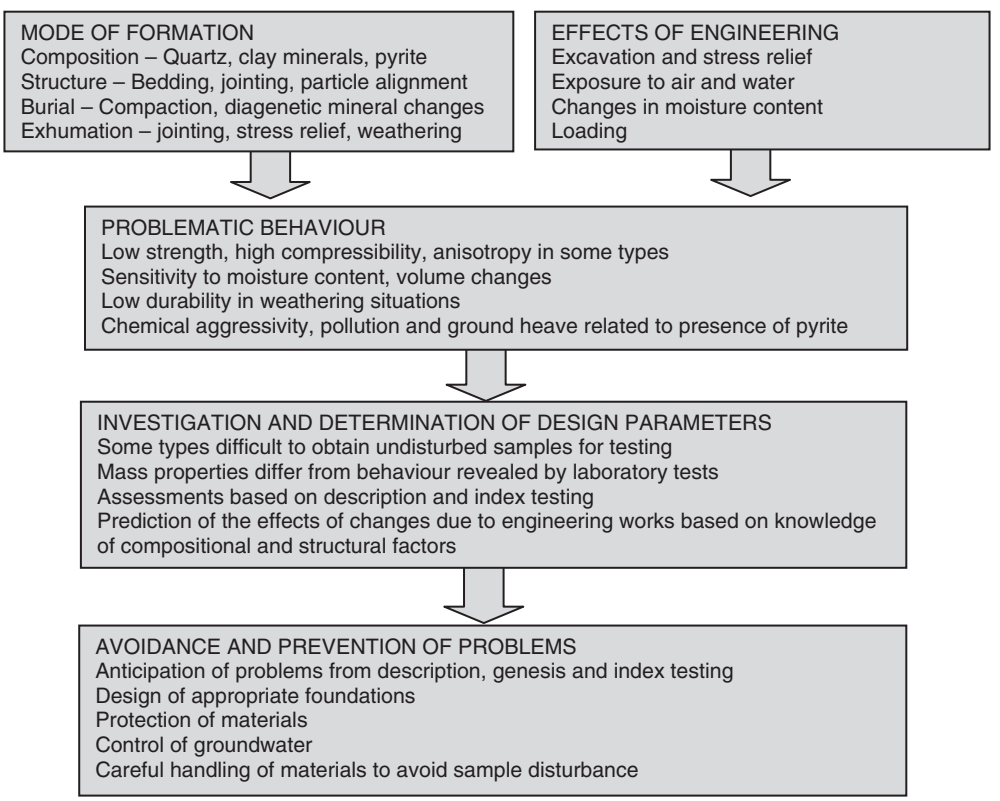
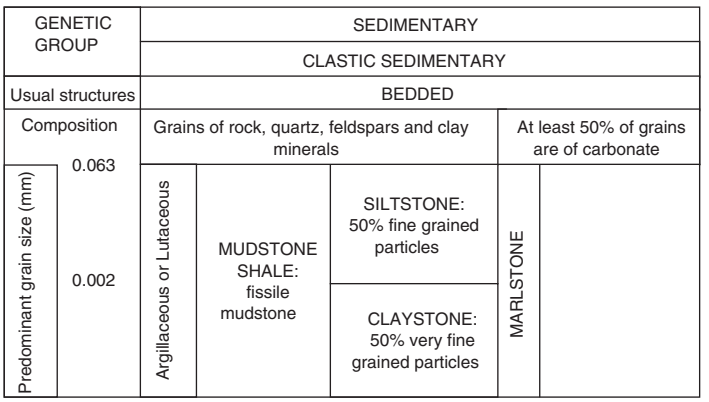
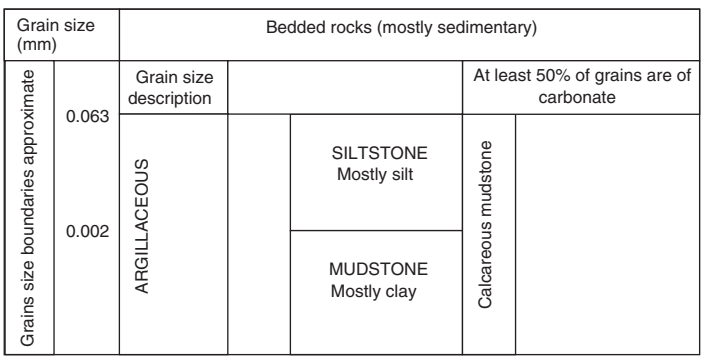


Figure 36.1 Interactions between mode of formation, effect of engineering and engineering problems in mudrocks

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(a)



(b)

Figure 36.2 Classification of mudrocks according to (a) BS EN ISO14689:2003 and (b) BS 5930 + A2:2010

Care must be exercised over the use of the term 'clay' which is used in three different ways:

- an average particle size of <2 μm;
- a type of mineral, such as illite, kaolinite, smectite, chlorite;
- a type of soil, including over-consolidated clays, that contains clay minerals and other constituents.

It needs to be understood that whereas clay minerals tend to be very small – they commonly form larger crystals, for example aggregations or flocs may exceed 10–20 μm in size. Although 2 μm is used in European classifications (including British ones) as the upper limit of clay size, some American standards regard 4 or 5 μm as the upper limit. A potential source of confusion arises from the names of some geological formations. For example, although Oxford Clay and similar deposits contain much clay, they also include other lithologies, such as limestone.

As Reeves *et al*. (2006) explain, although particle size is used as the main characteristic to classify mudrocks, it can be difficult accurately to determine the grain size distribution as most of the grains are too small to discern with the naked eye or even with optical microscopy. Furthermore, disaggregation can result in breakage of grains rather than breakage of the bonds holding them together. For instance, Davis (1967) noted that in Mercia Mudstone, clay mineral grains are cemented together in clusters so the measured clay size fraction is less than the actual amount of clay present. Features that can be used to assist with assessment of the grain size and identification are listed in section 36.3.2 of this chapter.

Various authors, including Spears (1980) have suggested that mineralogical data, rather than size, can be used as a guide to the sizes of particles present. However, this approach entails the use of relatively expensive X-ray diffraction or scanning electron microscope (SEM) methods, which therefore hampers its use in everyday investigations.

36.1.2 Compaction and cemented mudrocks

Whether the material behaves in a rock-like or soil-like fashion is highly relevant in many applications. Mead (1936) suggested the use of the terms:

- compaction mudrock – formed just by compaction due to burial;
- cemented mudrock – the grains are attached together by a significant amount of cement.

As further explained in section 36.2.2, in the former, the grains are not bonded together and the materials display soil-like behaviour, whereas intergranular cementation and recrystallisation result in rock-like behaviour in the latter. Mead used the terms compaction and cementation shales, but for the reasons stated above, it is preferable to refer to them as mudrocks. Morgenstern and Eigenbrod (1974), Wood and Deo (1975) and Hopkins and Dean (1984) suggest tests that can be used to distinguish between these types of mudrock.

The scheme shown in **Figure 36.3**, in which mudrocks are classified in terms of grain size, uniaxial compressive strength (σ_c) and second cycle slake durability index (Id_2) (see section 36.3.2.3), was devised by Grainger (1984) to provide a classification that reflects the engineering performance of mudrocks:

Soil (clay)	$\sigma_c < 0.60$ MPa
Non-durable mudrock	$0.60 < \sigma_c < 3.6$ MPa; $Id_2 < 90\%$
Durable mudrock	$3.6 < \sigma_c < 100$ MPa; $Id_2 > 90\%$
Metamudrock*	$\sigma_c > 100$ MPa

*Metamudrock is mudrock affected by low-grade metamorphism.

As mentioned above a uniaxial compressive strength (UCS) of 0.60 MPa is equivalent to an undrained shear strength of 300 kPa, where measurement is made perpendicular to any fissility or cleavage in the rock (see section 36.2.4.1). The quartz content distinguishes between claystone, shale, silty shale, mudstone and siltstone as follows:

Claystone and shale	< 20% quartz
Silty shale and mudstone	20–40% quartz
Siltstone	> 40% quartz

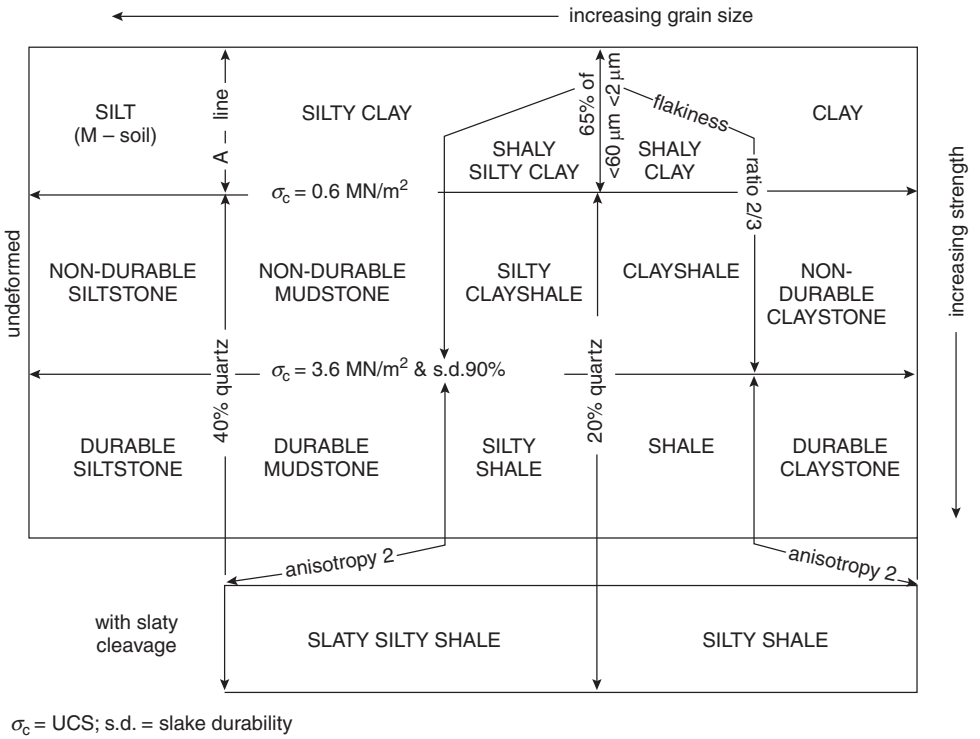


Figure 36.3 Grainger's (1984) classification of mudrocks

where the latter figure conforms with the suggestion made by Spears (1980). Grainger's (1984) classification also separates fissile from non-fissile material, where rocks with a flakiness ratio (i.e. shortest dimension of a fragment divided by the intermediate dimension) greater than 2/3 are regarded as fissile, while those with a ratio less than 1/3 are massive. For durable and slaty mudrocks the anisotropy may be determined using Point Load Test or Cone Indenter Test to determine the compressive strengths parallel and perpendicular to the predominant weakness surfaces (bedding or foliation), with a proposed anisotropy ratio >2 distinguishing fissile mudstones. As noted above, to avoid the possibility of the rock name changing as the rock becomes weathered, it is preferable to avoid the term shale and refer to fissile mudstone or claystone instead. Argillite is an indurated clay-rich rock with a random arrangement of particles, whereas in slate the rock possesses cleavage due to a preferred orientation of grains.

36.1.3 Distribution of mudrocks in the UK

As **Figure 36.4** shows, mudrocks occur widely in the UK. Generally speaking, younger, less indurated varieties underlie large areas of the south and east of England, including the London area. Thus a number of important urban areas and infrastructure routes lie on the outcrop of mudrock formations. Older, more indurated mudrocks occur in the Triassic, Permian and Carboniferous rocks of the Midlands, west and north England, south and north Wales and the central valley area of Scotland. These regions coincide with large urban areas and transport links.

Older terrain, ranging from Precambrian to Devonian in age, outcrops in the western and northern parts of the UK. Whereas most mudrocks in these areas have been metamorphosed into slates and schists, there are isolated outcrops of less indurated rocks. Although less populated, the presence of mudrocks may still affect engineering design and construction in these areas.

36.2 Controls on mudrock behaviour

By definition, mudrocks consist of deposited mineral grains such as quartz and clay minerals, with an average size of less than 0.06 mm. In addition to clay minerals and fine-grained quartz, minor amounts of feldspar, calcite and mica together with organic matter and, as explained below, various diagenetic minerals, may be present. Although the properties of mudrocks are very much affected by the nature and the relative amounts of the mineral constituents, the density and structure are also important.

36.2.1 Constituents of mudrocks

As clay minerals tend to exert a dominant control over the physical properties of mudrocks, knowledge of these minerals assists in understanding mudrock behaviour. Clay minerals have a sheet-like structure typified by mica which, as shown in **Figure 36.5**, is composed of alternating tetrahedral layers of silica (Si_2O_4) and alumina ($Al_2(OH)_6$), accompanied by

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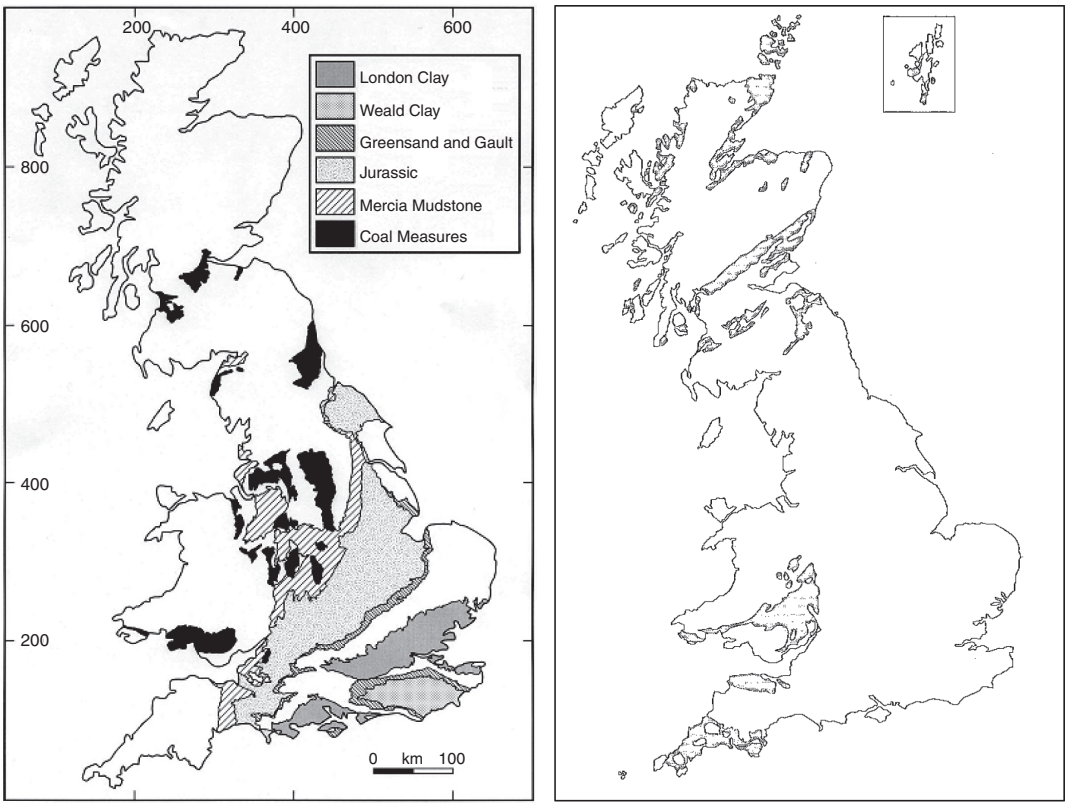


Figure 36.4 Distribution of main mudrock formations in the UK: (a) Carboniferous to Eocene; (b) Pre-Cambrian to Devonian mudrocks
Reproduced from Reeves *et al.* (2006) © The Geological Society of London

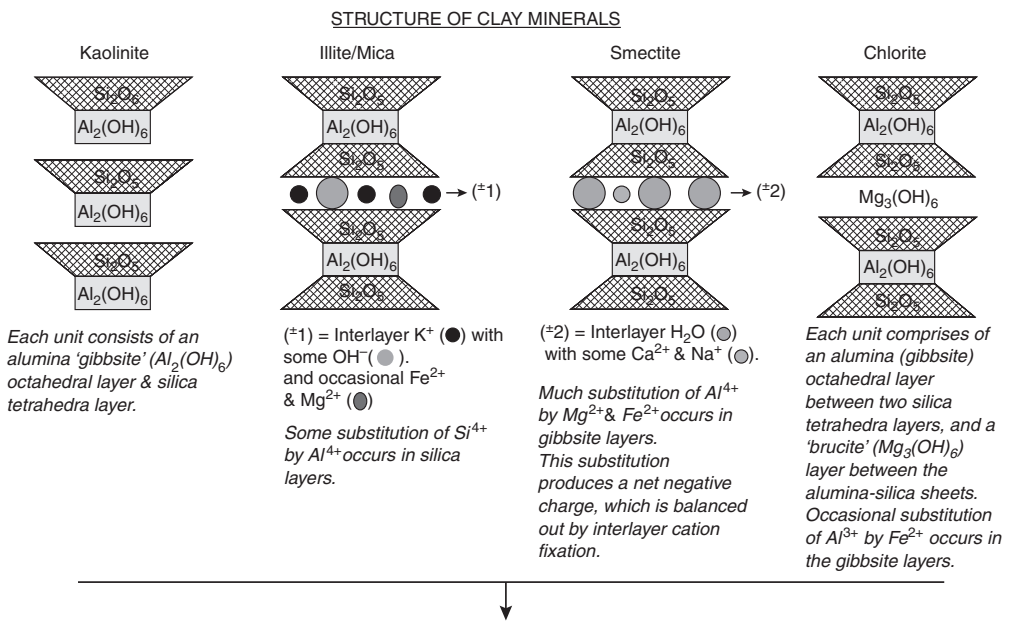


Figure 36.5 Platy structure of typical clay minerals showing potential for swelling by ion exchange

intermediate layers of exchangeable cations. Within this basic framework a complex mineralogy exists, based on the number of silica–alumina sheets and their arrangement. Ionic substitution within the basic sheet framework by ions such as Mg^{2+}; Fe^{2+} also influences the properties of the mineral as well as its cation exchange capacity and shrink–swell behaviour.

The formation of clay minerals by the chemical weathering of silicate minerals such as feldspar is illustrated in **Figure 36.6** (see also section 36.2.3). Whether mudrocks originate as *in situ* residual or transported deposits depends mainly on the climatic conditions prevailing in the area. Residual deposits, usually dominated by kaolinite, commonly occur in hot and humid weathering environments (see Lum, 1965). On the other hand, in temperate climates the products of weathering are usually dominated by illite.

The type of rock being weathered also influences the clay mineral assemblage; for instance, kaolinite is derived from granite degradation, whereas illite results from the weathering of pre-existing mudrocks and mica-rich rocks. Smectite forms in volcanic terrains and low silica parent rock. As explained below, some clay minerals referred to as authigenic minerals are precipitated from pore solutions. Chlorite may be present in most mudrocks, particularly those derived from weathering of metamorphic materials such as argillite, phyllite or schist. Several non-clay minerals such as quartz, various carbonates and pyrite, together with organic matter, are commonly present.

Because of the atomic structure of clay minerals, they typically occur as relatively small flat or platy particles where, as illustrated in **Table 36.1**, the average size, specific surface area and activity contrast dramatically with those of equidimensional minerals like quartz and feldspar. The relatively high surface area of swelling clays (such as smectite) is further increased as the interior of the crystal and the exchangeable

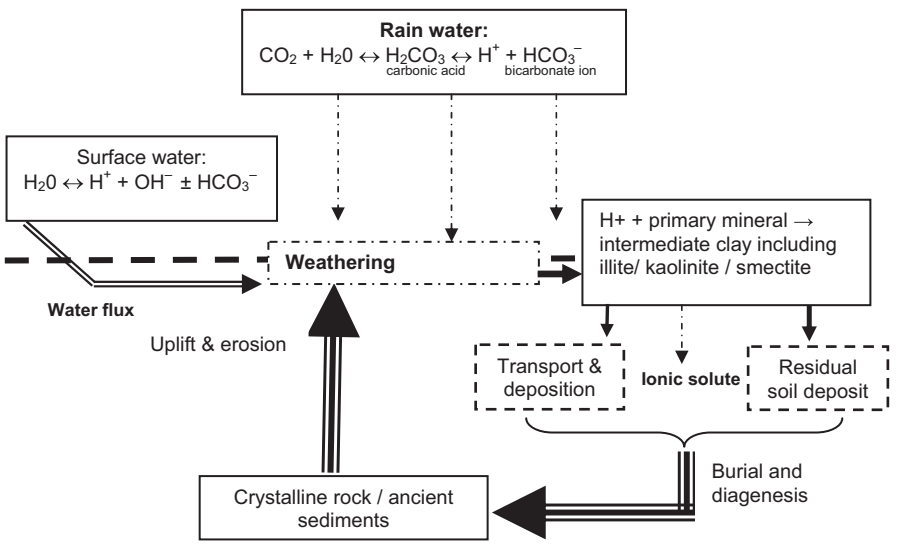


Figure 36.6 Weathering of existing rocks to produce clay minerals

	Montmorillonite	Illite	Kaolinite	Chlorite	Allophane	Quartz
Basal diameter (μm)	0.0–2	0.1–2	0.1–4	0.1–2		50–5 000
Basal layer thickness (Å)	10	10	7.2	14		
Particle thickness (Å)	10–100	50–300	500	100–1 000		$10^5 – 10^7$
Specific surface area (m^2/g)	700–800	80–120	5–20	80		10^{-4}–10^{-6}
Cation exchange capacity (meq/100g)	80–100	15–40	3–15	20–40	40–70	
Area per charge ($Å^2$)	100	50	25	50	120	
Liquid limit (%)	100–900	60–120	30–110	44–47	200–250	
Plastic limit (%)	50–100	35–60	25–40	36–40	130–140	
Shrinkage limit (%)	8.5–15	15–17	25–29			
Activity	1.5–7	0.5–1	0.5	0.5–1.2		

Table 36.1 Specific surface area and activity of quartz and certain clay minerals
Data taken from Mitchell (1993) and Hillel (1980)

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ions may be accessed by the pore fluid. Reeves *et al.* (2006) explain that minerals such as illite and smectite and, to a much smaller extent, kaolinite, have a net negative charge on their composite layers due to the substitution of high valency cations by lower valency ones. Such charges are balanced by the interlayer cations and when suspended in water, each clay particle is surrounded by a diffuse double layer of hydrated cations. Due to broken ionic bonds at the edges of clay particles, the edges become negatively or positively charged depending on whether conditions are alkaline or acidic, respectively. The ions adsorbed onto clay particles may be exchanged by other ions in the solution, resulting in changes in the thickness of the double layer and of the mineral, giving rise to cation exchange capacity in clays. The exchange of cations into and out of the double layer occurs in response to imbalances in the concentration of cations in the double layer and the surrounding solution.

The relatively large surface area of clay minerals means that they adsorb large amounts of water onto their surfaces, and the shared water between particles imparts plasticity to clays. The activity, which is a function of plasticity and percentage of particles less than 2 μm, is an indication of the sensitivity of the material to water. A high activity indicates a more plastic behaviour and greater shrink–swell potential in response to changes in moisture content.

The relatively small size of clay particles also means that they are attracted to each other by van der Waals forces, which operate over very short distances, and in addition are subject to electrostatic attraction and repulsion forces which act over longer distances. The strength of the latter forces depends on the ionic strength and composition of the pore water surrounding the particles as well as the distance between them. Thus particles in solution are subject to forces of attraction and repulsion depending on the distance to neighbouring particles and the chemistry of the solution. Generally speaking, in conditions of high ionic strength, in other words saline conditions, the diffuse double layer is suppressed and attraction forces tend to dominate – which causes the particles to form flocculated groups with many face-to-edge contacts. This so-called card-house structure gives rise to a relatively low density, high water content, clay deposit. In low ionic strength conditions, the repulsive forces dominate and the clay particles are deposited in a dispersed state which has lower porosity. High amounts of organic matter also increase the tendency for clay minerals to form dispersed structures.

The non-clay constituents of mudrocks include quartz, feldspar, micas, carbonates, heavy minerals, organic matter, pyrite and iron minerals. Quartz and finely crystalline (cryptocrystalline) forms of silica, such as flint or chert, are usually present as detrital (or deposited) grains. Mudrocks are liable to contain iron oxides and hydroxides as well as the iron sulfide mineral pyrite which, as explained in section 36.2.4.3, is a common and very important, though minor, constituent.

36.2.2 Deposition, burial and diagenesis

The environment of deposition exerts a strong control – not just over the type of material deposited, but also over the presence and spacing of bedding surfaces and other sedimentary structures. As explained in section 36.2.1, clay particles may be deposited as dispersed individual particles in fairly dense arrangements, with the platy particles aligned approximately horizontally or in flocculated aggregations of clay particles, with an open card-house or honeycomb-type structure in which the particles are in a random arrangement. Such deposits possess relatively high porosity – in the region of 70–90%, and high water content.

Burial of the sediment beneath younger sediments removes access to air and increases both the vertical stress and temperature. An associated reduction in porosity and moisture content is accompanied by the formation of diagenetic minerals, changes to clay minerals and the precipitation of cements, as illustrated in **Figure 36.7**. The exclusion of air can result in sulfate-reducing bacteria releasing oxygen from seawater-derived sulfate (SO_4^-) by reaction with detrital iron in the sediment. The result of this process is the formation of pyrite 'framboids' in the upper levels of the sediment. As **Figure 36.8(a)** indicates, framboids comprise clusters (measuring about 1–10 μm across) of minute pyrite grains. In more oxygenated conditions, nodules or amorphous masses of calcite or iron carbonate (siderite) may be formed (**Figure 36.9**).

Burial of muddy sediment beneath subsequent deposits brings about progressive reductions in porosity and moisture content, as well as changes (diagenesis) to some minerals and production of new authigenic minerals (see **Figures 36.10** and **36.11**). These diagenetic effects increase with burial depth and duration of burial, until depths of 10–12 km are reached, where there is a transition to low-grade metamorphism.

The results of studies into the effects of burial of mud to different depths are summarised in **Figure 36.7** and the related structural changes to mudrocks are illustrated in **Figures 36.11** and **36.12**. Czerewko and Cripps (2006b) explain that in the course of this process, clay minerals are squeezed and re-orientated to fill the voids between the clastic grains. Any surviving clay flocs are destroyed and porosity, water and volatiles are lost. In clay-rich deposits or clay layers in laminated deposits (see **Figure 36.12(c)**), this results in the re-orientation of clay particles towards the horizontal (see **Figure 36.12(b)**), which will tend to render the material anisotropic – so it may be weaker in one direction. The tendency to develop a preferred orientation in deposits comprising clay mixed with silt particles is hindered because the clay particles become packed into the spaces between silt grains (see **Figure 36.12(a)**).

Compaction mudrocks (see section 36.1.2) are liable to contain at least some swelling clay minerals such as smectite and mixed layer illite–smectite. Kaolinite and vermiculite may also be present, and pyrite will typically occur as framboids or very small crystals (see **Figure 36.8(a)**). The compaction experienced by the deposit will have increased its density, with

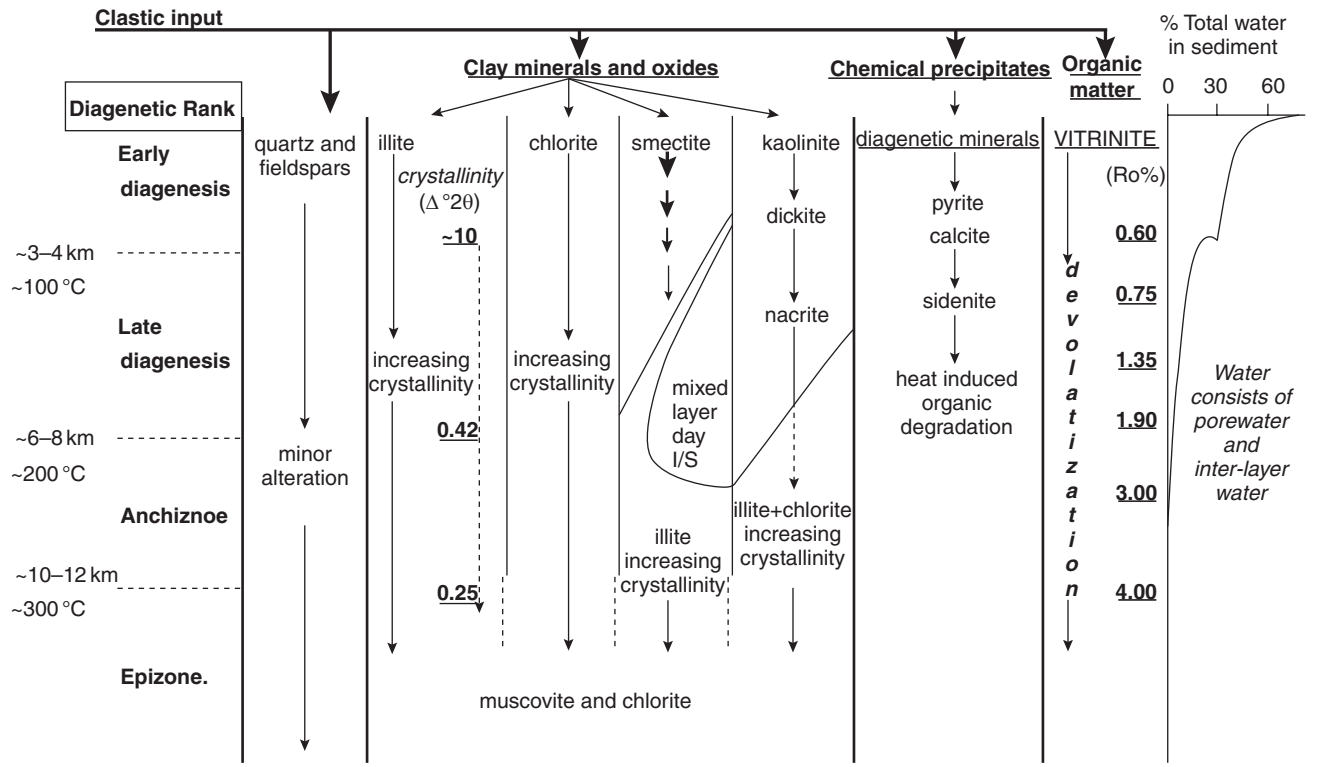


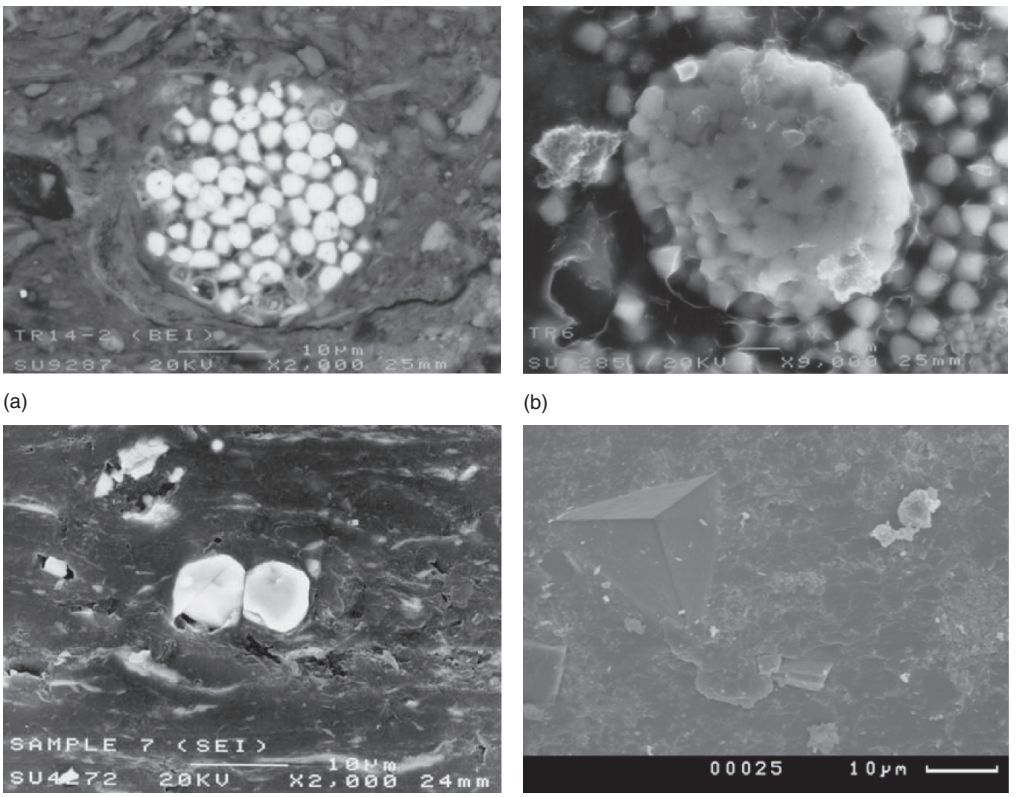
Figure 36.7 Changes in mudrock mineralogy and moisture content associated with burial Mixed layer clay I/S = illite/smectite
Reproduced from Czerewko and Cripps (2006b)

a commensurate reduction in moisture content, but the mineralogical changes will have been relatively minor.

Burial of about 1 km is required to bring about significant changes to clay minerals and pyrite. Besides further reductions in moisture content and increases in density, smectite and smectite–illite mixed layer clays are progressively converted to illite and ultimately, by metamorphic processes, to muscovite or sericite. This process involves the release of ions and water from the interlayer sites of these minerals, which may participate in the precipitation of authigenic clay minerals and cements in the pore space. If the clastic components are sufficient in quantity then, at a certain degree of compaction, a skeletal framework is formed within the deposit (**Figure 36.12(e)**). Where no skeletal clastic framework develops, as in clay-rich muds, then (as illustrated in **Figure 36.12(b)**) a deposit with an approximately horizontal preferred orientation of clay particles is the likely outcome and a fissile mudstone or claystone (see section 36.2.4.1) is produced. Weakly cemented mudrocks may contain some swelling clay minerals with illite of enhanced crystal size, together with a partial skeletal framework. Chlorite may also form during deep diagenesis and vermiculite may be converted during burial diagenesis through mixed layer corrensite to chlorite at depth.

Burial to 2–3 km usually results in a porosity reduction of 3–5% in strongly cemented mudrocks with elimination of most, if not all, of the low-stability clay minerals. A series of discrete bonded crystal domains is produced which, with a continued increase in burial depth and temperature, increase in size. They eventually coalesce to form a continuous framework of cemented grains such that the material progresses to become a cemented mudrock (**Figure 36.12(f)**). Ultimately, at burial depths of approximately 5 km, all the grains may become cemented. Not only is the total pore volume in the deposit decreased, but a reduction in pore throat diameter, which controls the movement of water, also occurs. This (as explained in section 36.3.2.3), together with the removal of swelling clay minerals and increased size of pyrite crystals (see **Figures 36.8(a)–(d)**), considerably enhances the durability of the material. In this process, any microfractures and other structural discontinuities in the material are healed – which augments its resistance to breakdown. As illustrated in **Figure 36.8**, the progressive recrystallisation of pyrite results in the conversion of framboids to fine-grained and larger crystals to the extent that eventually visible cubic crystals are produced.

Ancient residual deposits that have been indurated by compaction and cementation may form mudrock formations similar to those derived from transported clay-rich deposits. However, some residual deposits will appear to display rock-like behaviour due to the effects of high pore suction pressures caused by desiccation. Suction pressures that hold the particles in such



(a) (b)

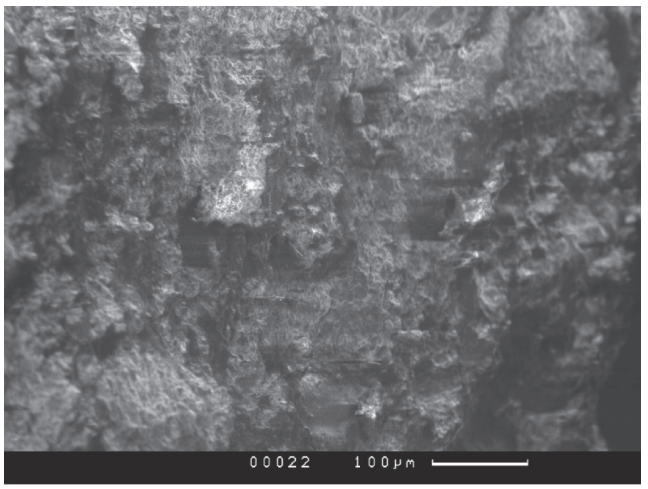
(c) (d)

Figure 36.8 Diagenetic changes in pyrite morphology associated with burial and recrystallisation, from poorly indurated compaction mudrock in (a) to well indurated mudrock in. (a) Pyrite framboid comprising a collection of discretepyrite crystallites (Lias Clay). (b) Partly cemented framboid with partialrecrystallisation of discrete crystallites (Oxford Clay). (c) Pyritohedral pyrite crystals resulting fromcementation of former framboids (UpperCarboniferous mudrock). (d) Pyrite cube from a well indurated mudrock (LowerCarboniferous).

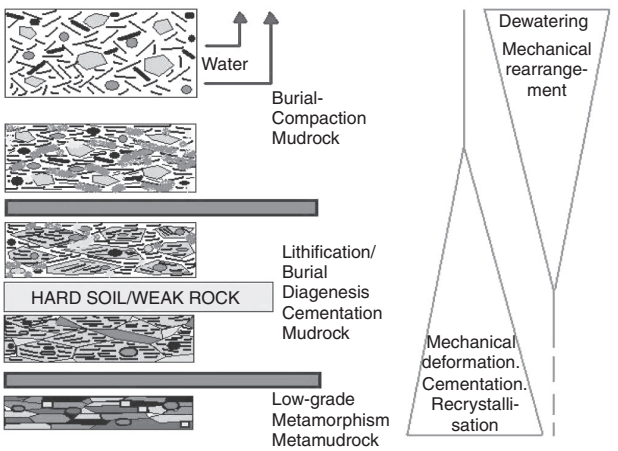


(a) (b)

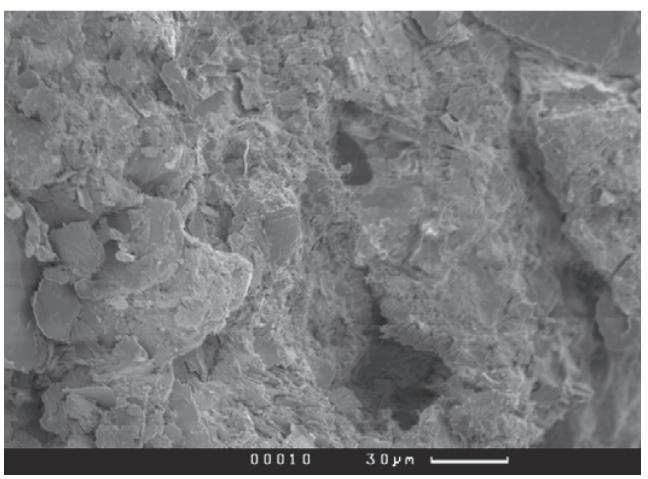
Figure 36.9 (a) Nodules of pyritic claystone in Jurassic mudstone from northeast England (note wrap-round distortion of bedding due to differential compaction of sediment); (b) fragmentation of core due to presence of strong claystone nodules in Carboniferous Coal Measures mudstone from the Midlands, England



(a)



(b)



(c)

Figure 36.10 Effects of burial on mudrock textures: (a) open 'loose' texture of a poorly indurated mudrock as seen in a sample of London Clay from southeast England; (b) changes occurring in conversion of a mud into a mudrock; (c) dense 'tight' texture with encapsulation of silt-sized quartz grains by clay minerals seen in indurated Carboniferous Namurian mudstone

deposits are destroyed by even slight wetting of the material, and with further wetting the material very rapidly reverts to soft clay.

36.2.3 Uplift, unloading and weathering

The changes in void ratio or moisture content, shear strength and effective horizontal stress that accompany burial of muddy sediment followed by reduction of overburden are illustrated by Cripps and Taylor (1981) in **Figure 36.13**. The effects of burial on mineralogy and microstructure are described in section 36.2.2 and are illustrated in **Figures 36.7** and **36.10**. As explained in Chapter 17 *Strength and deformation*, unloading results in the deposit becoming overconsolidated; in other words, it is denser and stiffer than it was for the same value of vertical effective stress during burial. Weathering processes entailing both physical breakdown and chemical alteration result in the fragmentation of the material, together with progressive dissolution of cements and other mineralogical changes. As **Figure 36.13** shows, these processes eventually result in the loss of the effects of diagenetic compaction and cementation, and the material is returned to a condition similar to the state when it was deposited.

One of the physical processes of weathering caused by the removal of overburden entails the vertical expansion of the material and the development of stress relief joints that lie roughly parallel to the ground surface. Due to lateral restraint, the horizontal stress induced by the original overburden pressure is not reduced in a commensurate manner, resulting in high horizontal stress. This is manifested by high K_o values in near-surface overconsolidated clay formations, as shown in **Figure 36.13**, as well as by the formation of systems of inclined fractures. These discontinuities facilitate access for water and gasses that participate in both physical and chemical weathering processes. Weathering processes cause progressive opening of discontinuities as well as the development of new ones. Softening and expansion of the material occurs as a result of the uptake of water and the weakening of bonds. The loss of strength of the material near to the ground surface means that there is a rapid drop in the K_o value at shallow depths.

Whereas in temperate climates physical weathering processes predominate, in tropical zones the bedrock is typically mantled with tens of metres of residual soil formed by chemical alteration of the underlying rock. For example, in Hong Kong the granitic and volcanic areas are mantled by up to 60 m of clay-rich residual soils (see Lum, 1965). The main weathering effects will be concentrated mainly along discontinuities, and a progressive increase in the proportion of weathered products is accompanied by a decrease in the size and number of corestones towards the surface.

Chemical weathering processes are illustrated in **Figure 36.6**; however, as explained in section 36.2.4.3, microbial activities that participate in the weathering of pyrite are also very important. Other biological processes, such as the growth of plant roots, can have both physical and chemical impacts.

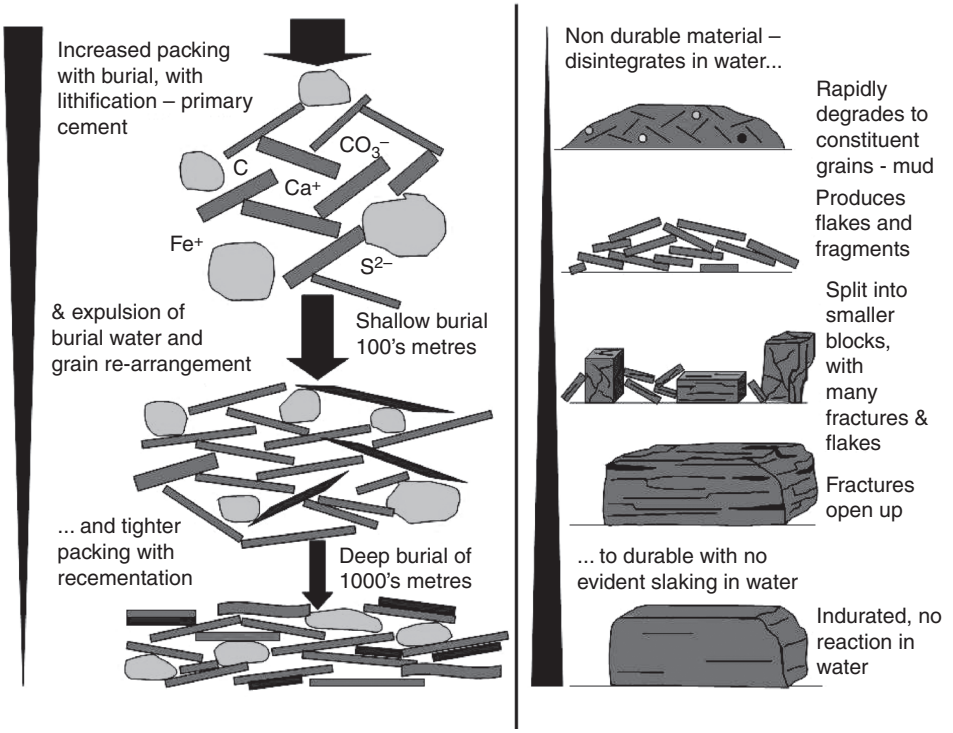


Figure 36.11 Effects of burial on the structure and behaviour of mudrocks

The impact of unloading, and the style and rate of changes to the material in response to weathering action, are strongly influenced by structure and composition, themselves products of the original material and diagenetic changes. Generally speaking, in compaction mudrocks and overconsolidated clays the effects of burial are rapidly lost. However, in cemented mudrocks degradation may occur over a protracted period. The release of stored elastic strain energy leads to the progressive development of fractures with little initial impact on the intact mudrock between the fractures. Subsequently the interparticle bonding and cementation will be weakened and further fractures will form. Ultimately, the effects of burial and diagenetic changes are lost as the material becomes disaggregated and returns to a normally consolidated condition.

The fragmentation process in indurated mudrocks is strongly influenced by their degree of microfracturing (see **Figure 36.14**). In indurated mudrocks, diagenetic cements may provide a rigid framework that encapsulates strained mineral grains and crystals (see **Figure 36.12(e)**). The removal of constraints due to overburden and confinement puts this framework and diagenetic bonding under stress and with weathering-induced removal of the cement and weakening of bonds, the material is apt to slake into sand- or gravel-sized pieces (see **Figure 36.15**). In some rare cases, such as that shown in **Figure 36.16**, rock bursts in with an explosive release of energy.

36.2.4 Genetic features of mudrocks

36.2.4.1 Laminations and fissility

Laminations comprise discrete bedded units less than 20 mm thick and, as **Figure 36.12(c)** illustrates, the units may comprise horizons of differing grain size, or they may be successive units of the same grain size. Mudrocks that possess a laminated structure usually display anisotropy such that they break preferentially along the laminae. The fact that a mudrock can be split into thin laminae defines it as fissile. In some clay-rich mudrocks, fissility is enhanced by a preferred alignment of platy clay particles. As explained in section 36.1.1, fissile mudrocks are sometimes called shales, but because this feature may become apparent only after a degree of weathering action, it is better not to use the term in this way.

36.2.4.2 Nodules and concretions

Nodules or concretions of various shapes, types and sizes are commonly found in mudrocks. They consist of harder, more durable material and are formed by diagenetic processes (see section 36.2.2) in which localised precipitation of minerals has cemented the sediment. Examples are shown in **Figure 36.9**. As explained in section 36.3.1, nodules may sometimes be formed in soil horizons or weathered mudrocks.

36.2.4.3 Pyrite in mudrocks

Pyrite is an iron sulfide mineral which, although constituting only a few percent of the composition of mudrocks and

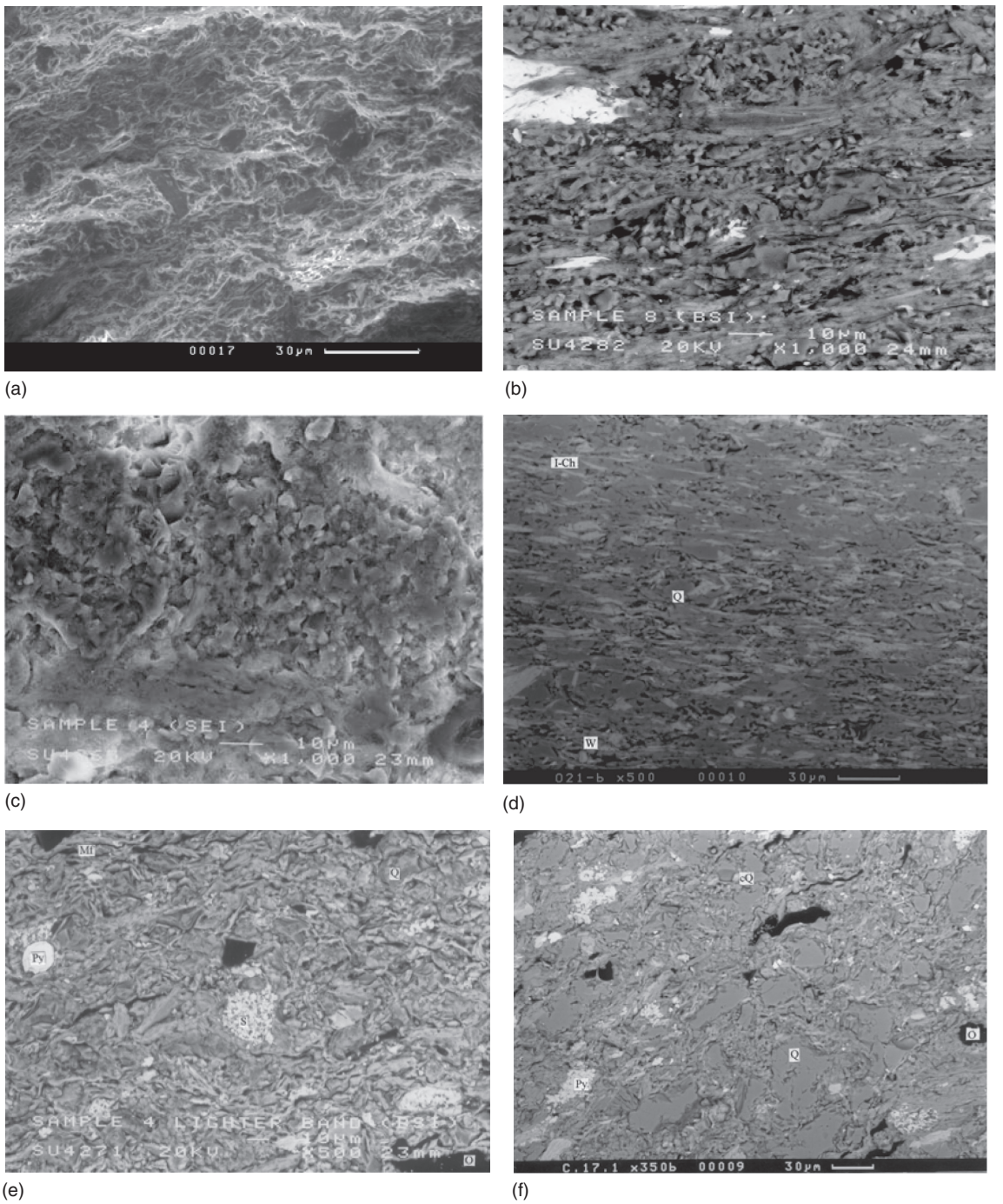


Figure 36.12 Scanning electron microscope (secondary and backscatter) images illustrating the effects of compaction and recrystallisation on the textures of mudrocks. (a) Random orientation of clay particles with wrappingaround silt-size quartz grains in Jurassic Lias Clay. (b) Preferred alignment of grains in fissile mudrockfrom the Carboniferous Upper Coal Measures. (c) Thin lamination of silt grade quartz in CarboniferousLower Coal Measures laminated mudrock. (d) Evidence of recrystallisation and cementation inOrdovician argillite. (e) Randomly orientated clay minerals in tightly compactedwell indurated massive mudrock from NE England. (f) Cemented mudrock from Scotland. Bright areas:pyrite, medium bright areas: calcite cement; grey: clay; black: organic matter.

overconsolidated clays, can give rise to several types of problem, as further detailed in sections 36.4.2 and 36.4.6. It is particularly associated with dark coloured organic-rich mudrocks. As Czerewko et al. (2003a) explain, pyrite is not the only potentially troublesome sulfide mineral, but it is the most commonly occurring one. Pyrite commonly occurs as framboids which are spherical clusters typically less than 3 µm across (see **Figure 36.8(a)**). Due to their great collective surface area,

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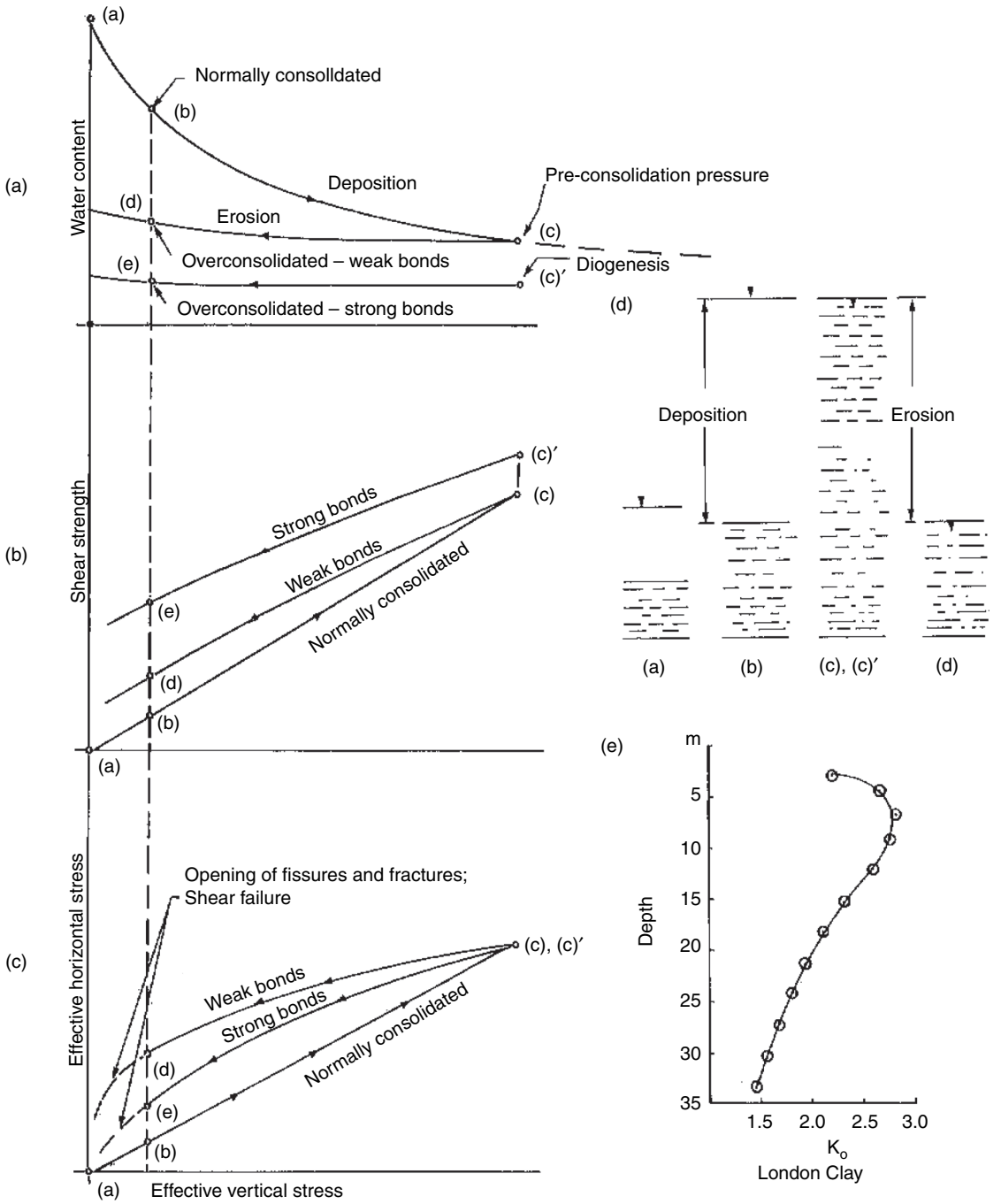


Figure 36.13 Changes in water content, shear strength and horizontal stress due to loading and unloading of sediments
Reproduced from Cripps and Taylor (1981) © The Geological Society of London

pyrite framboids are very reactive in oxygenated and humid weathering environments. **Table 36.2** lists UK sediments that contain pyrite. It is particularly common in marine overconsolidated clays and mudrocks of Jurassic, Cretaceous and Cenozoic ages which occupy about 25% of the land surface of southern and eastern England, and small areas of Wales and Scotland.

Diagenetic processes (referred to in section 36.2.2) progressively convert open textured framboids into octahedral or cubic crystal forms up to several millimetres across. At each stage of the process the pyrite becomes more resistant to oxidation. Exhumation of mudrock formations allows them access to air and water, so the pyrite may become oxidised by the process illustrated in **Box 36.1**.

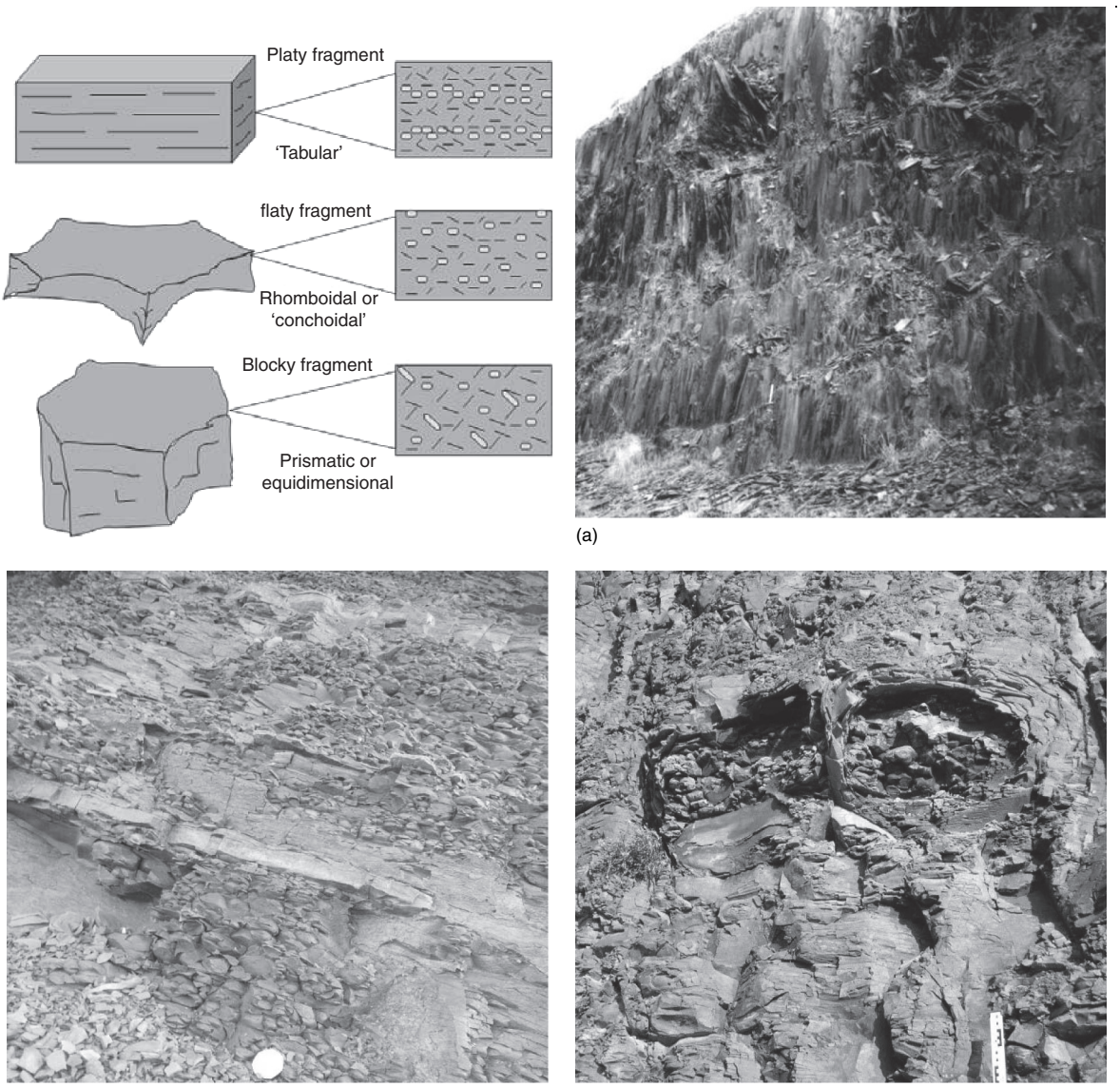


Figure 36.14 Effect of rock microstructure on the morphology of the weathered product. (a) Platy or 'fissile' (Ordovician, West Wales). (b) Flaky (Carboniferous Coal Measures, South Yorkshire); (c) blocky (Carboniferous Coal Measures, South Yorkshire)

Exposure to oxygen and water facilitates slow chemical oxidation of pyrite according to reaction 1. This reaction is self-limiting at pH < 4. However, ferric iron – which is produced by the activities of sulfate-reducing bacteria – acts as an oxidant to pyrite. The oxidation reaction is exothermic and the conditions become more acidic due to the release of $H^{\cdot}$ ions. As the bacteria are at their most active in warm (~30ºC) acid conditions (pH ~ 3), the reaction proceeds 3 to 100 times faster than for purely chemical oxidation.

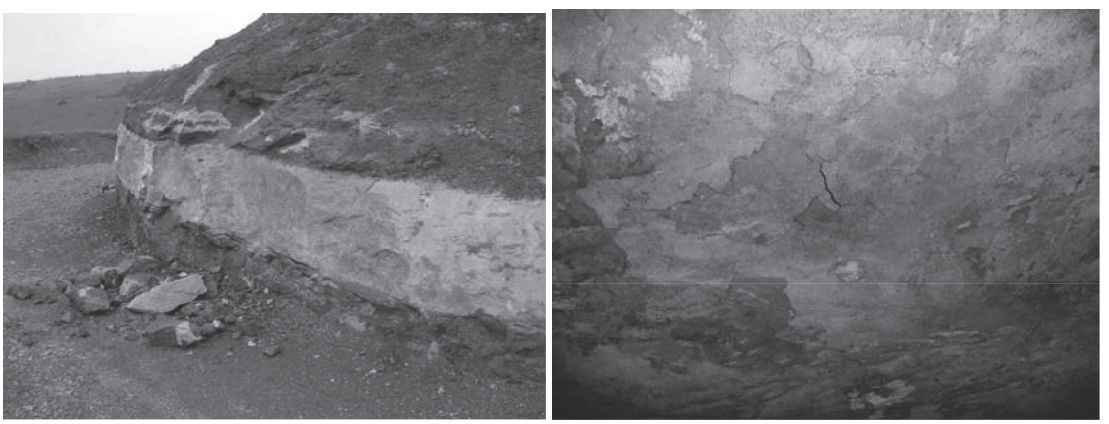
If there is insufficient water present for the products of pyrite oxidation to remain in solution, the reaction becomes arrested with the formation of hydrated iron sulfate. In fact the process can have a desiccating effect on clays (see Cripps and Edwards, 1997). Although iron sulfate is stable under acidic conditions, flowing water removes the soluble minerals and sulfuric acid in solution, and produces highly aggressive acid pore waters as are typified by the highly polluting acid mine discharges (AMD) from flooded mines (Banks, 1997) and mine-waste spoil tips.

With abundant water in the system, the acidity is moderated to pH > 4 and iron oxyhydroxide ($Fe(OH)_3$), which is a distinctive highly insoluble orange-brown residue known as ochre, is usually produced. The sulfuric acid generated in the oxidation process is biotoxic and highly corrosive to metals, concrete and rocks. As a strong leachant, its effect is to release potentially harmful elements, including heavy metals, into the environment.



(a) (b)

Figure 36.15 Examples of slaking on exposure of Carboniferous Coal Measures mudrocks. (a) Indurated mudrock slaked to gravel size domains after 2 months exposure. Carboniferous Coal Measures mudrocksfrom north Derbyshire . (b) Rapid slope deterioration due to slaking in Coal Measuresmudrock, Sheffield



(a) (b)

Figure 36.16 Effects of stain energy release in mudrocks: (a) failure of cut slope in Mercia Mudstone, East Midlands, UK; (b) failure of roof in mine – coal measures, North England

If calcium carbonate is present, it will react with the acid to produce gypsum ($CaSO_4.2H_2O$) or selenite, as **Figure 36.17** shows. Once selenite crystallises it reluctantly redissolves, thus acting as a nucleus for further precipitation of calcium sulfate. The conditions remain acidic (pH < 2.5) if the amount of calcium carbonate is low – so acid attack on illite and albite feldspar produces jarosite ($KFe_3(SO_4)_2(OH)_6$) and alunite ($KAl_3(SO_4)_2(OH)_6$) respectively. On the other hand, a high calcium carbonate content (see **Table 36.2**) renders the environment less hospitable to microbes, so the rate of reaction decreases.

36.3 Engineering properties and performance

36.3.1 Physical properties and characteristics of mudrocks

Mudrocks display a wide range of physical properties and characteristics depending mainly on composition and structure, which depend on the origins and history of the materials (see sections 36.2.2 and 36.2.3). In terms of engineering performance, mudrocks can be divided into three main types, depending upon the extent of intergranular cementation and recrystallisation. Although the burial history rather than age

Stratigraphic unit	Pyrite content (% FeS_2)	Carbonate mineral content (%)	Carbonate mineralogy
Eocene			
London Clay – blue grey fissured silty clay	0.6–4	0.4–5.2	Calcite, dolomite
Basal London Clay – sandy clay	2.5–41	N/A	Calcite
Palaeogene			
Lambeth Group – laminated and shelly clays	0.4–19.2	0–18	Calcite
Lambeth Group – mottled clays	<0.2	0	Absent
Cretaceous			
Weald Clay	0.6	1–6	Calcite
Gault Clay	0.7–1.0	0.7–33	Calcite
Jurassic			
Ancholme Clay	0.2–4	2.7–4.3	Calcite
Kimmeridge Clay	0.4–4	8–36	Calcite
Oxford Clay	3–15	4.5–19	Calcite
Whitby Mudstone	3–17	1–16	Calcite, siderite
Lias Clay	1–8	2–49	Calcite
Carboniferous			
Coal Measures – claystone	0.1–6.9	2.2–3.2	Siderite, calcite
Coal Measures – mudstone	1.2–8.2	0–9	Calcite, siderite
Coal Measures – siltstone	0–2.3	0–37	Siderite
Limestone Rock	0.2–10	N/A	Calcite
Culm Measures	2.4	N/A	N/A
Namurian Mudstone	0–6	0–0.5	Calcite
Devonian			
SW England – mudrock - argilite	0–4.3	0–12	Calcite, dolomite, siderite
Silurian			
Wales & Southern Uplands, Scotland - metamudrock and argilite	0.1–4.2	0–2.2	Calcite, siderite
Ordovician			
Wales – mudrock – slate	0.4–7.1	0–2.7	Calcite, siderite
Cambrian			
North Wales – slate	0.5–5.4	0–0.8	Dolomite

N/A: not applicable

Table 36.2 Pyrite content range and significant carbonate mineralogy of selected UK mudrock formations
After Czerewko and Cripps et al. (2003a)

exerts a strong control over the state of induration, a general trend of increasing induration and strength with age occurs in UK mudrocks (see Taylor, 1984), as shown below:

Overconsolidated clays	Cenozoic, Cretaceous and most Jurassic clays/ mudstones
Weaker mudrocks	Lias (Lower Jurassic), Mercia Mudstone (Triassic), Permian and some Coal Measures (Carboniferous) mudstones
Stronger mudrocks	Carboniferous mudstones, Devonian and older mudrocks/metamudstones

In some cases, well cemented mudrocks may occur in younger sequences. Conversely, weathering action and/or fracturing can result in weaker materials being present in older formations. **Table 36.3** presents strength and also compressibility and plasticity data typical of UK unweathered mudrocks of different types. The performance of these materials is discussed below. Further data on other formations are given by Reeves *et al*. (2006).

Published values of geotechnical parameters such as those in **Table 36.3** are site-specific and should be used solely for guidance for preliminary designs and to assist with appropriate scoping of investigation and testing programs. This is because the values obtained from geomechanical tests depend on the

sampling, test conditions and other factors. Because weathering has an extremely variable impact on mudrocks, only very general guidance about the likely effects of weathering can be given. The remarks given below are intended only to demonstrate very broad differences in the ways the parameters change for different lithologies.

36.3.1.1 Compaction mudrocks and over-consolidated clays

Only relatively weak interparticle bonds are formed during the burial of compaction mudrocks, so in many engineering situations they have little impact and effective stress has a larger influence on interparticle friction forces. In the UK, as shown in **Figure 36.4**, a number of important overconsolidated clay formations of the Jurassic period and younger occur widely in the east and south of England. The older formations, such as the Lower Jurassic Lias Clay and siltier parts of others, such as Oxford Clay and London Clay, display properties which place them on the boundary between hard soils and weak rocks.

The clay within about 3 m of the ground surface will typically be in partly saturated (partly desiccated) condition and will tend to be stiffer than the material it overlies. Wetting reduces the effective stress and causes rapid softening of the clay, which may then break down into individual grains or partly bonded domains. In compaction mudrocks, the effects of overconsolidation are rapidly destroyed by weathering processes and breakdown is likely within an engineering timescale and, in some cases, a construction timescale. The relative strength of interparticle bonding in mudrocks (which will provide an indicative rate of deterioration) may be assessed from the results of durability tests such as the static jar slake test (Czerewko and Cripps, 2001) or the dynamic slake durability test (Franklin and Chandra, 1972).

Higher values of strength and density with lower moisture content are found for the siltier and more deeply buried clays; in some cases, they are due to the presence of interparticle bonding and some cementation of particular horizons. Higher values of plasticity would indicate the presence of smectite and mixed layer clays. Weathering is typically responsible for reducing the undrained shear strength to about half of the unweathered value, whereas moisture content is increased by up to 50% and liquid limit by about 10% due to weathering processes. In parallel, density is typically decreased by about 5% and compressibility increased by about one order of magnitude.

Some formations contain systems of fissures such that the mass strength will be less than the strength of small laboratory samples. Some display anisotropy due to the presence of laminations and/or the preferred alignment of platy clay minerals. For reasons explained in section 36.2.3, the horizontal *in situ* stress can be three or four times greater than the vertical stress in the upper 10 m or so of the formation. In the UK, weathering effects generally extend to depths of 7–8 m, and in the upper few metres, deformation relieves the horizontal stress. On the other hand, joint and fissure development, which are the result

Box 36.1 Process of the oxidation of pyrite

1. Oxidation of pyrite by molecular oxygen produces ferrous iron in a purely chemical reaction. Under saturated conditions iron remains in solution:

   $2FeS_{2(s)}$ (pyrite) + $2H_2O$ + $7O_2$ $2Fe^{2+}$ (ferrous iron) + $4SO_4^{2-}$ + $4H^+_{(aq)}$.

2. Oxidation of ferrous iron by molecular oxygen to ferric iron which is a strong oxidising agent:

   $4Fe^{2+}$ (ferrous iron) + $4H^+_{(aq)}$ + O_2 $4Fe^{3+}$ (ferric iron) + $2H_2O$.

3. Further oxidation of pyrite by ferric iron which acts as an electron acceptor producing ferrous iron, sulfate anion and protons:

   FeS_2 (pyrite) + $14Fe^{3+}$ (ferric iron) + $8H_2O$ $15Fe^{2+}$ (ferrous iron) + $2SO_4^{2-}$ + $16H^+_{(aq)}$.

4. If calcium carbonate is present the acid will be buffered and gypsum and carbon dioxide will be produced. The reactions require sufficient water for the reaction products to be in solution and gypsum will be precipitated if the water is evaporated or consumed so that the solution becomes saturated in that compound

   $CaCO_3(s)$ + $2H^+_{(aq)}$ + $SO_4^{2-}{}_{(aq)}$ + $H_2O_{(l)}$ $CaSO_4.2H_2O_{(s)}$ + $C0_{2(g)}$.



(a) (b) (c)

Figure 36.17 Effects of gypsum precipitation in mudrocks. (a) Edale Shale from Mam Tor, Derbyshire, UK showing expansive gypsum crystallisation along bedding discontinuities. (b) Random gypsum crystal growth in Lias Clay destroying the structure of less indurated rock and causing volume increase. (c) Gypsum 'selenite-star' and induced microfracturing in Edale Shale from Mam Tor, Derbysire, UK producing silt size fragments

Palaeogene – London Clay: Overconsolidated clay – compaction mudrock with soil-like characteristics. When fresh, tends to be very stiff to hard, fissured blue grey locally calcareous pyritic silty clay

Id_3	I_j'	Durability	Strength c_u (kPa)	Compressibility m_v (m²/MN)	MC (%)	LL (%)	PL (%)	PI (%)	Void ratio, *e*
0–12	8	ND	70–280	0.01–0.002	19–28	50–106	22–35	40–65	0.6–0.8

Jurassic – Lias Clay: Overconsolidated clay – compaction and weakly cemented mudrock with soil-like characteristics. When fresh, tends to be very weak to weak, laminated grey locally calcareous silty mudstone

Id_3	I_j'	Durability	Strength UCS (MPa)	Strength c_u (kPa)	Compressibility m_v (m²/MN)	*E* (MPa)	MC (%)	LL (%)	PI (%)	Void ratio, *e*
2–8	6–8	ND	0.38–45	80–1200	0.002–0.004	22–35	16–22	30–63	8–55	0.5–0.7

Triassic – Mercia Mudstone: Overconsolidated clay to mudrock – compaction and cementation mudrock with soil-like and weak rock-like characteristics. Is weak when fresh, may be medium strong when cemented; red brown locally calcareous or dolomitic mudstone and silty mudstone

Id_3	I_j'	Durability UCS (MPa)	Strength c_u (kPa)	Strength m_v (m²/MN)	Compressibility	*E* (MPa)	MC (%)	LL (%)	PI (%)	Void ratio, *e*
19–81	2–7	D-ND	0.55–36	80–2800	0.008–0.03	100–1200	16–22	19–35	8–35	8–55

Carboniferous – Coal Measures: Mudstone: Coal Measures mudrocks include claystone, mudstone and siltstone; only mudstone is considered here. Tends to be mudrocks with variable properties depending on silt content and degree of cementation, but generally classed as a cementation mudrock with rock-like characteristics. When fresh is weak to strong, locally interlaminated to thinly bedded grey mudstone

Id_3	I_j'	Durability	Strength UCS (MPa)	*E* (GPa)	Is (50) (MPa)	MC (%)	MC_{ABS} (%)	E_V (%)	E_A (%)	$LL^{(3)}$ (%)
10–95	2–7	ED-ND	3.4–50.1[(1)] 33–128[(2)]	5–50	0.21–3.69[(1)] 0.6–7.2[(2)]	1.8–5.7[(1)] 1.4–2.9[(2)]	2.3–10.3	0.28–3.12	0.6–7.8	27–33

Lower Palaeozoic – Ordovician and Cambrian: Metamudstone to slate: end to be cementation deposits of moderately high strength depending on silt content and induration. Tend to be of variable properties. When fresh, are medium strong to very strong, occasionally interlaminated fissir le to cleaved grey, purple, green grey mudstone, metamudrock, to slate.

Id_3	I_j'	Durability	Strength	*E* (GPa)	Is (50) (MPa)	MC (%)	MC_{ABS} (%)	E_V (%)	E_A (%)	UCS (MPa)
98–99	1–3	ED	66–272	≥31	2.1–15.8	0.11–1.03	0.1–2.2	0.005–0.17	0.02–0.43	

[(1)]Mudrocks from England and Scotland; [(2)]Mudrocks from South Wales; [(3)]Material prepared by mechanical means from fresh mudrock; *e*: void ratio; *E*: Deformation modulus, analogous to Young's modulus but behaviour not strictly elastic, and varies with confining pressure; E_A: axial swell; ED: Extremely durable; D: Durable; ND: Non durable; E_V: volumetric swell; Is (50): point load test strength; LL: liquid limit (%); MC: Moisture content (%); MC_{ABS}: moisture absorption (%); PI: Plasticity index = LL – PL (%); PL: plastic limit; Id3: 3-cycle slake durability index value; Ij': Jar slake test index; UCS: Uniaxial compressive strength

Table 36.3 Examples of mudrock properties from a number of prominent UK stratigraphic units showing typical range of values encountered in fresh unweathered material
Data taken from Cripps and Taylor (1981), Reeves *et al.* (2006) and Czerewko and Cripp (2006b)

of overburden reduction, extends to tens of metres below the surface.

The glacial conditions that occurred during the Pleistocene Era had very important consequences on the behaviour of mudrocks throughout the UK. In compaction mudrocks and overconsolidated clays, localised zones of disturbance to near-surface strata were formed and solifluction deposits mantled most escapments and outcrops on sloping ground. The solifluction deposits contain shear surfaces that mobilise shear strength near to the residual value, so they can undergo shearing if subjected to relatively low values of shear stress. Weeks (1969) and Symonds and Booth (1971) describe hillsides that became unstable due to changes in groundwater conditions, loading or the removal of support.

36.3.1.2 Weakly cemented mudrocks

The physical and mineralogical changes that characterise cemented mudrocks also enhance durability by restricting the access of air and water to any remaining reactive clay minerals. However, because distorted mineral particles are restrained by intergranular bonds and cements, the reduction in confining stress and weathering effects results in time-dependent strain energy release. Such materials tend to undergo rapid slaking as the volume changes that accompany alternating changes in moisture content result in the breakage of bonds and interparticle cements.

One of the initial consequences of overburden reduction is the development of systems of sub-horizontal and inclined joints that facilitate air and water penetration into the rock mass.

Together with shrink–swell effects, these result in the development of further joints and microfractures which develop along bedding discontinuities and grain boundaries (see **Figure 36.14**). Generally speaking, these processes result in degradation to sand- to fine gravel-sized mineral grains. Further breakdown of the material to individual mineral grains then usually requires breakage of the interparticle bonds by a combination of fatigue failure – caused by repeated volume changes and dissolution of cements (see **Figure 36.15**).

Weakly cemented mudrocks may well be weak rocks in terms of their undrained shear strength, but they have a tendency to undergo rapid slaking when they are exposed in excavations and slopes, as seen in **Figure 36.14(b)**.

The changes in parameters due to weathering action are greater in cemented mudrocks than in compaction mudrocks, as the former are initially more indurated. However, the product materials are similar, notwithstanding that degradation is more rapid for compaction mudrocks. The moisture content of highly weathered mudstone is typically 30–35%, whereas bulk density will be 1.95–2.05 Mg/m^3, undrained shear strength 70–100 kPa, uniaxial compressive strength 10–500 kPa, and coefficient of volume compressibility 0.1–0.24 m^2/MN. Higher values of moisture content plasticity and compressibility, and lower values of strength and density are liable to be found in less deeply buried, poorly indurated clay-rich deposits, or in more indurated materials containing swelling or mixed layer clays.

Disturbance during the sampling process and sensitivity to changes in stress and moisture conditions render many weakly cemented mudrocks challenging to test and classify, which can lead to the underestimation of the *in situ* behaviour. Soil classification tests, such as particle size analysis and Atterberg limits which require destructuring of material, do not usually provide an accurate indication of its behaviour even on a small scale. For instance, many published values of high activity in Mercia Mudstone are probably the result of the underestimation of the <2 mm fraction rather than an indication that they contain high values of swelling clay minerals.

36.3.1.3 Strongly cemented mudrocks

In strongly cemented mudrocks, a skeletal framework consisting of bonded clay grains and a continuous framework of cemented grains are present with the elimination of any low-strength micro-discontinuities. In addition, stored strain energy due to the presence of distorted mineral grains will have been gradually eradicated by recrystallisation of minerals. Any swelling clay minerals will have been replaced by more stable forms. In practice, like weakly cemented mudrocks, ultimate breakdown to constituent particles is a two-stage process in which the rock mass is initially broken down into sand- or gravel-sized fragments which may then degrade into mineral grains. In strongly cemented mudrocks, the fragments are usually larger and they take longer to break down.

The liquid limit of disaggregated strongly cemented mudrock usually ranges between 35 and 45%, which reflects a predominance of illite in the clay mineral fraction. The extent of degradation due to weathering is greater than for other mudrocks. Typical values of parameters for weathered cemented mudrocks are as follows: moisture content 15–30%, liquid limit 35–45%, bulk density 1.9–2.0 Mg/m^3, undrained shear strength 15–70 kPa.

The impact of weathering on well-bonded mudrocks is particularly apparent in triaxial strength tests on fresh and weathered samples. In the results of tests by Taylor (1988) shown in **Figure 36.18**, a curved Mohr envelope typical of Carboniferous and other fresh cemented mudrocks was obtained. There is apparent cohesion at low values of confining pressure, but at higher confining stress the breakage of grains occurs. The removal of the bonding effects by weathering action results in a linear envelope with a lower friction angle typical of unbonded clays. Shearing results in a further reduction in strength to the residual value as a consequence of the orientation of clay minerals on shear surfaces.

36.3.2 Investigation, description and assessment

In this section, the techniques of investigation are reviewed, particularly with regard to some of the problems that may arise with mudrocks. Features of mudrocks that help with the anticipation and avoidance of adverse forms of behaviour are also discussed.

As explained in section 36.1.1, BS 5930 + A2: 2010 provides guidance on the procedures for carrying out ground investigations for civil engineering works in the UK. It is argued that the money spent on carrying out ground investigations is saved by avoiding additional costs due to changes and delays in the construction programme. Savings in materials and construction costs may be possible because a more refined design can be adopted. Parts of BS 5930:1999 + A2:2010 are superseded by Eurocodes: BS EN ISO 22475–1:2006, BS EN ISO 14689–1:2003 and BS EN ISO 14688–1:2002 and 14688–2:2004.

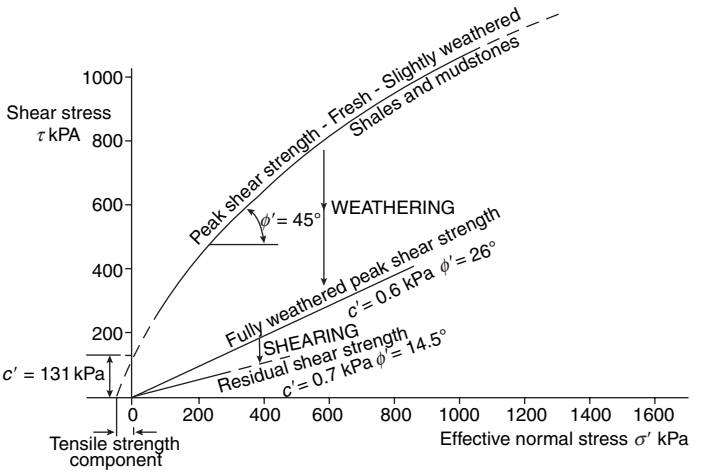


Figure 36.18 Mohr envelopes for bonded and weathered Carboniferous mudrocks

Reproduced from Taylor (1988) © The Geological Society of London

Laboratory classification tests for soils are described in BS 1377:1990. These documents specify minimum standards for commonly occurring engineering situations. So far as mudrocks are concerned, there are a number of deficiencies. Due to uncertainty of meaning, some of the terminology and methods need to be clarified.

The density of the material is an extremely useful parameter for assessing its state of induration and likely performance. As mudrocks have broadly similar compositions it may also be indicative of mineral composition and void volume.

36.3.2.1 Soil and rock

Deciding whether to base designs on soil mechanics or rock mechanics principles is not a straightforward issue for some mudrocks, and assuming a worse case would not normally provide the best engineering solution. Soil mechanics deals with material as assemblages of discrete particles or a continuum (where design is usually based on limit state modelling of a plastic material of low shear strength) and in which changes in moisture content bring about significant changes to plasticity and volume. On the other hand, the rock mechanics approach considers an intact material of a significant rigidity where the mass properties are controlled by the nature and geometry of dislocations present in the rock mass. The engineering assessment of mudrocks and clays needs to consider the most appropriate design approach, given the material and the effects on the material of the imposed stresses and the environmental conditions at different stages of the project.

At a certain stage of induration, the network of cemented grains will resist deformation such that failure of the material requires brittle fracture of the framework. Such behaviour results in fragmentation if the confining stress is low, whereas the same material can deform in a plastic manner in high confining stress conditions. As mentioned in section 36.1.2, this makes the material difficult to classify and, as these changes are progressive, the effects may vary within the formation. Norbury (2010) points out that the boundary between soil and rock is taken as an undrained shear strength of 300 kPa, which is equivalent to a uniaxial compressive strength of 0.6 MPa. He recommends the following field tests to distinguish between soils and rocks. With rocks:

- finger nail does not penetrate the surface;
- fragments behave in a brittle manner in the hands;
- material does not immediately soften or disintegrate on contact with water.

Changes with time must also be considered as some mudrocks degrade rapidly. A small change in weathering grade, which might occur during or after the construction period, has a significant impact on the appropriateness of a construction process or design. It can be very difficult to decide whether it is appropriate to regard the material as an engineering soil or as a rock; sometimes it is necessary to apply both approaches and then optimise the design so that both forms of behaviour are accommodated. Further relevant discussion is presented in the following section.

36.3.2.2 Description

In view of the high possibility of sample disturbance influencing the results of laboratory tests, it is particularly important that descriptions of mudrocks are accurate and clear. Both the material and the structures in the rock or soil mass should be recognised and correctly characterised. In practice, an accurate interpretation of laboratory test results depends on descriptions of the different materials and of their spatial relationships with each other. It is usually possible to obtain good quality undisturbed samples of either strongly cemented mudrocks, which survive the sampling procedures and preparation for testing, or compaction mudrocks, because of their low strength and plasticity.

The aim of description is to record unambiguously essential features of the material so that (a) similar material can be recognised elsewhere, and (b) accurate prediction of the engineering behaviour is possible. Norbury (2010) provides highly useful advice on the description of mudrocks and other geomaterials.

Unfortunately the job of logging cores and exposures is regarded by many as a menial task to be assigned to junior, less experienced staff. This can result in a weak link developing in the transfer of information about the ground conditions to designers and construction engineers. It is also wasteful given the high investment of time, effort and money expended on ground investigations, and the additional costs and delays that inappropriate design can cause. Therefore description of mudrocks should be undertaken in a well-informed and focused manner with thorough attention to detail. Distinguishing between soil and rock is an essential part of description as there are different approaches to describing these materials. However, as discussed in section 36.3.2.1, mudrocks can span the range from rock- to soil-like behaviour and some rock or soil masses contain both materials. It is often necessary to describe the weathered parts of a rock mass using the terminology for soils, whereas some soils contain hard inclusions that need to be described in the manner of rocks. The proportions and relationships between the different materials should also be recorded.

Although it is preferable to use a standard approach to description, according to a specific standard, it is also necessary to ensure that all information that will enable a material to be recognised (and its behaviour to be anticipated) has been recorded. The standard schemes given in BS EN ISO 14688–1:2002, BS EN ISO 14689–1:2004 and BS 5930:1999 +A2:2010 should be regarded as the minimum amount of information. The recording of additional information relating to factors controlling the behaviour of mudrocks facilitates more reliable assessments of their performance in engineering situations. This approach is supported by Franklin and Dusseault (1989); Dick and Shakoor (1992); and Hawkins and Pinches (1992) who emphasise that a thorough geological description

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Mudrock type	Average grain size < 63 μm (<50% of grains less than 63 μm in size)				
		% < 2 μm	Characteristics	Material type	
Non-indurated	s_u < 300 kPa	66–100	Smooth, soapy feel with fingers	Clay	
	σ_c < 0.6 MPa		Plastic*		
	Can be broken by hand		Some particles visible with hand lens		
	Rapid breakdown in water	33–65	Slightly gritty feel with fingers	Silty clay	
			Slightly plastic(1)	Clayey silt	
			Particles visible with hand lens		
		0–32	Gritty to touch with fingers.	Silt	
			Non-plastic(1)		
			Some particles visible with naked eye		
Indurated	σ_c 0.6–50 MPa	66–100	Smooth non-gritty feel with blade/fingers	Claystone	
	Retains form in water, but surface softens		Some particles visible with hand lens		
	Immersion in water causes slaking		Surface softens with water and water becomes cloudy		
		33–65	Slightly gritty feel with blade/fingers	Mudstone	
			Particles visible with hand lens		
			Surface softens with water and water becomes slightly cloudy		
		0–32	Gritty to touch with blade/fingers	Siltstone	
			Some particles visible with naked eye		
			Does not soften with water		
			Sparkly appearance		
				Massive	Foliated
Metamudrock – very low grade	σ_c >25 MPa(2)	66–100	Partly to wholly recrystallised	Argilite	Slate
			Grains visible with hand lens		
		33–65	Some grains visible to naked eye		
			Slate: flat platy particles may be broken from edges with blade		
		0–32	Partly to wholly recrystallised	Quartz argilite	Siltstone-quarzite
			Grains visible with hand lens		
			Grains visible to naked eye		
Metamudrock – low grade	σ_c >25 MPa(2)	66–100	Recrystallised	Argilite-hornfels	Phyllite
			Some grains visible to naked eye		
		33–65	Phyllite: silky sheen and crenulated cleavage		
		0–32	Recrystallised	Hornfels- quartzite	Siltstone-quarzite
			Grains visible to naked eye		
			Quartzite: lustrous appearance		

(1) Tests to assess plastic or non-plastic behaviour in terms of dilatancy, plasticity and dry strength are described by BS EN ISO 14688-1:2002.
(2) Strength values for unweathered (fresh) material.

Table 36.4 Identification and classification of mudrocks
Based on data from Potter *et al.* (1980) and Attewell (1997)

is the essential first stage of making an engineering assessment of mudrocks, with further classification based on the results of suitable index testing accompanied by strength and slake durability tests.

Although both BS EN ISO 14689–1:2003 and BS 5930: 1999 + A2:2010 contain deficiencies, they provide good bases for descriptions, and only the aspects requiring additional details are described below. As discussed in section 36.1.1, particular inadequacy of the standards is the choice of rock names; the scheme given in **Table 36.4**, which groups the materials into classes that relate to their engineering performance, is more appropriate.

Using **Table 36.5**, the distinction between soil and rock material is based upon the state of induration determined by the tests described by Norbury (2010) as shown in section 36.3.2.1. Rocks do not readily disaggregate when a dry sample is immersed in water and it is not possible to indent the material with a finger nail. The grain size, in particular in the silt and clay portions, is judged according to the surface texture and appearance of the material. In the case of non-indurated materials the dilatancy, dry strength and plasticity tests (see BS 5930: 1999 + A2:2010 and BS EN ISO 14688–1:2002) should be used. The distinction between indurated mudstone and metamudrock is also based on strength, where the latter is typically medium strong or stronger (UCS > 25 MPa).

According to BS EN ISO 14688–1:2002, BS 5930: 1999 +A2:2010 and BS EN ISO 14689–1:2003, information that needs to be recorded in rock and soil material should include the following aspects, although (as noted above) this should be regarded as the minimum and attention should be given to the details listed in **Table 36.5**:

- strength: very soft to very stiff for soils; extremely weak to extremely strong for rocks (**Table 36.6**);
- structure: lamination and bedding etc.; see **Table 36.7**, and in particular, the footnotes;
- colour: describe in terms of depth, chroma and hue, preferably with reference to a Munsell chart, and record mixing such as mottling or banding;
- fabric: spatial arrangement of solid grains and associated voids, such as graded or granular;
- texture: shape and size of constituent grains, including groundmass, inclusions and clasts, etc.;
- rock or soil name: in capitals such as MUDSTONE or SILTSTONE (see **Table 36.4**);
- other relevant information: includes minor constituents such as fossil debris, calcareous and ferruginous nodules, disseminated or crystalline pyrite, or iron oxide;
- geological formation: where known, such as LOWER LIAS CLAY or LOWER COAL MEASURES.

Attribute	Descriptive Adjectives
Induration	Enables decision on description as soil or rock. If resistant to slaking in water and hard – rock; if susceptible to slaking in water, deformable and 'earthy' consistency – soil. Strength depends on moisture state; dry sediment is stronger than wet, and rock strength varies with moisture content; sampling may impair strength.
Strength	Strength is designated based on the degree of induration. For soil, use field consistency values based on manual assessment, e.g. stiff; when shear strength measurements are made, use strength terms, e.g. high strength. For rock, definition based principally on manual field assessment using geological hammer and knife, may be confirmed with UCS measurement: indurated mudrocks range from extremely weak to medium strong; metamudrocks are stronger depending on weathering.
Structure	Standard terms – see **Table 36.7**. Include description of lithology and textural inter-relationship, as complex features may be present with structured strata such as 'thin beds of cross bedded' mudstone.
Colour	Use Munsell colour chart for consistency. Important for correlation; likely environment of formation and indication of likely behaviour of material, i.e. red colour – likely formation under oxidising continental environment. Most important to mudrocks is relationship between colour on the Fe^{3+}/Fe^{2+} ratio. A decrease in this ratio gives an increase in colour from red → green → grey (more Fe^{2+} indicates the presence of pyrite). Organic carbon controls colour: <0.2–0.3%C = light-grey to olive grey; 0.3–0.5%C = mid-grey; >0.5%C = dark-grey to black.
Accessory minerals	Calcareous (slightly to very) – based on level of effervescence when assessed with HCl). May also be carbonaceous, dolomitic, ferruginous, glauconitic, gypsiferous, pyritic, micaceous, sideritic, phosphatic, etc..
Rock name	See **Table 36.4.**
Additional information	Presence of fossils – record type (generic such as bivalve and retain for identification), abundance, condition, orientation. Inclusions – nodules (with mineral type and details); gravel, sand, silt partings or pockets, etc.
State of weathering	Alteration seen as distinct discolouration, significant strength reduction to discontinuities and presence of lithorelicts (note orientation). Standard Eurocode 7 approach too limited for mudrocks, use specific schemes where available as guides, i.e. Lias and London Clay, Mercia Mudstone.
Fractures	Use EC7 standard terms and procedures. For rock supplement with details such as nature of fragmentation, e.g. conchoidal, hackly, brittle, splintery, slabby, fissile.

Full engineering descriptions of the intrinsic condition and mass properties using descriptive adjectives provided in EC7 (BS EN ISO 14688–1:2002; BS EN ISO 14688–2:2004; BS EN ISO 14689–1:2003, supplemented with additional information to assist assessment.
This list offers limited guidance for key features particular to mudrocks.

Table 36.5 Guide to the description of mudrock features

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Term	Strength	Description
Strong mudrock	σ_c 50–100 MPa	Can only be scratched by knife or pick end of a geological hammer, and can only be broken with more than one firm hammer blow.
Medium strong mudrock	σ_c 25–50 MPa	Can be deeply scored by a knife or pick end of a geological hammer, and a thin slab can be broken by heavy hand pressure. Specimen is readily fractured with a single firm blow of geological hammer or split with a knife blade. Cannot be peeled with a pocket knife.
Weak mudrock	σ_c 5–25 MPa	Small gravel-sized fragment can be deformed with heavy finger pressure, shallow indentations readily made by firm blow with point of geological hammer. Can be peeled by a pocket knife with difficulty.
Very weak mudrock	σ_c 1–5 MPa	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife.
Extremely weak mudrock	σ_c 0.6–1 MPa	Can be indented by thumbnail.
Extremely high strength clay	s_u 300–600	Field description will generally be as a 'very stiff clay'. Crumbles, does not remould, can be indented by thumbnail.
Very high strength clay	s_u 150–300	Determine by testing – field description as 'very stiff clay'.
High strength clay	s_u 75–150	
Medium strength clay	s_u 40–75	Field description will generally be as a 'stiff clay'. Crumbles, breaks, remoulds to lump.
Low strength clay	s_u 20–40	Field description will generally be as a 'firm clay'. Cannot be moulded, rolls to thread.
Very low strength clay	s_u 10–20	Field description will generally be as a 'soft clay'. Moulds by light finger pressure.
Extremely low strength clay	s_u <10	Field description will generally be as a 'very soft clay'. Extrudes between fingers.

σ_c uniaxial compressive strength.
s_u undrained shear strength; $\sigma_c = 2 \times s_u$. Extremely high strength clay soils and extremely weak mudrocks have similar characteristics - choice of term depends on the geological setting and engineering situation.
Field consistency descriptions for clays and strength tests descriptions should be used separately and treated as mutually exclusive.

Table 36.6 Strength criteria for mudrocks

The second part of the description deals with the structure of the rock or soil mass, and includes the following characteristics:

- weathering: description is required of observed material changes including colour changes, differences in strength and fracture state, as well as the presence and character of any weathering products. Where possible, a weathering classification scheme should be used to denote the degree and style of weathering (see **Table 36.8(a) and (b)**);
- discontinuities: information should be recorded about the distribution and features of discontinuities including the type, orientation, spacing, geometrical form, shape, rugosity (surface roughness) and evidence of alteration of rock adjacent to the discontinuities, as well as the nature and thickness of any infill material.

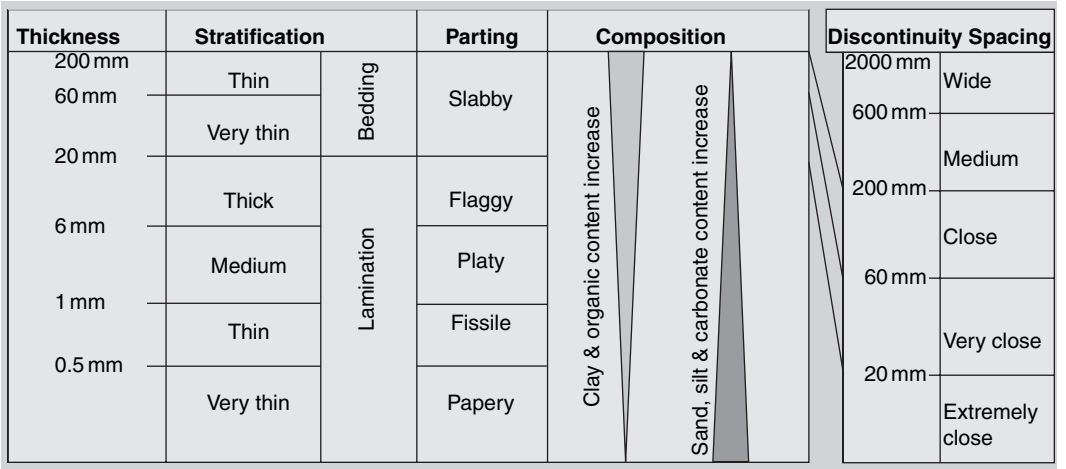
The level of detail it is possible to record depends upon the scale of exposure; for example, it may not be possible to comment on the planarity of joints in borehole cores. **Table 36.5** provides guidance about the terms which should be used in the description of mudrocks. The strength categories in **Table 36.6** are based on measured *in situ* or laboratory shear strength values, whereas the strength assessed using the manual (or hand) tests should use the consistency terms 'very soft' to 'very stiff'. Details of manual tests are given in BS EN ISO 14688–1:2002 and classification based on these tests are presented in BS EN ISO 14688–2:2004. For rock descriptions, guidance is provided in BS EN ISO 14689–1:2003, but as some of these do not give reliable results for mudrocks, the tests detailed in **Table 36.6** are usually more helpful.

Although BS EN ISO 14688–1:2002 and BS EN ISO 14689–1:2003 provide schemes for classifying the spacing of structural features, these lack subdivisions within the thinly and thickly laminated categories that are useful for assessing the potential style and rate of weathering degradation.

The colour of rocks and soils depends on whether the surface is wet or dry. Generally speaking, the material should be described in the condition it was in when it was taken from the ground – but this should be stated. Care must be taken to ensure that the surface of the sample is clean of coating materials which may be different colours from the bulk of the material. Samples should be broken open to facilitate a description of both the interior and exterior. In addition, if there are any changes in the appearance of broken surfaces some hours later, they should be recorded.

Texture of soils and rocks refers to the relationships between the constituent grains, including the variation in size. Mudrocks tend to have a fine granular texture, implying that the main constituents are detrital grains that are cemented together. The particle shapes and crystallinity should be recorded, although this is often impossible even using a hand lens. Details of the composition should also be recorded, for example the presence of organic material (by visual assessment or smell), or carbonate (as indicated by effervescence when hydrochloric acid is applied), using the categories slightly, highly, etc., according to the criteria given in BS EN ISO 14688–1:2002, BS EN ISO 14689–1:2003.

In some cases, it is best to describe the material rather than attempting to determine an accurate geological name. The rock or



Mudrocks that occur without laminations or bedded stratification re described as massive.
Textural features should be used as *descriptive adjectives* when describing mudrocks and not for naming rocks similar to generic schemes used for other rock types, as this may be confusing and ambiguous; for example 'shale' can mean a fissile, or laminated, or massive mudstone or claystone; in preference, use the adjective as a prefix such as fissile claystone, laminated mudstone.
Stratification is a structural feature concerned with the inter-relationship of textural features and lithology caused by differences of grain size, fabric and composition; partings is the tenacity of a mudrock to split along lamination, bedding or mineral orientation, which is greatly enhanced by weathering; whereas discontinuities are mechanical breaks or fractures with low to no tensile strength.

Table 36.7 Description of structural features of mudrocks (stratification, partings and discontinuities)
Data taken from BS EN ISO 14689–1:2003, and Potter *et al.* (1990)

soil type should always be written in capitals, which implies that the rest of the description should be presented in sentence case.

The presence of constituents that are not implied by the rock name should be specified to aid recognition of the material. The description should also include details of observable features not already covered. Examples would include the presence of fossils, nodules, inclusions, etc. Where it is known, the geological formation name should be specified.

The presence of pyrite is a possible adverse feature of mudrocks that should be considered in the description of the materials. Characteristics of mudrocks that make the presence of pyrite more likely are:

- dark coloured (grey or dark grey), organic-rich mudrock or clay;
- finely disseminated gold coloured crystals, greenish grey hue or orange-brown and brown staining;
- sulfurous odour when hit or scratched;
- clear or white-grey crystals on surfaces which can be scratched and broken with a finger nail, or a white surface coating when the material is dried (liable to be gypsum/selenite).

Recording the effects of weathering is a highly important aspect of the description of a rock or soil mass. The appearance of the material should be recorded, particularly any changes in colour, strength or fracture spacing that are a consequence of weathering processes. This is difficult where the material has not been viewed in its fresh state for comparison. If possible, the condition of the rock or soil mass should be classified. Although weathering commences along discontinuities in mudrocks, there also tends to be an impact on the intervening blocks, whereas in stronger rocks there is usually little effect on these – so that core stones are formed. Unfortunately the Eurocode schemes are lacking in this regard and although BS 5930: 1999 + A2:2010 provides a comprehensive scheme, it is suggested that for mudrocks the schemes presented in **Table 36.8** (a) and (b), where (a) provides a scheme for non-indurated mudrocks and clays and (b) provides one for indurated mudrocks and metamudrocks should be used.

The features that need to be recorded in the description of structures should include, where possible, the origin of discontinuities. Where this is not discernable, the term 'fracture' should be used. In some mudrocks, especially weakly cemented types, it can be difficult to distinguish between *in situ* and induced discontinuities, where the latter have been formed during or after removal from the ground. Stress relief, change in moisture content, and physical disturbance may all result in such fracturing. Usually such fractures have a fresher and cleaner appearance than *in situ* ones, but this is not always the case.

Figure 36.14 demonstrates how the characteristics of the weathered material can be used to deduce the internal structure of the material. Platy fragments will be produced in a rock possessing tectonic cleavage or a laminated structure; flaky and blocky fragments are produced by a uniform distribution of the minerals in the rock. The bedding is thicker in the latter case.

36.3.2.3 Durability

Durability is a measure of the rate that a rock will degrade when exposed to weathering action. Compared with most other types of rocks, mudrocks degrade rapidly. Hence significant changes to geotechnical properties occur on timescales of significance to

Classifier	Nature	Typical Characteristic	
Unweathered	Fresh	No visible sign of weathering. Typically grey colour, clean fissured clay or mudrock.	Pyrite
	Faintly Weathered	Trace of brown surface discolouration on discontinuities otherwise intact fissured clay or mudrock	
Partially Weathered	Slightly Weathered	Fissures more closely spaced, slight strength reduction evident as surface softening some bedding disturbance. Discontinuity surfaces with pervasive brown discolouration & slight penetration typically 1-2 mm. Rare selenite crystals on discontinuity surfaces	Increasing Oxidation
	Moderately Weathered	Fissures more closely spaced, slight strength reduction evident as surface softening, some bedding disturbance. Discontinuity surfaces with pervasive brown discolouration & slight penetration typically few mm to cm with centre of fissure bound block still grey Occasional selenite crystals.	Selenite
Distinctly Weathered	Highly Weathered	Fissure spacing much closer with lensoidal fissuring – development of matrix bound regular angular block & platy orientated 'lithorelicts' & matrix as few mm to cm of claysilt on discontinuities, with occasional polished fissures or gleying. Material is generally brown to grey brown with completed discolouration – occasional fissure bound blocks with trace of grey core. Selenite crystals common in matrix & blocks.	Oxidation complete increase in reduction producing gleying
Destructured	Completely Weathered	Greatly weakened material with numerous horizontally aligned ordered lithorelicts in clay-silt matrix. Bedding disturbed. Generally brown & Grey brown may have light grey mottling. Fissure blocks & lithorelicts brown throughout. Selenite crystals common.	
		Greatly weakened material comprising numerous randomly orientated lithorelicts in a clay silt matrix. Lithorelicts are completely discoloured to brown or grey brown and softened. Gypsum crystals common. Bedding heavily disturbed.	
Reworked	Completely reworked or residual soil	Unbound sediment with occasional randomly orientated predominantly sub-angular to sub-rounded lithorelicts and occasional foreign material. No original bedding or structure remaining. Generally mottled brown, orange brown & light grey. Rare selenite crystals may be present but generally absent.	Surface water infiltration - leaching

Table 36.8(a) Weathering classification and description for non-indurated (compaction mudrocks) or overconsolidated clays

engineering designs, whilst for some low-durability mudrocks, changes may occur within the construction period. In the process, a seemingly competent rock material becomes transformed into clay or silty clay, whereas a rock mass may well become degraded into sand and gravel-sized fragments so thus there is a serious loss of strength and increase in compressibility.

The main process of degradation of mudrocks is slaking, which occurs due to alternating changes in water content. Because volume changes to clay minerals are greater for swelling clay minerals, mudrocks containing smectite or mixed layer smectite–illite are particularly vulnerable to this type of degradation. Pore water suctions, produced as water is drawn out of narrow pores during drying cycles, also cause shrinkage. This effect is greatest in fine-grained rocks, especially where the pore size has been reduced through a certain degree of compaction and cementation. Taylor (1988) attributes the breakdown of mudrocks in colliery spoils to the process of air breakage – resulting from high pore air pressures produced as water enters narrow capillary pores during the saturation of mudrocks. The shrinkage and swelling of the rock, associated with changes in moisture content, stress the intergranular bonds and partly cemented skeletal framework possessed by weakly cemented mudrocks such that breakage predominantly occurs along existing structural weaknesses within the material. This causes the rock mass to break down into sand or gravel-sized pieces, as illustrated in **Figure 36.15**.

Durability tests are designed to provide both qualitative and quantitative assessments of the susceptibility of the rock to degradation processes. Ideally, to provide accurate guidance about the performance of a material, the test used should simulate the dominant degradation processes to which it will be subjected in the engineering environment. Unfortunately, only the slake durability test described by Franklin and Chandra (1972) and ISRM (1979) has been standardised. The test entails subjecting samples to both wetting and drying action and mechanical disturbance. The degree of degradation is assessed by the proportion of particles of less than 2 mm produced by the degradation process expressed as a percentage of the original dry mass of the sample. In the standard test, the number of cycles of drying and slaking (n) equals 2, where $Id_2 > 90\%$ distinguishes non-indurated from indurated mudrocks. However, Taylor (1988) suggested that the third cycle durability value $Id_3 > 60\%$ is a more reliable indication of this boundary; this was later corroborated by Czerewko and Cripps (2001). Two of the drawbacks of this approach

Classifier	Nature	Typical Characteristics
Unweathered	Fresh	No visible signs of weathering or alteration. Primary colours of dark grey, grey, purple, red brown, green grey.
	Faintly weathered	Colour of original material is changed – Discolouration tends to be confined to major discontinuity surfaces – the colour change and extent of discolouration should be described. Generally change to light brown, range brown, light brown and red brown discolouration.
Partially weathered	Slightly weathered	Colour of original material is changes – Discolouration tends to be confined to discontinuity surfaces with pervasive penetrative discolouration. Discolouration should be described and any evidence of material weakening along discontinuity should be described and quantified where possible.
	Moderately weathered	Considerable weakening with alteration & comminuition to matrix of clay and silt penetrating in from discontinuities accompanied by penetrative discolouration. Rock framework intact with <50% of material decomposed. Rick framework generally locked 7 controls strength & stiffness. Matrix affects permeability.
Distinctly weathered	Highly weathered	Generally >50% of rock material decomposed. The rock framework may contribute to strength but weathered decomposed material forming the matrix controls stiffness & permeability – Consider structural design based on Soil Mechanics principles.
Destructured	Completely weathered	Rock material largely decomposed and considerably weakened. The weak 'soil-like' material controls behaviour. Lithorelicts may be present with random orientation. Hard corestones/concretions may be present and affect construction – Consider structural design based on Soil Mechanics principles.
Reworked – Residual soil	Residual/Reworked soil	Unbound soil material comprising of clay and silt with occasional or rare randomly orientated lithorelicts and foreign material. No original bedding, structure or fabric preserved. Properties of soil, moderate compressibility, low permeability, low to moderate soil strength.

Table 36.8(b) Weathering classification and description for cementation or rock-like mudrocks

are that: (a) complete drying of the material in the test does not simulate the more subtle changes in moisture content that occur in weathering environments, and the degradation to particles larger than 2 mm is not directly assessed; (b) there is a lack of sensitivity in compaction mudrocks and some weakly cemented mudrocks, which would make it preferable to carry out static slaking tests. Drawback (a) can be overcome by carrying out plasticity testing on the <2 mm fraction. Difficulty (b) can also be overcome by carrying out plasticity testing on the <2 mm fraction and/or by particle size analyses of the drum-retained fragments. Regrettably these important aspects of durability assessment are frequently omitted.

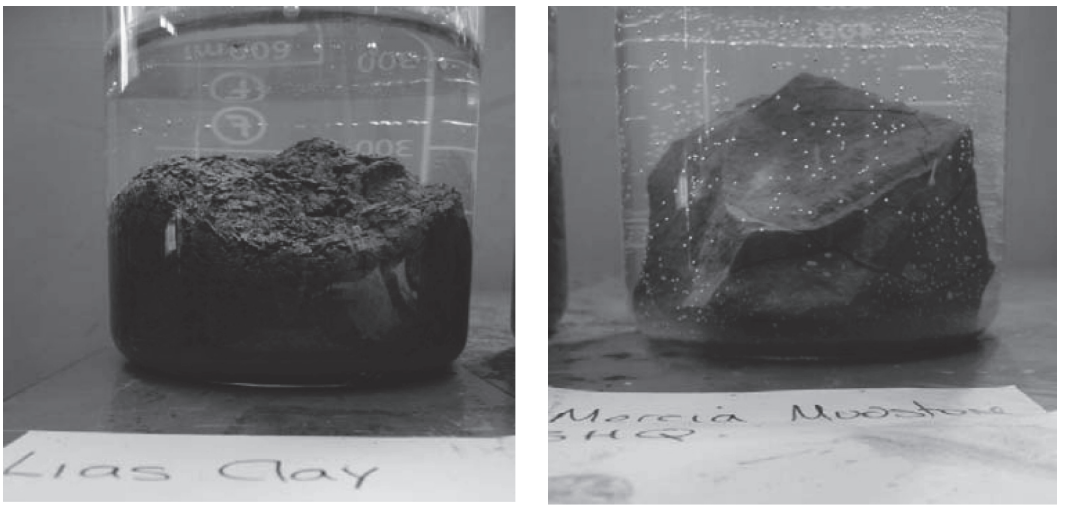
The static jar slake test procedure was advocated by Lutton (1977) and further developed by Czerewko in 1999, (see Czerewko and Cripps, 2001). Following immersion of an oven-dried sample in water, the degree of fracture development and of slaking are monitored over a 24-hour period. **Figure 36.19** shows the results of jar slake tests on two mudrocks of contrasting durability. Experience with a wide range of UK mudrocks and using strength (UCS), static jar slake (I_j') and dynamic slake durability (Id_3) test procedures, suggests that UCS = 5 MN/m^2, $Id_3 > 60\%$ and $I_j' < 6$ enable bonded and non-bonded mudrocks to be distinguished from each other, as detailed in **Table 36.9**.

The likelihood of rapid degradation of mudrocks and the size of the breakdown products may also be ascertained by observing samples or outcrops that have been subjected to short periods of exposure (see **Figures 36.14** and **36.15**). For example, **Figure 36.20** shows the complete degradation of Lias mudstone over a period of 41 days.

36.3.2.4 Investigation techniques

Sample disturbance is a significant problem with mudrocks, especially weakly cemented ones which can become distorted by the process of taking or extracting samples. They also tend to be vulnerable to volume changes – in response to removal of overburden pressure and changes in moisture content. The use of wire-line coring, triple-tube core barrels or plastic core-lining tubes in double-tube core barrels greatly enhances the recovery and quality of cores. Other methods for obtaining undisturbed samples of mudrocks, which are potentially suitable for strength and compressibility testing, are outlined in **Table 36.10**.

As explained in BS EN ISO 14688–1:2002 and BS EN ISO 22475–1:2006, the undrained shear strength and effective stress shear strength parameters may be determined by triaxial testing, and consolidation and swelling characteristics C_c – compression index; C_s – swelling index; m_V – coefficient of volume compressibility; c_V – coefficient of vertical consolidation can be determined by oedometer testing. The relevant test procedures are given in BS 1377:1990 and are described by Head (1994, 1998). Rock-like materials are classified using geomechanical rock quality classification systems (see Barton *et al.*, 1974; Bieniawski, 1976) which are based on intact uniaxial compressive strength values and mass discontinuity characteristics, usually from detailed descriptions of exposed rock or good quality borehole cores. Additionally, the point load strength (PLS) may be determined on cores or samples from trial pits or exposures. It is likely that point load testing will be the only viable means of strength determination in weakly cemented mudrocks.



(a) (b)

Figure 36.19 Behaviour of non-durable and durable mudrocks to water immersion in jar slake tests. (a) Lias Clay from East Midlands, England: rapid slaking after 3hr immersion to sand-to gravel-sized material. (b) Durable Basal Mercia Mudstone from East Midlands, England showing no slaking after 24h immersion in water.

Slake durability – Id_s (%)	Jar slake – I_j'	Compressive strength – UCS (MPa)	Classification
<60	6–8	<5 (≤ very weak)	Non-durable mudrock
60–80	3–6	5–100 (weak to strong)	Durable mudrock
80–97	1–3		Extremely durable mudrock
>97	1–2	>100 (≥ very strong)	Metamudrock

Table 36.9 Durability classification of mudrocks for engineering purposes

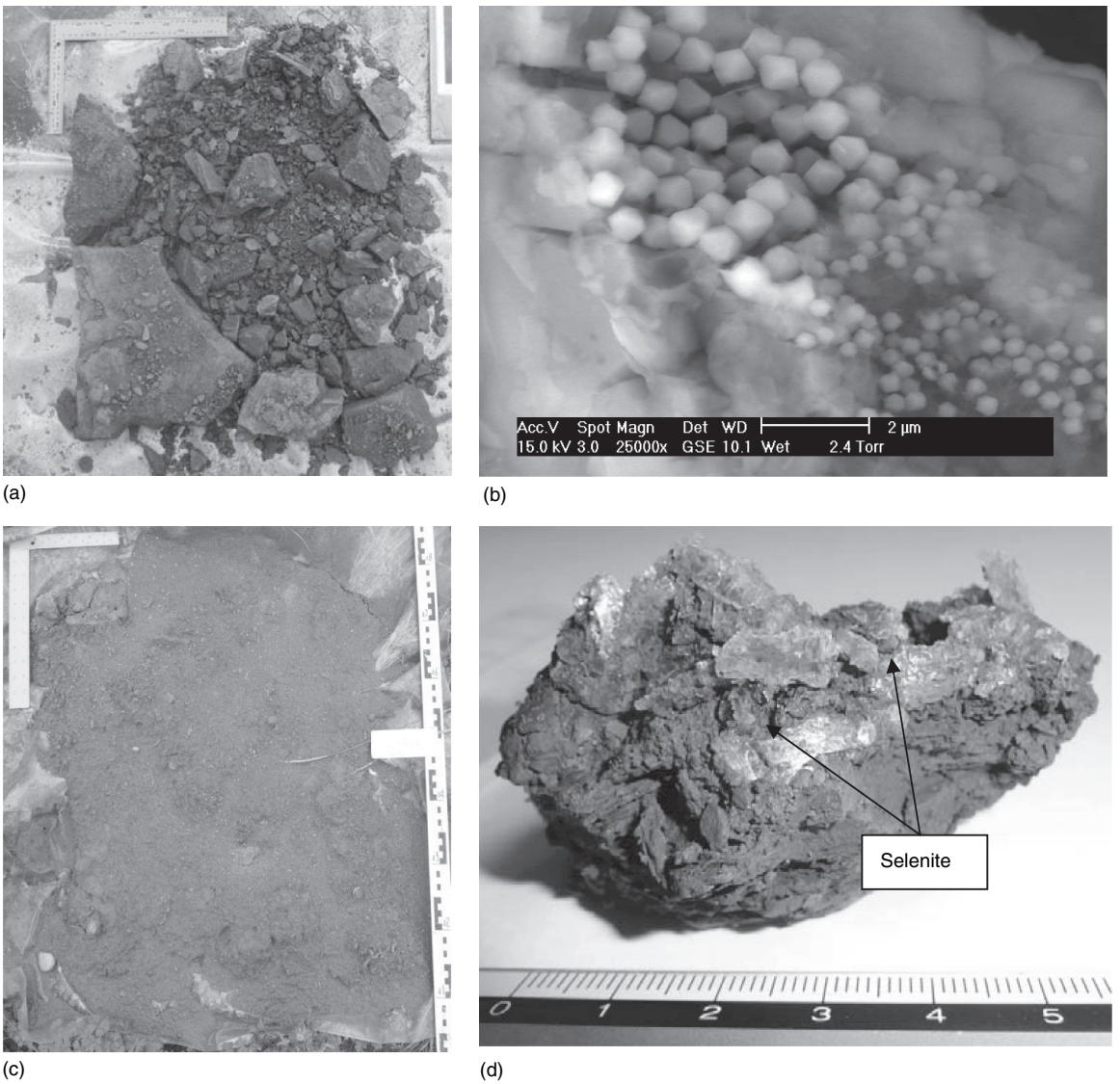
The slake durability test can be used to determine the rate and extent of breakdown included in these rocks. Gamble's (1971) slake durability–plasticity scheme is the most suitable classification for compaction mudrocks; in low durability mudrocks, the slake durability–strength schemes of Grainger (1984) and Taylor (1988) are more suitable; finally, the strength–swell coefficient approach of Olivier (1976) is best for medium strong to strong mudrocks. Triaxial testing of rock to determine shear strength (c' and φ') and elastic deformation (Young's modulus, E; Poisson's ratio, ν) parameters necessitate the preparation of intact samples which will not usually be possible in all but the strongest and most intact mudrocks.

An alternative approach is to estimate strength and deformation parameters from comprehensive descriptions and use of index testing. Various schemes are provided by Hoek and Brown (1997), Hudson and Harrison (2000), Wyllie and Mah (2004). *In situ* pressuremeter tests avoid problems with obtaining samples, but disturbance of borehole walls can still give unrepresentative results. However, examination of excavations and, if possible, boreholes by the use of CCTV, may well be beneficial for obtaining a more realistic assessment of the rock–mass condition.

36.3.2.5 Assessment of pyrite

As explained in section 36.2.4.3, pyrite is a commonly occurring (but minor) constituent of mudrocks that usually has a negative impact in engineering applications. As **Figure 36.8** illustrates, the mineral occurs in a different number of forms, depending upon the state of induration of the rock (see section 36.2.2) and other factors. It can be widely and erratically distributed through the deposit which makes it difficult to detect and to estimate the total pyrite content. Only in the crystalline form is it easy to recognise in hand specimens; furthermore, in this form, it is sufficiently hard to scratch glass and produces a slightly sulfurous odour when struck. Czerewko *et al*. (2003b) explain that other sulfur minerals may also cause problems in engineering situations and they provide guidance concerning the identification and evaluation of the various forms of sulfur.

In compaction mudrocks, pyrite usually occurs in an indiscernible, framboidal or fine-grained dark coloured form (see **Figure 36.8(a)**) which may be a dark greenish grey colour with slightly sparkling appearance and may, when rubbed or hit, produce a slightly sulfurous odour. In this condition, to achieve certain identification requires the use of optical or electron microscopy or X-ray diffraction analysis (see Reeves *et al*., 2006). These methods will facilitate a definite identification but only provide an estimate of the quantity present. The accurate determination of concentration of pyrite in mudrocks requires the use of chemical analysis methods. TRL447 (Reid *et al*., 2005), test 3 is an analysis method specific to sulfide minerals, but usually the amount of pyrite is estimated from the difference between the total sulfur and the amount of acid



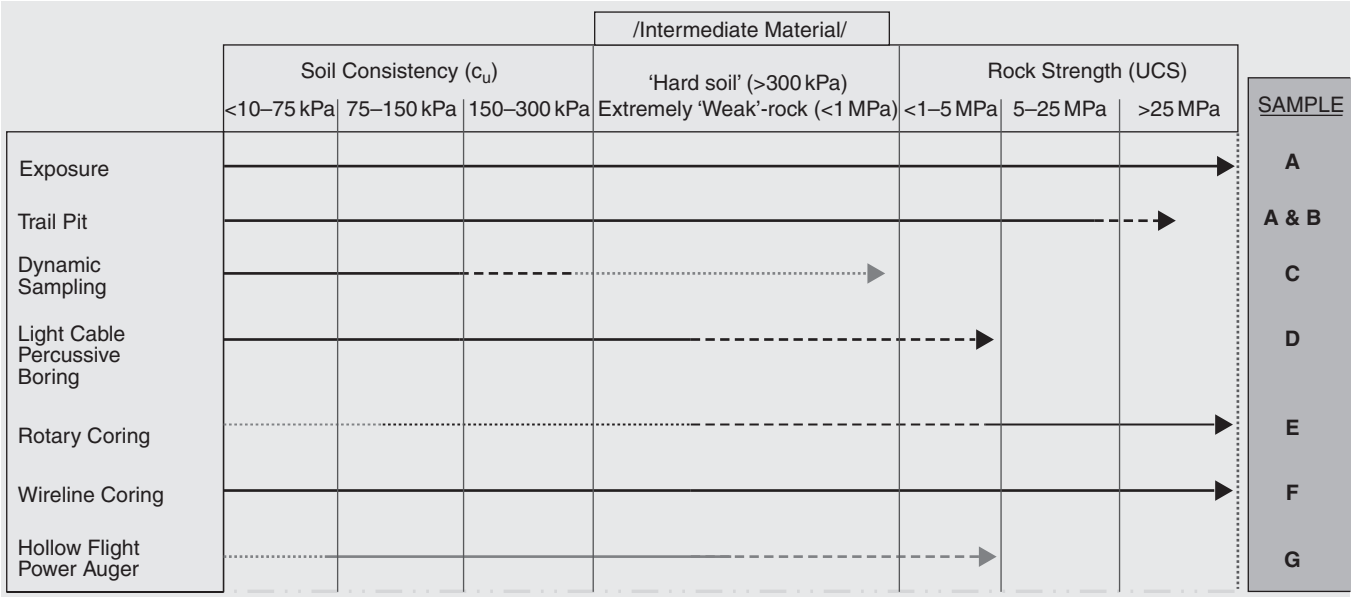
(a) (b) (c) (d)

Figure 36.20 Example of the field assessment of material weathering due to pyrite oxidation, allowing consideration for design assumptions and beneficial construction sequencing. (a) Lias Clay material proposed for use as earthwork fill was exposed to atmospheric conditions to observe for signs of deleterious reaction of pyrite. Pyrite: 2% content, with calcite observed as shell debris and nodules. (b) Presence of framboidal pyrite confirmed by SEM analysis. (c) Complete breakdown of sample shown in (a) in a 41day period with oxidation of majority of pyrite and production of ali selenite crystals and significant softening.

soluble sulfur present (TRL447 (Reid *et al*., 2005), tests 1 and 4). If sulfur is present in other substances, for example organic material, or acid insoluble minerals such as barites ($BaSO_4$) are present, this difference method will overestimate the quantity of pyrite present. The carbonate content should also be determined (BS1377:1990) as this provides an indication of the potential for buffering the acid produced by pyrite oxidation (see section 36.2.4.3). In addition, to determine the potential for sulfate attack of concrete and other construction materials, the water-soluble sulfur should be ascertained.

BRE Special Digest 1 (Reid *et al*., 2005) deals with the determination of pyrite with respect to chemical attack of buried concrete and TRL447 (2005) to the performance of fill associated with highway structures. They also make recommendations concerning the taking and storage of samples required for these assessments. Relevant information on the storage of samples is also provided by Hawkins and Pinches (1986) and Czerewko *et al*. (2003c). In particular, precautions are required to ensure that the chemical and mineralogical determinations accurately measure the condition of pyrite in the ground. Contamination of samples with groundwater or drilling water can also cause problems.

Table 36.2 shows UK mudrock formations known to contain pyrite. This list does not include all pyrite-bearing formations and inclusion in the list does not imply the material will always be problematic. In addition, the development of difficulties due to pyrite oxidation depends on diverse environmental factors.



A = *In situ* examination of exposure allows accurate assessment of mass properties and recovery of block samples – limited by age of exposure.
B = Trial pits allow inspection of mass properties and weathering of freshly exposed material; recovery of Class 1 block samples, typically limited to 4–6 m excavation depth; support required for entry.
C = Dynamic samples allow recovery of continuous ground profile; material retained in rigid plastic liner; material damage by percussive driving method; material Class 2–3.
D = Standard UK procedure; percussive recovery of U100 as Class 2–3 sample; weak rock recovered as fragments for confirmation; allows use of hydraulic thin walled push sampler with recovery of Class 1–2 material, limited to approx 100–120 kPa strength material. Gravel inclusions or nodules liable to cause damage to samples and sample tubes.
E = Hard soil and weak rock recovery, improved by use of triple tube retractor barrels (at high cost); use of large diameter (>100 mm) double/triple tube core barrels to aid appropriate recovery of weak material; use of conventional diameter (76–92 mm) core barrels for stronger competent mudrock; choice of tungsten carbide bits for weak material, and diamond impregnated and diamond surface set bits for harder rock; consider cost; sample preservation is paramount.
F = Excellent recovery of all material type as Class 1; may lose borehole progress with bit wipe-out; very expensive and requires heavy equipment.
G = Uncommon in UK due to cost and requirement of heavy equipment; drive samples Class 2–3; unsuitable for continuous sample recovery.

Table 36.10 Outline of sampling options for mudrocks and their limitations

Care must be exercised over application of the guidance, as forms of sulfur other than pyrite may be present in some rocks. Also, as the guidance is intended for the design of commonly occurring applications of concrete and fill materials, the ground aggressivity classification values obtained do not necessarily apply in other situations, for example in open excavations or in fill placed to support floor slabs and shallow foundations.

36.4 Engineering considerations

Problems in engineering are often exacerbated by being unexpected. It is argued here that by taking account of (a) the controls exerted by the composition and structure of mudrocks, and (b) the impact of the environmental changes resulting from the engineering works, unexpected forms of behaviour can be avoided. As explained in sections 36.2.2 and 36.2.3, mineralogy, compaction, cementation and structures are linked and depend on the original sedimentary processes, burial diagenesis and changes due to exhumation and weathering. This section includes descriptions of commonly occurring situations in which problems are liable to arise unless suitable counter measures are taken.

36.4.1 Foundations

Indurated strongly cemented mudrocks in a structurally sound condition present few problems in most engineering situations. Allowable foundation loads are usually derived on an empirical basis such as that described by Lambe and Whitman (1979). It is advisable to construct foundations immediately after excavation to minimise deterioration of the rock mass by exposure to the atmosphere. However, where closely spaced discontinuities or relatively weak rocks are present, the bearing capacity of shallow foundations is liable to be inadequate. It is common practice to use Young's Modulus of elasticity as the deformation modulus, notwithstanding the possibility of unrecoverable stain and a non-linear stress–stain relationship. Anisotropy, displayed by many mudrocks, may cause appreciable differential settlement which would necessitate the use of a strong reinforced concrete raft or provision for articulation in the structure.

In cemented mudrocks, settlements will usually occur immediately loads are imposed during construction, with insignificant post-construction settlement. However, consideration needs to be given to the possibility of weaker, more compressible layers being present within the ground below the structure. The impacts of discontinuities and weaker zones need to be assessed by undertaking loading or pressuremeter tests. If the magnitude and duration of long-term settlements are considered unacceptable, options such as pre-loading or deep foundations may be taken. It is also essential that the weathering grade is correctly assessed and taken into account in designs.

This includes estimation of the extent and effect of weathering, and the proportions of weathered and unweathered materials so that the foundation may be located at an appropriate level.

Where ground conditions are inappropriate for shallow foundations, piled foundations may be used. Bored piling allows more control over the installation process and causes less disturbance to the surrounding ground, although the presence of softened material at the base of the hole may result in low end-bearing resistance. On the other hand, driving piles can result in shattering of the ground and changes in groundwater conditions. This is particularly a problem with cemented mudrocks. A typical design of end-bearing piles would provide for a socket some 3–4 pile diameters deep to be formed in sound rock. Reassurance of sufficient load bearing capacity would be determined by carrying out representative pile load–settlement tests.

Due to the difficulties of taking and testing samples of mudrocks, end-bearing piles and deep foundation are often designed on the basis of the results of dynamic penetration, plate bearing and pressuremeter tests. Although the standard penetration test (SPT) is a useful means of identifying the variability of ground conditions, the parameters are not normally used for design purposes. However, in lower durability mudrocks, *in situ* plate bearing and cone penetrometer tests usually provide reliable data for design. In stronger mudrocks, penetrometer testing may be used to provide the required design parameters.

The end-bearing capacity of piles is usually based on the uniaxial compressive strength and discontinuity spacing, where the former is determined by laboratory index tests.

36.4.2 Ground volume changes

As noted above in section 36.2.4.3, the oxidation of pyrite produces sulfate-rich acidic pore waters. Neutralisation of the acid by reaction with calcite, concrete and clays results in the expansive precipitation of gypsum and other minerals. Precipitation usually occurs some distance away from areas of oxidation, particularly in areas of least stress such as bedding planes, joints or construction interfaces. Nixon (1978) describes structural damage to buildings in northeastern England. A number of cases in eastern Canada are described by Penner *et al*. (1970), and Hawkins and Pinches (1987) give details of a similar problem in Cardiff in South Wales.

Significant problems with the damage to houses in Ireland are apparently due to the expansion of pyrite-bearing fill placed beneath ground supported concrete floor slabs. Theoretical heave pressures are given in **Table 36.11**. Heave may be in excess of 100–150 mm, depending on the loading. Hoover and Lehmann (2009) calculate the potential heave based on the pyrite content of the ground, but the result is dependent on many assumptions.

In compaction mudrocks and overconsolidated clays, pyrite tends to occur as finely disseminated framboids which readily oxidise and produce randomly distributed gypsum crystals, as shown in **Figure 36.17(b)**. On the other hand, in cemented and indurated mudrocks (see **Figure 36.17(c)**), the gypsum will tend to grow as bladed crystals in a stratified manner along discontinuities such as bedding planes and joints, tending to push them apart.

Temperature °C	Concentration of solution c/c_s		
	2	4	10
5	29.2	58.5	97.1
10	29.7	59.7	98.7
20	30.8	61.8	102.2

Table 36.11 Theoretical pressures (MPa) due to the precipitation of gypsum
Data taken from Winkler and Singer (1972)

The crystallisation of selenite gypsum may occur at some distance from the site of oxidation due to solute transportation, resulting in a net translocation of sulfur from the original location. It is very difficult to specify a lower limit for the amount of pyrite acceptable in a fill. Experience in Canada (CTQ-M200, 2001) suggests that problems are unlikely to be encountered if less than 0.5% pyrite is present but that there is a high risk for values above 1%; whether higher amounts cause problems depends on the circumstances. Also, it is uncertain how differences in building construction and climatic conditions between Canada and the UK would influence this value.

Like BRE Special Digest 1 (2005), the maximum values specified by European Standard BS EN ISO13242:2002 + A1:2007 and NSAI document SR21:2004+A1:2007 are related to the chemistry of the ground adjacent to the concrete. In both documents, the total sulfur threshold is 1%, which is equivalent to 1.9% pyrite, but SR21 states that the total sulfur value should be less than 0.1% (0.2% pyrite). BRE Special Digest 1 (2005) recommends less than 0.8% sulfur (1.5% pyrite) for the ground adjacent to high quality concrete. For highway fills, TRL 447 (Reid *et al*., 2005) proposes limits of oxidisable sulfur of less than 0.3% sulfate (0.2% pyrite) and warns against the use of material in which pyrite is in the framboidal form. Hawkins and Pinches (1992) recommend caution as regards ground heave if equivalent pyrite as sulfate is above 0.5% (0.3% pyrite).

Whether a particular pyrite bearing mudrock will cause heave problems depends on many factors, including the nature of the pyrite, the presence of other minerals in the mudrock, the environmental conditions and the design of the structure. For example, in well engineered earthworks the presence of a relatively impermeable capping can limit the extent of pyrite oxidation, so reactions become self-limiting.

Figure 36.17(a) illustrates the breakdown of indurated mudrock (Edale Shale) and in **Figure 36.17(b)** the performance of less indurated Lias Clay is illustrated. Cripps and Edwards (1997) suggest that desiccation due to the removal of water and the action of acid on swelling clays can cause, shrinkage and breakdown of mudrocks.

36.4.3 Degradation of mudrock

The failure in 1984 of the Carsington Dam in Derbyshire, UK, during the latter stages of construction highlighted a number

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of issues related to the presence of pyrite in the mudrocks used for constructing the embankment. Pyrite oxidation in the mudstone fill may have compromised the effectiveness of the limestone drainage layers contained within it. Pye and Miller (1990) and Anderson and Cripps (1993) suggest that an overly optimistic view had been taken of the shear strength parameters used in the design, especially in view of the rapidity with which the mudstone degraded during construction operations. Further consideration of the engineering problems due to rapid degradation of mudrocks is given in section 36.4.4.

Breakdown processes in mudrocks in colliery spoils were extensively investigated by Taylor (1984), and a summary of this research is given in Taylor (1988). He notes that, depending on the character of the rock (controlled by the presence of structural features), slaking occurs due to wetting and drying under atmospheric conditions, and the volume changes due to swelling clays, if present. Czerewko and Cripps (2001) provide guidance on the assessment of durability and the performance of a number of UK mudrock formations of different ages and states of induration.

Although the use of specific index properties has been found to be useful for predicting the behaviour of the mudrocks in particular situations, applying the classification schemes to other formations or different applications has proved less successful. For example, although Gamble's (1971) scheme – based on second-cycle slake durability and Atterberg limits of the fines – improved on previous classification schemes, it was found to be inappropriate by Olivier (1979) when applied to tunnelling projects in South Africa. Olivier's scheme was based on uniaxial compressive strength and uniaxial free-swell testing. However, Varley (1990) reported that excessive slurrying was not predicted by Olivier's scheme in tunnels in UK Carboniferous Coal Measures mudrock strata. Problems with the operation of tunnelling machines due to the breakdown of Mercia Mudstone in Leicester, UK, were attributed by Atkinson *et al*. (2003) to the presence of swelling clay minerals in the rock.

36.4.4 Excavations and slopes

Most types of mudrocks are relatively easy to excavate using earth excavation machinery. The methods of excavation are usually determined on the basis of the seismic velocity measurements or on an assessment of uniaxial compressive strength and fracture spacing.

The excavation of tunnels, shafts and other underground cavities requires assessment of the slaking properties of the mudrock. This is to allow determination of the stand-up time for the excavations and of the handling and disposal methods for the spoil. Low silt–high clay content compaction mudrocks may be sufficiently strong to remain stable during the excavation process, but may then undergo rapid deterioration before support from the lining is fully operable.

In excavations, mudrocks may undergo lateral displacement as a result of stored strain energy being released. Some low durability cemented or compaction mudrocks, especially those containing swelling clay minerals, undergo significant swelling when the overburden and lateral restraint is reduced, resulting in uplift and lateral displacements. Rigid support systems can become overloaded and on reloading, excessive settlement may occur. This process may be exacerbated by the presence of water and tends to be more rapid in overconsolidated clay deposits than in more durable cemented mudrocks. Heave effects may often be avoided by minimising the time of exposure, for example by immediately covering exposed mudrock with sprayed concrete. Although covering does not prevent uplift due to pyrite oxidation and gypsum precipitation, preventing water entry does reduce clay mineral swelling and slaking.

In sequences of rocks in which mudrocks are inter-bedded with more durable sandstones or limestones, more rapid erosion of the mudrocks may leave overlying more competent beds in an unsupported condition. Although common in surface exposures, this effect may also lead to instability in underground openings. Behaviour of this type may be predicted by examination of the material with associated laboratory testing to assess the relative erosion rates and strengths. The stability is highly dependent on the attitudes of the beds and the rock mass jointing. Solutions to problems of this type include measures such as removal and control bolting of potentially unstable blocks, sprayed concrete or masonry work. The latter methods protect the mudrocks from weathering and erosion as well as regulating run-off and groundwater. Eyre (1973) provides details of these and similar measures required to ensure stability of steep rock faces in sequences of cemented mudstones and limestones in southwest England.

Many slopes in mudrock formations in the south of England are mantled by considerable thicknesses of solifluction deposits that contain shear surfaces which mobilise residual strength. Relatively small increases in loading, the removal of support, or increases in pore water pressures, can lead to instability (see Symonds and Booth, 1971; Weeks, 1969). Some indurated mudrock formations in other parts of the country are also mantled by solifluction or 'head' colluvium deposits, which are liable to become unstable during engineering works.

36.4.5 Highway construction

In highway construction, problems with mudrocks containing pyrite can be reduced by leaving the weathered material in place for as long as possible, or by covering exposed ground with a layer of compacted clay or a geomembrane barrier. An alternative strategy is to expose the mudstone to weathering for a suitable period of time so that pyrite oxidation and the reaction products are generated before construction takes place. The first potential drawback of this procedure is the delay to the construction programme; the second is that the sulfate now present may participate in chemical attack and cause heave due to the dissolution and reprecipitation of sulfate minerals.

Compaction of mudrocks fills is best performed using a sheepsfoot roller, which thoroughly mixes the material and facilitates a more effective reduction of voids, whilst avoiding the

formation of smooth, potentially low-strength surfaces within a layered fill. However, the final proof rolling with a smooth roller will form a smooth, low-permeability surface, which will tend to shed water. Both hot and dry *and* wet weather can cause excessive break up and/or softening of the fill during construction, which may render the material unsuitable for use.

The weight of free-draining uniformly graded granular capping layers placed over freshly exposed pyritic mudrocks in cuttings can act as a surcharge which inhibits expansion. If expansion does occur, such layers can also act as a compressive layer, and thus reduce uplift. Such an interface has the further effect of forming a barrier to the capillary rise of sulfate-rich water that could otherwise cause chemical attack or heave problems. However it is important that a separator layer prevents migration of degraded mudrock into the granular fill.

Improvement in the workability and bearing characteristics of clay soils for road and building construction involves soil stabilisation with one of the following: (a) lime, slaked lime ($Ca(OH)_2$) and cement; (b) granulated ground blast furnace slag (GGBS) and cement; or (c) pulverised fuel ash (PFA) and cement. Such methods are cheaper and more sustainable than the use of rock aggregates Pozzolanic effects produce a cementitious product known as hydraulically bound material (HBM) or cement bound material (CBM). Documented cases in the UK of problems due to expansive sulfate minerals formation include the M40 near Banbury on Lias Clay and the A10 Wadesmill bypass in Hertfordshire (Ground Engineering, December 2004). Snedker and Temporal (1990) explain that in the case of the M40, heave of 150 mm occurred in a 250 mm thick stabilised layer over a few months. It would appear that the treatment of the ground when it is in a dry condition makes heave more likely to occur. Thus timing the treatment and allowing time for reactions to occur, for example over a winter, reduce the risk of problems with expansion. Road and foundation construction may be further proofed against heave problems by the provision of appropriate drainage to maintain the groundwater level and capillary fringe below the stabilised horizon. Well designed flexible drainage is also beneficial in the case of construction of earthworks involving the use of pyritic mudrocks.

Potential problems were detected on recent and ongoing highway earthwork construction projects involving Jurassic Ancholme Clay and Lower Lias Clay (Czerewko *et al.*, 2011) in the east midlands of England. Investigations provided data on the rates of pyrite oxidation and selenite formation in these pyritic, calcite bearing mudrocks. Prior to the use of the mudrock excavated on site for use as fill, it was subjected to a three-week period of exposure oxidation and selenite formation to occur, therefore mitigating post-construction heave.

36.4.6 Aggressive ground conditions

As explained in section 36.2.4.3, the weathering of pyrite gives rise to acidic solutions rich in sulfates that are detrimental to concrete, steel and other materials (see Czerewko and Cripps, 2006a; BRE Special Digest 1, 2005). For many decades it has been standard practice to determine the sulfate content and aggressive nature of soils and fills for the design of cement and concrete in buried structures, thus sulfate attack on buried concrete has been avoided. However, the discovery of serious deterioration of the concrete foundations of 30-year-old bridge structures on the M5 motorway in south-western England revealed serious deficiencies in the assessment methodology used at that time. It is possible that thaumasite sulfate attack on concrete is much more extensive than previously thought, but there has been a lack of reporting of cases of deterioration and, where it has been reported, thaumasite has not been correctly identified.

BRE Special Digest 1 (2005) provides comprehensive guidance on procedures to predict, prevent and mitigate against deterioration of concrete due to the presence of aggressive ground conditions. Various preventative and protective measures are possible including the modified mix designs, the use of bitumen coatings to concrete, and the addition of a capillary break to prevent contact with aggressive solutions.

36.5 Conclusions

As mudrocks occur extensively in the geological sequence and are geographically widely distributed, they are frequently encountered in engineering situations. Certain types are troublesome, in that it is difficult to predict their behaviour during construction and to determine reliable values of geotechnical parameters for design. It has been explained that mudrocks are the product of the burial and subsequent exhumation of muddy sediments which have undergone change due to these processes. The resulting materials are a function mainly of the composition and structure of the original sediment, the depth of burial and local geothermal gradient. It has been suggested that understanding the process of formation of these materials can help with predicting how they will perform in different engineering situations. It is convenient to consider mudrocks under the following headings:

- compaction mudrocks (overconsolidated clays);
- weakly cemented mudrocks;
- strongly cemented mudrocks.

Compaction mudrocks are weak and compressible materials. They will degrade rapidly at the ground surface and in most engineering situations they behave as soils. Most types contain swelling clays and they will therefore undergo volume changes in response to changes in moisture content. They may be anisotropic if clay rich and usually contain pyrite in its more reactive framboidal form.

Weakly cemented mudrocks are stronger and less compressible, ranging across the boundary from strong soils to weak rocks. They are difficult to characterise and in some situations will behave as rocks; in other situations, as soils. Due to stored strain energy, the removal of overburden, and the presence of a partly bonded structure and a skeletal framework, they undergo time-dependent degradation in response to weathering. Cyclical

changes in moisture content are liable to lead to slaking in which the rock degrades into sand- and gravel-sized fragments that are quite durable. Pyrite may be present in its framboidal form.

Strongly cemented mudrocks are stronger and less compressible. They are liable to range downwards into weak rocks, but in most engineering applications they will behave as rocks. Weathering action can result in them consisting of a mixture of weaker and stronger materials. Pyrite is liable to be in a more stable crystalline form although polyframboidal pyrite may be present.

Guidance has been given concerning the investigation, classification and description of mudrocks. The fact that many mudrock formations also contain pyrite gives rise to problems with the generation of chemically aggressive solutions and heave due to the precipitation that the reaction produces. Finally, a number of potential difficulties with engineering works involving mudrocks were presented.

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It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 33 *Expansive soils*
- Chapter 40 *The ground as a hazard*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Glossary

The list below contains definitions for terms used in this chapter. Words presented in italics are themselves defined elsewhere in the list. Further definitions are given in *Dictionary of Earth Sciences*, edited by M. Allaby, Oxford University Press, 2008.

Air breakage	Process of *slaking* controlled by air pressures produced by the capillary water in the pore spaces of fine-grained, partly saturated rocks.
Anisotropy	Term describing the physical property which depends on the direction relative to a defined axis, such as *bedding* or *foliation*.
Argillaceous	Sediments or sedimentary rocks that are composed predominantly of *clay* and *silt*-sized material, comprising significant quantities of *clay minerals*.
Argillite	Compact, strong, fine-grained, *clay*-rich sedimentary rock which has been strongly *lithified* by deep burial or *low-grade metamorphism*, but lacking *lamination*, *fissility* or *cleavage* structures.
Authigenic	*Mineral* that has formed *in situ* in deposits during progressive *diagenesis*, resulting from changes in burial and temperature.
Burial diagenesis	*Diagenesis* that occurs due to burial of sediments beneath later sediments.
Cation	An atom (or group of atoms) which is deficient in one or more electrons to produce a net positive charge e.g. Al^{3+}.
Cation exchange capacity	The maximum quantity of *cations* that may be sorbed onto the surfaces of *mineral* particles at a given pH value.
Cement	Mineral material precipitated in the pore space and interparticle chemical bonds that hold grains together.
Cementation	Process by which the grains of geomaterials become attached together by the precipitation of mineral matter between the grains and the formation of interparticle chemical bonds.
Clast	A fragment of rock or *mineral* grain that has been derived by erosional processes from older rock.
Clastic	*Sediment* or *sedimentary* rock that is composed of (broken) fragments of other rocks and minerals derived by *weathering* and *erosional* processes.
Clay grade	An average grain size of less than 0.002 mm. Although clay grade material usually contains significant amounts of *clay* minerals, other material such as quartz, organic matter, sulfides and carbonates may also be present.

Clay mineral	A group of *minerals* of aluminosilicate composition and a phyllosilicate crystal structure similar to that of mica i.e. a sheet layer structure with strong inter- and intrasheet bonding, but weak interlayer bonding. Commonly occurring clay minerals include illite, kaolinite and smectite (montmorillonite).
Claystone	An *argillaceous* sedimentary rock of *clay* size particles (grade) mainly comprising *clay* minerals.
Colliery spoil	Waste rock material produced in the course of coal mining or coal processing. Usually contains a high proportion of mudrocks.
Compaction	In geology, the physical processes that convert loose unconsolidated sediment into a dense coherent rock, but without significant change in mineralogy. In geotechnics, the process of expelling air and moisture from a fill material by application of a dynamic load such as rolling or vibration.
Consolidation	In geotechnics, the reduction in porosity due to the application of a constant *pressure*.
Detrital	Rock or mineral grains derived from pre-existing parent rock by process of *weathering* and/or *erosion*, transported as detritus and deposited as sediments.
Diagenesis	The chemical, physical and biological changes due to burial of sediment, including *consolidation, cementation,* growth of *authigenic minerals* and increases in grain size. Excludes changes due to *weathering* or *metamorphism.*
Diffuse double layer	Part of the ion exchange process common in *clay* minerals caused when clay particles are suspended in water and interlayer *cations* on the surface of the clay particles go into solution, producing a negatively charged *clay* surface surrounded by a diffuse double layer of hydrated cations.
Elastic modulus	Amount of *stress* to produce a unit amount *strain*, where removal of the stress results in recovery to the original dimensions.
Fissility	The ability of a rock to be broken along closely spaced parallel planes, resembling the pages of a book.
Flocc	An aggregation of *clay* particles bound to each other by electrostatic attraction present in a neutral electrolyte fluid.
Flocculation	The process of aggregation of *clay* particles into larger groups, either through chemical or biological means.
Foliation	The planar arrangement of textural features or platy particles in a rock such as schistosity in *metamorphic* rocks.
Framboid	A microscopic spherical aggregate of pyrite grains in a form resembling a raspberry in *argillaceous* deposits
Induration	The natural process of hardening of sediments into rock by pressure, heat or *cementation*.
K_o value	Earth pressure at rest, which is the ratio between horizontal and vertical effective stresses in the ground.
Laminae	Distinct layers of sediment, often with different grain size or colour and typically less than 10 mm (an upper boundary of 20 mm is used in the engineering description of rocks and soils).
Liquid limit	Minimum *moisture content* for wet soil to behave as a liquid under specified conditions.
Low-grade metamorphism	Alteration by the action of heat and/or *pressure* of existing rocks, under conditions in which recrystallisation and new mineral growth in the solid state is just possible.
Lutaceous	A term used to describe a sedimentary rock formed from mud (clay and/or silt-sized particles).
Moisture content	Mass of water contained in a soil expressed as a percentage of the mass of dry solids.
Mudrocks	A rock formed by the *compaction* or *induration* of mud with a modal grain size in the mud grade (<0.063 mm). A generic group name for mudstone, siltstone and claystone, and *over-consolidated* clay.
Mudstone	A fine-grained sedimentary rock that is composed of *silt* and *clay*-sized particles.
Normally consolidated	A condition in which the present *overburden stress* being borne by a deposit is the maximum it has experienced since deposition.
Over-consolidated	A condition in which the *overburden stress* acting on a deposit has been greater in the past than it is at present; thus the deposit has been subject to a cycle or cycles of loading and unloading giving it greater density, *strength*, *durability* and lower compressibility compared to a similar normally *consolidated* deposit.
Periglacial	Climatic conditions experienced in the vicinity of retreating ice sheets or glaciers, entailing freeze–thaw action.
Petrology	The study of rocks through their mineralogy, geochemistry, textures, field relationships and their formation.
Phyllite	An argillaceous rock that has been altered by *low-grade metamorphism*; of coarser grain size and less perfectly cleaved than a slate, but finer grained and better cleaved than a schist.
Phyllosilicates	Sheet or layer silicate minerals composed of tetrahedral and octahedral sheets joined together to form 1:1 or 2:1 layers. In between the layers there may be interlayers of hydrated cations and water, which may compensate excess negative charges arising from the substitution of intra- and intersheet cations by other cations of lower valency.
Plastic limit	Minimum moisture content for soil to behave in a plastic manner – in which it is possible to mould the material by hand pressure and it will retain its shape.
Pressure	Force per unit area.
Schist	A type of 'medium grained' *metamorphic* rock characterised by a distinct crenulated particle alignment (foliation) known as schistosity.
Shale	A *fissile mudrock*. (As fissility is a feature that can develop in mudrocks as a consequence of weathering action, it is preferable to refer to such materials as fissile mudrocks.)
Shear strength	The minimum *shear stress* that causes a specimen to fail under conditions of zero *normal* stress. In a saturated clay, this is half the uniaxial compressive strength.
Siltstone	A fine-grained sedimentary rock consisting of particles of average size between 0.002 and 0.063 mm.
Slaking	A degradation process in geomaterials caused by alternate cycles of wetting and drying.
Solifluction	Process by which layers of soil move downslope due to freeze–thaw action under *periglacial* conditions.
Uniaxial/unconfined compressive strength	Compressive *pressure* to bring about failure of a specimen where loading is applied along only one axis and the specimen is an otherwise unconfined condition.
Weathering	A series of physical, chemical and biological processes by which a material undergoes degradation that brings it into equilibrium with its present environment.

Chapter 37
Sulfate/acid soils

J. Murray Reid TRL, Wokingham, UK

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CONTENTS

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Sulfur is widespread in rocks and soils and occurs in two principal forms; in an oxidised state as sulfate, principally gypsum ($Ca.SO_4.2H_2O$); and in a reduced state as sulfide, principally pyrite (FeS_2). Other forms can occur in special circumstances. The oxidation of pyrite and other sulfides can occur as a result of geotechnical activities, when soils and rocks are disturbed and exposed to air and water. This can lead to the generation of high concentrations of sulfate and low pH in percolating rainwater, which can have a number of adverse consequences including attacks on construction materials such as concrete and steel, precipitation of gypsum leading to heave of foundations and floor slabs, and pollution of watercourses with precipitation of hydrous iron oxides (ochre). Pyrite oxidation can also be triggered by mixing susceptible materials with lime or cement; the resulting high pH values lead to formation of expansive sulfates (ettringite and thaumasite) that cause heave of the stabilised soils. Test methods are available but have to be used with care because of the variability of sulfur compounds. Geotechnical engineers have to recognise the types of material and situation that could lead to problems and how these can be avoided or mitigated.

37.1 Introduction and key background information

Chemical reactions within fill materials or between fill materials and structures often come as a surprise to geotechnical engineers in the UK. This may be because many of them have little or no background in geochemistry and assume that chemical reactions only happen in laboratories or under strictly controlled conditions in industrial processes. Soils and rocks are assumed to be inert; once excavated and compacted they will not change their properties, other than physical changes associated with changes in moisture content. Many of the soils and rocks in the UK are chemically inert, but a surprising number will undergo chemical changes when disturbed and most of these changes will have adverse consequences for the civil engineering works in which they occur. Chief among the causes of these problems are sulfur compounds.

Sulfur is the sixteenth most abundant element in the earth's crust, with an average concentration of 260 mg/kg (Krauskopf, 1967), greater than carbon (200 mg/kg), chlorine (130 mg/kg) and zinc (70 mg/kg). It is widespread in rocks and soils and occurs in two principal forms; in an oxidised state as sulfate, principally gypsum ($Ca.SO_4.2H_2O$); and in a reduced state as sulfide, principally pyrite (FeS_2). Other forms can occur in special circumstances, such as gaseous hydrogen sulfide (H_2S) in landfill sites, sewers and areas of active volcanicity. The problems that affect geotechnical engineering works are largely due to sulfate in solution and to changes from one sulfur compound to another. Excavation can allow air and water to reach pyrite and other sulfide minerals, leading to oxidation to sulfates and the generation of acidic conditions. This can result in attacks on concrete, steel and other construction materials, blocking of drains and filters and pollution of watercourses. Treatment of soils with high concentrations of sulfates and sulfides with lime or cement can lead to the formation of complex expansive sulfate minerals that cause heave, with resulting disruption to overlying layers of asphalt or concrete. Heave of soils below floor slabs can also occur in untreated soils due to the inflow of sulfate-rich groundwater, leading to precipitation of gypsum in the fill.

A feature common to many of the problems listed above is that they do not occur immediately but some time – ranging from months to years – after construction. They can, thus, be very expensive to remediate, as the overlying structures may be disrupted to such an extent that they are no longer functional and have to be removed or significantly modified. It is, thus, important for geotechnical engineers to be able to recognise potentially susceptible materials and inappropriate treatments or construction methods and either avoid them or take mitigating measures. It is possible to achieve satisfactory engineering solutions in a number of situations provided the potential problems are foreseen and appropriate measures adopted. Problems mainly occur where they are not foreseen. This chapter will describe the potential situations where problems can arise, how to predict them and, where possible, how to provide engineering solutions.

The examples of problems due to sulfur compounds in soils primarily relate to damage to engineering structures and pollution of the environment. These are serious enough, but sulfur compounds can also lead to death, though thankfully very rarely. Poisoning by hydrogen sulfide can occur in enclosed spaces; this rarely happens other than in contaminated land, sewers or landfill sites, which are not the main focus of this chapter. However, potentially fatal situations can also arise in civil engineering works. **Figure 37.1** shows the memorial to four workmen who were killed due to carbon dioxide (CO_2) poisoning in a manhole during the construction of Carsington



Figure 37.1 Memorial to workmen killed by CO_2 poisoning, Carsington Dam, Derbyshire

Dam, Derbyshire, in 1982. The dam was being constructed of local highly weathered mudstone fill, with drainage blankets of limestone to help dissipate the pore pressure during construction. The fill was acidic with high levels of sulfate; this attacked the limestone, precipitating gypsum in the drainage blankets and generating carbon dioxide, which travelled down the drainage system to the manhole where the workmen entered. The behaviour of sulfur compounds is, thus, more than just an academic or engineering issue; it can be a matter of life and death, and something that geotechnical engineers need to be aware of.

37.2 Sulfur compounds in soils and rocks

37.2.1 Sulfur minerals

One of the confusing things about sulfur is that it can occur in a variety of forms and can change from one to another under certain conditions. The most common sulfur minerals are listed in **Table 37.1**. The principal form of sulfur in natural soils and rocks is in the reduced form, as sulfide. Pyrite (FeS_2) is the most common sulfide mineral and is a common minor constituent of many igneous, metamorphic and sedimentary rocks and the soils derived from them. Many mineral ores are sulfides, including galena (PbS) and sphalerite (ZnS). Sulfide minerals generally have very low solubility and are not in themselves a hazard to construction materials. However, weathering of sulfides under oxidising conditions gives rise to sulfates (SO_4^{2-}), which are soluble and can attack construction materials. Under natural conditions, weathering of sulfides normally leads to production of gypsum ($CaSO_4.2H_2O$), which has limited solubility under neutral pH conditions. This is the normal form of sulfate in near surface soils.

Sulfates also occur naturally in evaporate sequences as gypsum, anhydrite ($CaSO_4$), barytes ($BaSO_4$), epsomite ($MgSO_4.7H_2O$) and other salts. Barytes is extremely insoluble, but anhydrite, epsomite and other salts are highly soluble; as a result, in the UK they have generally been leached out of the near surface zone where most civil engineering works are carried out, and are only likely to be encountered in shafts and tunnels at depth. They can cause serious problems to civil engineering works in these situations, due to corrosion (they are often associated with chlorides and other salts) and swelling when exposed to water. These situations will not be considered further in this chapter.

Mineral	Formula	Description
Anhydrite	$CaSO_4$	Found in evaporite strata at depth; white crystals, highly soluble
Barytes	$BaSO_4$	Common minor constituent in wide range of rocks; white crystals, dense, very insoluble
Celestine	$SrSO_4$	Rare mineral, occasionally found in evaporite deposits
Epsomite	$MgSO_4.7H_2O$	Common in evaporite sequences; highly soluble, only found at depth
Gypsum	$CaSO_4.2H_2O$	Widespread occurrence in soils and rocks; white crystals and powder; slightly soluble at neutral pH, soluble in acid
Jarosite	$KFe_3(OH)_6(SO_4)_2$	Weathering product of pyrite often found in mudrocks, usually as a yellow-green powder
Marcasite	FeS_2	Found as nodules in chalk and limestone; different crystal structure to pyrite
Mirabilite (Glauber's salt)	$NaSO_4.10H_2O$	Found in evaporite strata at depth; highly soluble
Pyrite	FeS_2	The most common form of sulfur in soils and rocks, ranges from large yellow cubes to fine-grained crystals invisible to the naked eye; insoluble but oxidisable
Pyrrhotite	FeS	Occasionally found in soils and rocks; different crystal structure to pyrite
Organic sulfur		Common in peat and organic soils

Table 37.1 Sulfur minerals found in UK soils and rocks

Pyrite is immediately recognisable when present in large crystals as 'fool's gold'. It is bright brassy yellow, heavy (specific gravity 4.9), often occurring as cubes and octahedra. However, it can also be present in very fine-grained forms that are invisible to the naked eye; these forms are much more damaging because the crystals will react much more rapidly when exposed to air and water than the large visible crystals. Gypsum can occur in a wide variety of forms, from large clear crystals in undisturbed strata to fine white powdery crystals growing on exposed surfaces of reactive clays or mudstones. **Figure 37.2** shows white crystals of gypsum left behind when a pool of rainwater evaporated in a cutting in the Ancholme

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Clay, a Jurassic mudstone rich in sulfides and sulfates. Such fine white powders often form on the surface of fill materials that have a high sulfate or sulfide content, and are a sure sign of trouble. They are often accompanied by orange brown deposits of hydrous iron oxides (ochre), which are another product of pyrite oxidation (Equation (37.1)). These deposits can block drains and blanket the beds of watercourses, leading to the death of all aquatic life. It is important for geotechnical engineers to be able to recognise telltale signs such as precipitates of gypsum and ochre on fill surfaces, in seepages or pools at the toe of an embankment; they are better indicators of trouble than any number of chemical test results.

37.2.2 Weathering of pyrite

37.2.2.1 Weathering reactions: what happens?

The main weathering reaction under natural conditions is the oxidation of pyrite and other sulfide species to sulfates. This occurs slowly under normal conditions of soil formation. The top 2–3 m of natural strata in the UK often contain very low concentrations of sulfates and sulfides, as they have been leached out by rainfall. At the base of this zone there may be accumulations of sulfates, which may cause a hazard for foundations. At greater depths, sulfide is the dominant form except in evaporite sequences. This transition from sulfate to sulfide corresponds to the depth of surface weathering, often with black or dark grey colours at depth and yellow, brown or mottled colours in the weathered zone. Profiles like this are particularly common in overconsolidated clays and mudstones, which are widespread in the UK (see Chapter 36 *Mudrocks, clays and pyrite*).

Accelerated weathering can occur as a result of civil engineering works, particularly when materials are excavated and placed as fill with percolating rainwater and air; free-draining embankment fill is one of the most oxidising environments in nature. In these situations acidic conditions can develop, leading to greatly increased solubility of sulfate and metals and potential attack on construction materials. Oxidation of sulfides can also be greatly accelerated by microbiological activity. Initially, the iron is oxidised to the ferrous state (Fe^{2+}) and remains in solution. When the percolating water leaves the embankment and is exposed to the atmosphere, however, the iron is oxidised to the ferric state (Fe^{3+}) and precipitates as hydrous iron hydroxide, usually as an amorphous orange sludge. The main chemical reactions in the oxidation of pyrite are summarised below. While the details of the reactions can be complex and dependent on the specific situation, the overall result is the conversion of pyrite to iron hydroxides (ochre) with the production of sulphuric acid and generation of pH values as low as 2 or 3.

$$4FeS_2 + 15O_2 + 14H_2O \rightarrow 4Fe(OH)_3 + 8H_2SO_4. \quad (37.1)$$

If carbonate minerals such as calcite are present, the acid will react with them to produce gypsum and carbon dioxide.

$$CaCO_3 + H_2SO_4 + H_2O \rightarrow CaSO_4.2H_2O + CO_2. \quad (37.2)$$

The precipitation of gypsum involves a large increase in volume and can lead to heave of the strata involved; Bell (1983) quotes an eightfold increase in volume, exerting pressures of up to about 0.5 MPa. This is a major cause of problems with sulfur compounds. If carbonate minerals are not present, the acid may attack clay minerals producing minerals such as jarosite (**Table 37.1**). The drainage water from areas of pyrite oxidation can be highly acidic, with pH values in the range 2 to 4, and very high concentrations of dissolved sulfate and metals such as iron, manganese and aluminium. When these waters encounter the atmosphere, the iron is rapidly oxidised and precipitates as ochre, causing pollution of watercourses and blockage of drains (**Figure 37.3**).

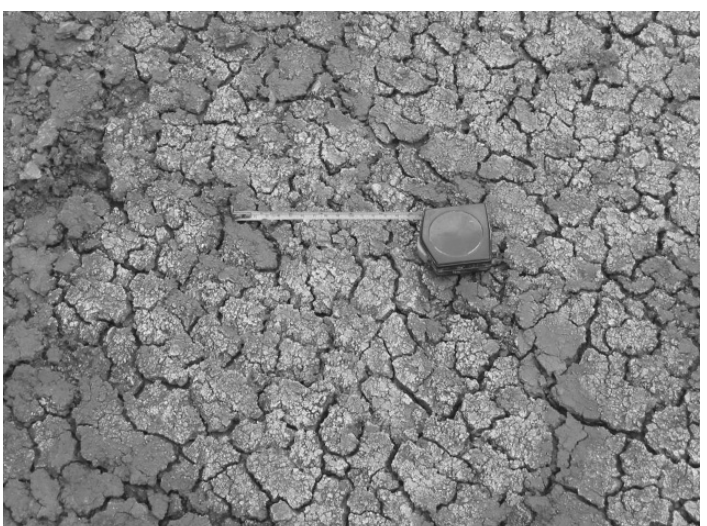


Figure 37.2 Precipitate of gypsum on surface of Ancholme Clay



Figure 37.3 Drainage outflow from mudstone embankment dam showing precipitation of ochre

37.2.2.2 Rate of reactions: how quickly will it happen?

A key factor in whether pyrite or other sulfides will cause problems in civil engineering works is the rate at which it will be oxidised to sulfates in any given situation. This depends on a number of variables:

- mineral grain size;
- form of mineral present;
- access to air and water;
- microbiological activity;
- total amount of sulfide present.

Mineral grain size

One of the most important influences on the rate of reaction of any mineral is the grain size. Reactivity is related to the specific surface area of individual particles, which is related to the inverse square of the grain size (surface area of a sphere $A = 4\pi r^2$). Large crystals visible to the naked eye will, thus, be much slower to react than very fine-grained crystals at a scale of a few microns. This leads to the paradox that pyrite is generally dangerous only if you cannot see it; it might be dubbed 'the invisible menace'. In any garden centre, ornamental lumps of granite and slate with abundant visible pyrite can be purchased for use in rockeries, with no fear that the pyrite will undergo any reactions (**Figure 37.4**). However, microscopic clusters of pyrite crystals known as 'framboids' are particularly reactive. These can be identified by electron microscopy (**Figure 37.5**), but this is not a tool available for routine investigation and assessment.



Figure 37.4 Coarse-grained granite with visible pyrite

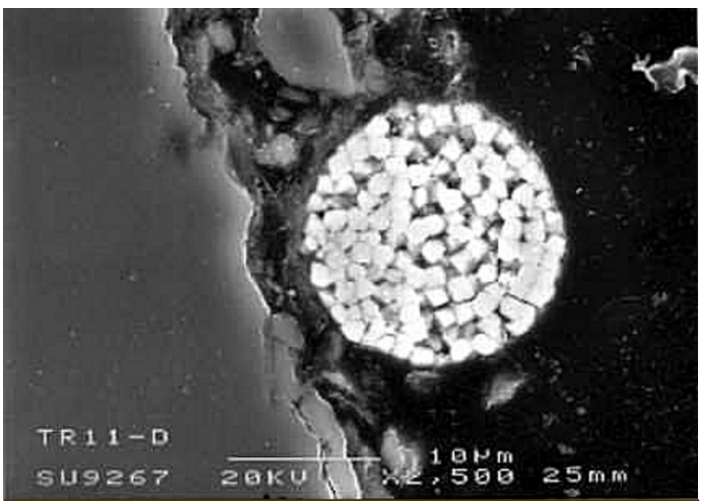


Figure 37.5 Framboidal pyrite from alluvial sand

Form of mineral

The reactivity of sulfides is also linked to the form or degree of crystallinity of the minerals. In general, the older the mineral, the more likely it is to have been recrystallised at higher temperatures and pressures, and the slower it is likely to be to react under atmospheric or in-ground conditions at the present day. This may explain why sulfides in granites and slates tend to be less reactive than those in mudstones and overconsolidated clays. The particularly high reactivity of framboids may be because in some cases they have formed very recently in alluvial strata such as sands and silts. However, where strata have been subject to weathering before being disturbed for civil engineering works, they are likely to be more reactive than fresh strata, regardless of their age. Thus, even older rocks that have been highly weathered may be susceptible to oxidation when exposed in engineering works. This aspect is harder to quantify than grain size, but it is important to bear in mind when assessing how likely a particular material will be to undergo sulfide oxidation reactions.

Particular problems may arise if certain iron sulfide minerals are present; these are found principally in recent alluvial or estuarine sediments, where they form in reducing conditions owing to the decay of organic matter. They generally only occur in very specific situations, in particular in backfill to sheet piles in marine works just below mean water level (Tiller, 1997). These compounds are particularly corrosive and cause major problems for steel sheet piles, but they do not occur in the majority of soils and rocks.

Access to air and water

Both air and water, in sufficient quantities, are required for sulfide oxidation (Equation (37.1)). However, unless the products of reaction are removed and a fresh supply of air and water provided, equilibrium will soon be reached and no further reaction will occur. If the strata are below the water table, very little reaction will occur because the supply of air is limited, although there may be an abundant flow of water. Above the water table a steady percolation of water and air

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through the strata is required to drive the reactions, removing the products through drainage and causing problems elsewhere. Embankments present a suitable environment for ongoing oxidation reactions. At Roadford Dam, Devon, the weak mudstone fill was found to contain pyrite during the site investigation phase, and measures were put in place to monitor the extent of this during construction and operation (Davies and Reid, 1997). Observations over a period of 20 years show that pyrite oxidation reactions are continuing to occur at much the same rate as during construction, despite the fill being covered by an asphaltic upstream membrane, an asphalt road on the crest, and topsoil and vegetation on the downstream shoulder (Hopkins *et al*., 2010). The rate at which reaction products are leached is controlled primarily by rainfall and the reservoir level. Similarly, use of colliery spoil to top up canal embankments subject to mining subsidence can lead to poor quality of seepage water due to oxidation of pyrite in the spoil (Perry *et al*., 2003).

Microbiological activity

The oxidation of pyrite occurs in a number of stages, which are discussed by numerous authors including Reid *et al*. (2005) and Chapter 36 *Mudrocks, clays and pyrite*. Many of these stages are catalysed by bacteria, which enable the reactions to proceed more rapidly than might be expected from considerations of chemical equilibrium alone. Different bacteria can feed off the oxidation and reduction of sulfur and iron, obtaining energy and nutrients from the reactions. They can exist and even thrive in the most extreme environments, including the very acidic conditions generated by oxidation of sulfides. So if a reaction is likely to occur, it should be assumed that microbiological activity will assist the process.

Total amount of sulfide present

It might be expected that the total quantity of sulfide would be the main factor in determining the extent of the reaction. In fact, as illustrated by the factors in the preceding sections, it is relatively unimportant; grain size, form of mineral and exposure to air and water are more important in determining the extent of reaction. Thus, some materials can have quite high contents of sulfides in the form of large crystals and cause no problems for engineering works, whereas other materials with small quantities of highly active sulfides can cause very significant problems. Sulfide concentrations have to be almost at the limit of detection before it can be certain that they will not cause problems. This causes difficulties for geotechnical engineers, as simple reference to sets of limiting values may give a conservative view of the likelihood of sulfide oxidation and its potential consequences. Moreover, while the total sulfur content of soils and rocks can be determined relatively reliably and cheaply, determination of the total sulfide content is more difficult. Even if this can be determined, an assessment then has to be made as to whether the sulfide is likely to oxidise and, if so, at what rate and what effect this will have on the works or the environment.

This is illustrated in **Table 37.2**, which summarises the rate of reaction from four sites in the UK that have experienced problems due to sulfur compounds. There is no relation between the sulfur content and the rate of reaction or the extent of the damage. The most severe damage occurred in the material with the lowest sulfur content, the alluvial sand and gravel, though this was also the application most sensitive to sulfate and acid attack. For the two embankment dams, the higher rate of reaction at Roadford is due to the greater permeability of the fill material compared to Carsington, which allowed greater percolation of air and water and more rapid reaction despite a significantly lower total sulfur content. The effects of the reactions were more severe at Carsington, however. Where large volumes of fill are concerned, even a small proportion of active material can have significant effects. For the case study of the M5 bridges (Thaumasite Expert Group, 1999), the quantities are much smaller but the extent of reaction appears to have been much greater due to the specific circumstances, which encouraged the thaumasite form of sulfate attack: disturbance

Construction type	Material	Total sulfur (% S)	Rate of oxidation	Effects of reaction
Roadford Dam	Carboniferous mudstone	0.6	0.15% over 1 year	No effects on fill detected; drainage outflow still requires treatment after 20 years because of high metal and suspended sediment content
Carsington Dam	Carboniferous mudstone	3.2	0.005% over 4 years	Attack on concrete and drainage blankets in original dam. Measures taken to mitigate damage in reconstructed dam. Drainage outflow requires treatment
A564 Hatton – Hilton – Foston	Alluvial sand and gravel	0.3	90% over 4 years	Rapid severe corrosion of galvanised corrugated steel buried culverts, which had to be replaced; fill removed and replaced with inert material
Backfill to bridge foundations on M5, Gloucestershire	Lias Clay	1.6	15% to 50% over 25 years	Attack on concrete to a depth of up to 40 mm (thaumasite form of sulfate attack)

Table 37.2 Rate of pyrite oxidation from UK sites
Data taken from Reid *et al*. (2005)

of the fill allowed oxidation of pyrite to start; cool, damp conditions; mobile groundwater; and the presence of carbonate aggregate in the concrete. Only by taking all the circumstances of each case into account and being aware of the likely geochemical behaviour of the materials can predictions be made of the likely results. Having done this, however, it may be possible to avoid or mitigate them.

37.2.3 Occurrence of sulfur compounds

Sulfides, particularly pyrite, are common as minor constituents in igneous, metamorphic and sedimentary rocks and in the sediments and soils derived from them. Lists of geological formations in the UK known to contain pyrite in sufficient quantities to cause problems are given in BRE Special Digest 1 (Building Research Establishment, 2005) and HA Advice Note 74 (Highways Agency *et al*., 2007). Some of the materials that are particularly likely to cause problems are discussed below; however, it is important that every situation is evaluated on its merits and not by reference to lists of 'usual suspects'.

37.2.3.1 Mudstones and clays

Mudstones and overconsolidated clays are particularly likely to contain significant concentrations of pyrite in fresh strata and a mixture of pyrite and gypsum in weathered strata. The top 1 to 2 m of natural ground may be low in both sulfates and sulfides as a result of leaching by rainfall, but for several metres below this the concentrations of sulfate and sulfide can be highly variable horizontally and vertically. Potential problems due to sulfates and sulfides should always be considered when engineering works in these strata are proposed. Among the key strata in this group are:

- mudstones of Carboniferous and more recent age;
- Mercia Mudstone (Keuper marl – sulfates only, not sulfides);
- Lower Lias Clay;
- Kimmeridge Clay (the Ancholme Clay in **Figure 37.2** is a lateral equivalent of the Kimmeridge Clay);
- Oxford Clay;
- Weald Clay;
- Gault Clay;
- London Clay;
- glacial till or other drift deposits derived from any of the above.

These strata are also likely to pose other problems for engineering works, including seasonal shrink-swell behaviour, low long-term shear strength in slopes and excavations and the presence of organic matter. These aspects are discussed by Cripps and Czerewko in Chapter 36 *Mudrocks, clays and pyrite*.

37.2.3.2 Alluvial strata

Active sulfates and sulfides are not restricted to fine-grained soils and rocks; a number of cases have been reported of problems due to sulfur compounds in alluvial sands and gravels. In these cases the sulfide is often in framboidal form and, hence, highly active (**Figure 37.5**). The framboidal pyrite is probably secondary in nature, derived from primary pyrite in the heavy mineral assemblage of the sediments. As these strata often have little or no carbonate content, highly acidic conditions can develop if they are excavated and used as fill, even if the sulfide and sulfate content is quite low. An example of this is given by Reid *et al*. (2005), where alluvial sands and gravels were used as backfill to galvanised corrugated steel buried culverts; this led to rapid corrosion of the steel and the culverts had to be replaced before the road was opened to traffic. Similar examples of problems in granular soils are reported by Sandover and Norbury (1993).

Organic soils, including peat, can also contain a significant sulfur content. These soils are also often acidic, owing to humic acid from decaying organic matter, but this will not produce pH values below about 3.5, whereas sulfuric acid can lower them to as low as 2 (Building Research Establishment, 2005).

37.2.3.3 Colliery spoil and related materials

Colliery spoil consists of strata excavated in the course of mining for coal. It includes strata immediately above and below the coal seams, fragments of the coal seams and strata from the driving of roadways and shafts to access the seams. For a large colliery working a number of seams it can, thus, be highly variable in composition. The dominant material tends to be mudstone and this can often contain significant amounts of sulfides, particularly in marine mudstones, which often form the roof of coal seams. The spoil will also contain significant amounts of coal, which also tends to have a high sulfide content. Modern collieries extract coal from considerable depths, so the spoil may contain high concentrations of chloride from the seawater in which the mudstones were laid down. The carbonate content can be highly variable, with significant concentrations of calcite in some marine mudstones. The mudstones and sandstones immediately below the coal seams generally have a low calcite content. Fresh colliery spoil generally has a pH in the range 7.5 to 8.5, but on exposure to the atmosphere, oxidation of the sulfides can occur and the pH can drop to values as low as 2 or 3.

The potentially corrosive properties of colliery spoil have long been recognised (West and O'Reilly, 1986) and it is not permitted as backfill to concrete and structural metallic elements by the UK Specification for Highway Works (Highways Agency *et al*., 2009a).

Colliery spoil that has been placed loosely in tips has often suffered ignition, resulting in combustion of the coal particles and changes to the chemical and physical properties of the material. Most noticeable is a change in colour from dark grey to red, accompanied by an increase in the particle strength, so that well-burnt colliery spoil is a more versatile and useful fill material than unburnt material, and is permitted in a wider range of applications by the Specification for Highway Works

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(Highways Agency *et al.*, 2009a). Most sulfide is oxidised to sulfate during the combustion process. Spent oil shale is similar in many ways to burnt colliery spoil; it is the residue from the extraction of oil from oil shale of Carboniferous age in the Lothian area of central Scotland. The shales were heated in retorts to drive off the oil, leaving a red material with variable quantities of sulfate but generally low sulfide content. Both burnt colliery spoil and spent oil shale are known colloquially as 'blaes' in Scotland.

37.3 Sampling and testing for sulfur compounds

In order to assess their potential impact on civil engineering works, it is necessary to quantify the amount of different sulfur compounds present in a material. This first requires the taking of representative samples and their storage in a way that preserves the compounds in the form that they were in when the sample was taken.

37.3.1 Sampling and storage

Generally it will be necessary to sample both the soil strata and groundwater for sulfur compounds during the site investigation. Samples of groundwater are particularly valuable, as this is the medium in which sulfur compounds can cause most damage to materials or the environment. It is important that water samples for chemical testing, whether obtained from boreholes, trial pits, seepages or watercourses, are taken in accordance with best practice guidelines and preserved so that the chemical composition does not change before analysis in the laboratory. It is also useful to determine basic parameters of the sample at the time of sampling, in particular temperature, pH value, conductivity and redox potential/dissolved O_2. These will aid interpretation of the results later and give a good general indication of the chemistry of the water and whether it is likely to be of concern.

Soil samples for chemical testing should also be taken and stored in accordance with best practice. This is particularly important given the propensity of sulfur compounds to change from one form to another when removed from the ground. Trials by Reid *et al.* (2005) found that significant changes can occur if samples are stored inappropriately. They recommended that samples be placed in airtight containers, stored at 0–4ºC and analysed as soon as possible. These protocols should be observed for samples taken during site investigations and also for samples taken during construction works as a check on the composition of materials being used in the works; samples left lying around a site laboratory or a storage container in plastic bags at ambient temperatures for months before analysis are unlikely to accurately represent conditions in the ground.

Ground investigation and testing in the UK is now covered by BS EN 1997–2:2007 and the corresponding National Annex, NA to BS EN 1997–2:2009. These documents state that chemical testing of soils should continue to be carried out in accordance with BS 1377–3 with the exception of pH value determination (acidity and alkalinity), where Annex N.5 of BS EN 1997–2 may be used. Further information on how to use the various tests in BS 1377–3 is given in NA3.15 of the National Annex.

37.3.2 Water-soluble sulfate (WS)

The most common, and useful, test is the sulfate content of water samples or the water-soluble sulfate content of soil or rock samples. This is the form of sulfate that attacks construction materials and instigates other chemical reactions in fill materials and waters; it, thus, represents the immediate danger posed by the material or water. For soil or rock samples, the sulfate is brought into solution with a 2:1 extraction in distilled water for 24 hours, after which the solution is filtered and the sulfate concentration determined. Traditionally, the sulfate content has been determined gravimetrically by adding barium chloride in acid solution to precipitate barium sulfate. This is still the reference method in the British Standard test methods for soils (BS1377–3) and aggregates (BS EN 1744–1). However, most commercial laboratories prefer to use rapid automated methods such as inductively coupled plasma – optical emission spectroscopy (ICP-OES) to determine the amount of sulfate directly in solution, and this method is described in TRL447 Test 1 (Reid *et al.*, 2005). For water samples, the amount of sulfate can be determined directly from the sample, though filtering may be necessary if it contains suspended solids.

The results should be expressed as mg/l SO_4, as this is the form used by most guidance documents when setting limiting values for specific applications and is the form recommended by TRL447. However, various formats are used by different standards. BS 1377–3 recommends reporting the water-soluble sulfate as g/l SO_3 or as % SO_3 (soil samples only). The BS EN 1744–1 test results are also reported as % SO_3. Sulfate occurs as SO_4, so this is the most useful way to express the results; SO_3 concentrations can be converted to SO_4 by multiplying by 1.2, and % SO_3 can be converted to g/l SO_3 by multiplying by 5 because the extraction ratio is fixed at 2:1. For all tests involving sulfur compounds, it is important to be clear about which units are being used when results and test methods are quoted.

When determining the water-soluble sulfate content of a soil or rock sample, it is useful to measure the pH value of the 2:1 water extract at the same time; this is recommended in TRL447 Test 1. The pH value of a soil may be determined in accordance with BS1377–3, but there is no test method for the pH value of aggregates, so it is important to measure it during the water-soluble sulfate test. The pH value may be determined in accordance with Annex N.5 of BS EN 1977–3, but this merely states that the electrochemical method (i.e. BS 1377–3 method) is the preferred method.

The water-soluble sulfate content is assigned the symbol WS in guidance documents such as TRL447 and BRE Special Digest 1. The water-soluble sulfate test will extract readily

soluble sulfates such as epsomite and mirabilite. Gypsum is sparingly soluble in water at neutral pH, so if this is the only salt present WS will not exceed about 1500 mg/l SO_4. If the pH is acidic, however, much higher values of WS can be obtained even if gypsum is the only sulfate present. Determination of the pH value, thus, helps to interpret the WS value and assess the threat posed by the material or water.

37.3.3 Acid-soluble sulfate (AS)

While the WS test determines the soluble sulfate content of a soil or rock in contact with distilled water, the total sulfate content may be significantly higher than this if the sulfate is in the form of sparingly soluble minerals such as gypsum. However, if the material was in contact with acidic water, the soluble sulfate content could rise considerably and this could pose a hazard to civil engineering works and the environment. It is, therefore, useful to determine the acid-soluble sulfate content. Test methods for this are given in TRL447 (Test 2), BS1377–3 and BS EN 1744–1. The tests involve grinding a sample to pass through a 2 mm sieve and extracting it with an excess of 10% hydrochloric acid to dissolve all the sulfate present. The sulfate content is determined from the extract gravimetrically by precipitation of barium sulfate (BS 1377–3 and BS EN 1744–1) or instrumentally by ICP-OES or similar methods (TRL447). The BS 1377–3 and BS EN 1744–1 test results are reported as % SO_3. The TRL447 Test 2 results are reported as % SO_4. The SO_3 values can be converted to SO_4 by multiplying by 1.2. Sulfate occurs as SO_4, so this is the preferred term for test results. As with the WS test, when instructing tests and assessing the results it is important to be clear what test methods are to be used and how the results are to be reported.

The acid-soluble sulfate content is assigned the symbol AS in guidance documents such as TRL447 and BRE Special Digest 1. It is used directly as a limiting value for some applications for fill materials, but can also be used in combination with the total sulfur content to provide an indirect estimate of the sulfide content. The AS content will include all potentially reactive sulfate minerals, but unreactive sulfates such as barytes are not mobilised by the extraction.

37.3.4 Total sulfur (TS)

The total sulfur content of a soil or rock will include the sulfur present as sulfate, sulfide and other forms of sulfur. These other forms include organic sulfur, which can be present in mudstones, overconsolidated clays and alluvial strata and sulfur present as inert minerals such as barytes, which occur as minor constituents in many limestones, sandstones and other rocks. Other forms of inert sulfur compounds are found in recycled concrete aggregate; many of the additives added to concrete mixtures contain organic sulfur compounds, and gypsum is often added to cement at up to 3% by weight to retard setting.

Two methods for determining total sulfur are given in TRL447 (Test 4 Procedures A and B); procedure A involves microwave digestion to bring the sample into solution followed by determination of the sulfur content by ICP-OES; procedure B involves direct determination of the total sulfur content by rapid high-temperature combustion (HTC) analysis. The results are reported as % S. In practice, the HTC method is more widely used as it is much quicker and cheaper. A chemical method for finding the total sulfur content is given in BS EN 1744–1; in this method, sulfides are converted to sulfates by treatment with hydrochloric acid and hydrogen peroxide, which is a powerful oxidising agent, and the sulphate content is determined gravimetrically as before. The results are given as % S.

The total sulfur content is assigned the symbol TS in guidance documents such as TRL447 and BRE Special Digest 1. It is rarely used directly as a limiting value, but is used to calculate two parameters that are used as limiting values:

- The total potential sulfate content (TPS) is the total amount of sulfate that could be generated if all the sulfur compounds in the sample were converted to sulfate. It, thus, gives a worst-case figure for the sulfate threat from a particular material. It is calculated as:
  - TPS (% SO_4) = 3 × TS (% S).
- The oxidisable sulfides content (OS) is the sulfate that could be produced if all the sulfur present as sulfides in the sample is oxidised to sulfates. It is, thus, lower than the TPS and is usually calculated indirectly from TS and AS as:
  - OS (% SO_4) = [3 × TS (% S)] – AS (% SO_4).

These three tests – WS, AS and TS – are the main ones that are routinely performed on soils and rocks. They have the advantage of being relatively rapid and cheap, thus allowing a large number of samples to be tested without incurring excessive cost or delay. This is important as sulfur compounds can be highly variable in soils and rocks, so it is necessary to carry out a number of tests to establish the range of concentrations that is present. Both TRL447 and BRE Special Digest 1 recommend that at least five samples should be tested for any material proposed for use as backfill to structures, with the mean of the highest two values used for comparison with limiting values.

37.3.5 Total reduced sulfur and monosulfides

The content of reduced sulfur compounds can be estimated from the difference between TS and AS (see section 37.3.4); however this figure will also include other inert forms of sulfur such as organic sulfur or barytes, and will, hence, be an overestimate of the sulfide content. It will also be of limited accuracy, particularly at low values, as there will be error margins associated with the measurement of both TS and AS. A direct test for the reduced sulfur content would improve the accuracy of the estimates. A number of methods were investigated by Reid *et al.* (2005), and chemical methods were developed for the total reduced sulfur content (TRS) (TRL447 Test 3) and the monosulfide content (MS) (TRL447 Test 5). However, these are specialist chemical tests that should only be carried out by skilled personnel; unlike the tests for WS, AS and TS they

are not automated and are liable to be expensive. They may be useful in specific situations to identify the nature of sulfur compounds present in a material in more detail. The results are reported as % S. A similar test to the TRS test is given in BS EN 1744–1 for acid-soluble sulfides.

Oxidisable sulfides (OS) can be determined directly from TRS as:

$$\text{OS } (\%\ SO_4) = 3 \times \text{TRS } (\%\ S).$$

37.3.6 Other tests

Techniques such as X-ray diffraction (XRD) can be used to test for the presence of pyrite, gypsum and other minerals on a qualitative basis in soils and rocks. This technique also gives information about other minerals in the material, such as carbonates and clay minerals. X-ray fluoresence (XRF) analysis of soils or rocks can give the overall chemical composition, including an estimate of the total sulfur content, but is less accurate than direct chemical determination. Scanning electron microscopy (SEM) can be used to investigate the crystal form and grain size of fine-grained sulfur compounds, as in **Figure 37.5**. This can be very helpful in understanding how reactive the compounds may be in a particular situation; however, the technique is expensive and not suitable for routine testing purposes.

37.3.7 Reactive sulfides

The oxidation of sulfides to sulfates in fill materials can be a major source of problems in civil engineering works; however, the extent to which this will occur in any given situation is hard to estimate. Results from four sites are given in **Table 37.2** and show a very wide range of oxidation rates. Where large volumes of fill are involved, however, even a small amount of sulfide oxidation can have serious consequences, particularly where critical structural elements such as metallic reinforcement or anchors are concerned. Limiting values for sulfides are usually based on the OS value, on the assumption that all the sulfides in the material will oxidise to sulfate. This will be a conservative assumption in most cases (**Table 37.2**), particularly where pyrite is present in a coarse-grained crystalline form where it will be very slow to react. The difficulties of measuring the sulfide content accurately have already been referred to in sections 37.3.4 and 37.3.5, adding to the difficulties in setting realistic limiting values for this parameter. It would be useful to have a test method for reactive sulfides as well as the OS test, in a similar way to the WS and AS tests for sulfates.

To address this issue, Reid and Avery (2009) investigated an accelerated weathering test. This involved grinding samples to pass through a 2 mm sieve and putting them in a climate chamber at 40ºC and 95% humidity for 28 days. Measurements of the pH value and conductivity, using the BS 1377–3 method, were taken at 0, 7, 14, 21, 28 and 35 days. The aim was to provide an atmosphere conducive to oxidation of any sulfides that were present; oxidation of sulfides would result in a rise in conductivity and a drop in the pH value if the amount of acid produced exceeded the buffering capacity of the material. For limestone, other calcareous materials and recycled aggregates with a high proportion of concrete, the buffering capacity would probably be greater than the amount of acid produced, so little or no drop in pH value would be expected; however, oxidation of sulfides would be revealed by a rise in the conductivity over the period of the test.

Trials were carried out with five commercially available aggregates, including two granites, a limestone, a recycled aggregate produced from highway arisings (concrete and asphalt) and a recycled aggregate produced from demolition and excavation material (brick, concrete and soil). A shale with visible pyrite and an acidic colliery spoil were also included in the trial. The results for pH and conductivity are shown in **Figures 37.6(a)** and **(b)**. The pH and conductivity of the aggregate samples did not change during the trial, but the pH of the shale dropped from 7.5 to 5.7 and the conductivity rose from 380 to 2555 µS/cm. This suggested that oxidation of the pyrite

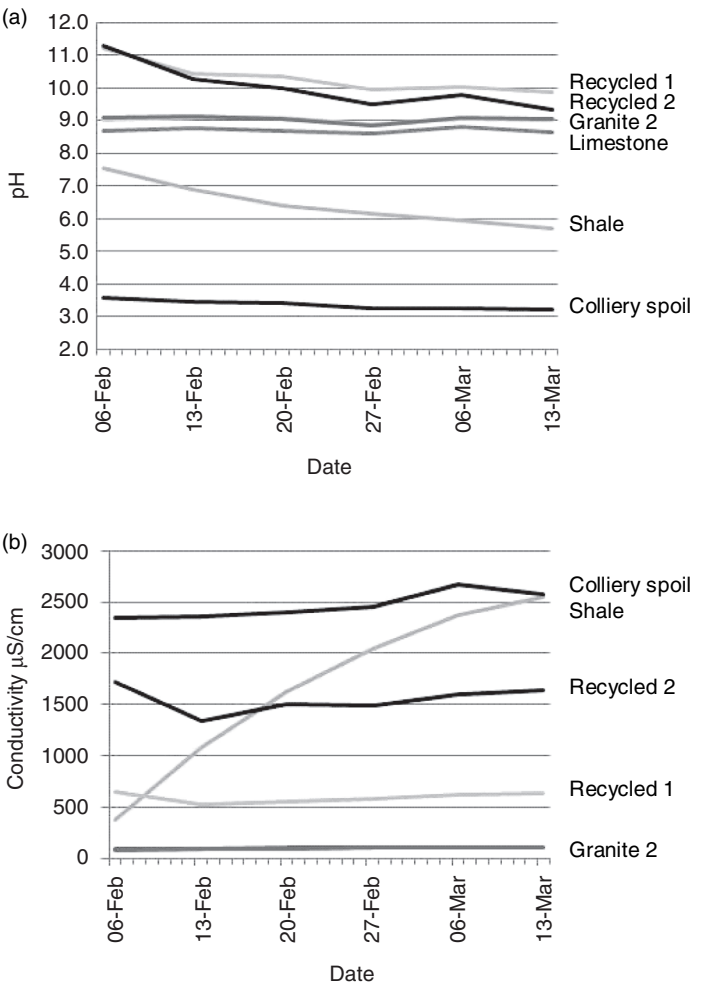


Figure 37.6 Change in pH value and conductivity during accelerated weathering test

Reproduced by permission of Mineral Industry Research Organisation (MIRO) from Reid and Avery (2009)

in the shale occurred during the test and correlated well with the behaviour of the material in the field; it is an overburden material from a quarry and is known to give acidic drainage after it has been exposed to the weather for some time (Reid and Avery, 2009).

The trial suggested that the accelerated weathering test may be a way to distinguish materials with reactive sulfides from those where the sulfides are not reactive. Further work is required to see if the method can be developed into a routine test method. It has the advantage of simplicity but requires a period of 28 days to carry out. It may, therefore, be appropriate as a characterisation test for aggregates or borrow pit materials, which would enable their suitability for various applications to be assessed in conjunction with the WS, AS and TS tests discussed above.

37.3.8 Test methods used for limiting values

Over the years, limiting values for different sulfur compounds have been set for different applications. The specific limiting values will be discussed where appropriate in section 37.4. The following are the main parameters for which limiting values are given in various specifications at the present:

- WS and pH value of groundwater;
- WS and pH value of soils and rocks;
- OS (usually derived from TS and AS) of soils and rocks;
- TPS (derived from TS) of soils and rocks;
- AS of soils and rocks.

In specific situations further information may be required, for instance:

- Magnesium content of groundwater or soil: water extract for concrete in brownfield locations (BRE, 2005).
- Chloride ion content, organic content, resistivity, redox potential and microbial activity index for backfill to anchored earth and reinforced soil structures (Highways Agency *et al*., 2009a).

It is important to remember that sulfur compounds are only one aspect of the chemistry of a material or water, and that it may be necessary to undertake a wider analysis to establish if there are likely to be any problems and to design mitigating measures.

It is important that sufficient samples are tested to obtain a clear picture of the concentration and distribution of sulfur compounds in the ground to enable selection of appropriate characteristic values for comparison with published limiting values for various applications [see Section 37.3.4, TRL447 (Reid *et al*., 2005) and BRE Special Digest 1 (Building Research Establishment, 2005)]. However, the limiting values for OS are known to be conservative (see section 37.3.7), so provision was made in TRL447 (Reid *et al*., 2005) for materials that exceed the limiting values for OS and TPS to be used if it can be established to the satisfaction of the client that:

- the material has been used in the past for the proposed application without leading to problems with sulfur compounds; and
- the reason why the material will not cause a problem is known, based on an understanding of its chemistry and mineralogy.

These provisions are included in the Notes for Guidance for the Specification for Highway Works, NG600 Series (Highways Agency *et al*., 2009b). The accelerated weathering test may provide a more quantitative method for assessing the risk from reactive sulfides in the future.

37.4 Specific problems and how to assess them

37.4.1 Attack on concrete

It has been known for over 70 years that sulfates and acids in the ground can attack concrete, resulting in cracking, expansion and softening of the concrete in the worst cases. Guidance on how to assess the risk and design concrete to withstand attack has been issued by the Building Research Establishment in a series of digests and guidance notes dating back to 1939. The most recent version is BRE Special Digest 1, published in 2005. This sets out detailed guidance for all parties involved in the design and construction of concrete structures, including the geotechnical specialist, who is responsible for carrying out the site investigation and making an assessment of the design sulfate class (DS class) and the aggressive chemical environment for concrete class (ACEC class) for the structure and passing this information to the designer of the building or structure.

To make an assessment of the DS class and the ACEC class, the geotechnical specialist has to carry out a desk study and site walkover to identify the type of site – e.g. whether it is a brownfield site or not – and assess whether the ground conditions are likely to be aggressive to concrete. Particular care is needed if the ground is liable to contain pyrite, as this increases the risk of attack on concrete. The groundwater conditions have to be assessed and classified as static, mobile or flowing, and concentrations of aggressive chemicals in soil and groundwater determined. A series of flowcharts and tables are then used to assign the DS class and ACEC class, from which the designer can produce a mix design and additional protective measures that will ensure the concrete can resist the anticipated chemical conditions. Flowcharts are given for three different types of site:

- locations on natural ground sites except where soils may contain pyrite;
- locations where disturbance of pyrite-bearing natural ground could result in additional sulfate; and
- locations on brownfield sites except where soils may contain pyrite.

Five DS and ACEC classes are defined, ranging from low concentrations of sulfates and sulfides and neutral pH values for DS-1 and AC-1 to very high values of sulfates and sulfides and highly acidic pH values for DS-5 and AC-5. The ACEC Classes are labelled AC-1 rather than ACEC-1. The system does not exclude any materials, but provides the designer with measures

to design concrete to withstand whatever the anticipated ground conditions will be. There are various subdivisions of the AC classes depending on factors such as whether the groundwater is static or mobile. The higher the AC class, the more expensive the concrete will be, so it is important to strike a balance between prudence and overdesign. It is, thus, important to carry out an adequate number of tests to be able to establish the DS class of the site with a degree of certainty; if only a few WS results are available, the highest measured sulfate concentration should be taken as the characteristic value. If five to nine WS results are available, the mean of the highest two sulfate test results should be taken as the characteristic value.

The limiting values for the DS and AC classes are for design purposes rather than pass/fail criteria. However, limiting values for backfill to concrete, cement-bound materials, other cementitious materials or stabilised capping forming part of the permanent works are given in the UK Specification for Highway Works (Highways Agency *et al.*, 2009a), Clause 601.14. These limiting values are based using the top of the DS-2 class from Special Digest 1 and roughly correspond to the solubility of gypsum at neutral pH values. The limiting values are, as at November 2009:

- WS content not exceeding 1500 mg/l SO_4 (TRL447 Test 1);
- OS content not exceeding 0.5% SO_4 (TRL447 Tests 2 & 4); and
- pH value not less than 7.2.

These limiting values are designed to ensure that materials placed directly against the concrete will not cause sulfate or acid attacks; this is subtly different to the BRE Special Digest 1 approach, which is based on the existing ground conditions. Backfill to structures can either be generated from arisings on site or imported to site if suitable material is not available on site. Hence, it is necessary to set pass/fail limits for this particular application.

While Special Digest 1 relates to concrete in the ground, structural concrete may often be exposed to fill materials above ground or subject to percolation of water through overlying embankment fill that then comes into contact with the foundation concrete, particularly for bridges. When assessing the DS and AC classes, the composition of any fill material that impacts on the concrete should be considered. An embankment is potentially a highly oxidising environment, so if materials containing active sulfates and sulfides are placed in an embankment, the implications for concrete structures on the site need to be assessed. The limiting values for backfill in the Specification for Highway Works normally only apply to material up to a distance of 500 mm from the structure, so it is possible for fill behind this clean structural backfill layer to generate acidic, sulphate-rich drainage that could attack the foundation concrete.

Problems could arise with bulk fill materials such as colliery spoil that are known to frequently contain high concentrations of sulfates and sulfides (section 37.2.3.3). This was the case in two highway improvement schemes in Yorkshire and Humberside; both required the import of large volumes of bulk fill for embankments, and colliery spoil was the most readily available material. Chemical testing showed that the colliery spoil had high concentrations of sulfates and sulfides; while it was suitable for use as bulk fill, it would not have been suitable in locations where it could shed drainage onto structural concrete. The problem was solved by increasing the area of clean structural backfill – a local limestone – so that it formed a wedge rising over the colliery spoil in a series of benches away from the structure. In this way the colliery spoil was always below the structural backfill and, hence, could not drain into it. This enabled large volumes of colliery spoil to be used as bulk fill, preserving primary aggregates for higher value uses such as structural backfill.

37.4.2 Attack on structural metallic elements

Metallic elements forming part of the permanent works are particularly susceptible to corrosion from sulfates, sulfides and acids; hence, it is very important to ensure that they are not exposed to drainage from fill materials that contain such compounds. Applications where structural metallic elements are used in civil engineering include reinforcing elements in anchored earth and reinforced soil, and corrugated steel buried culverts. The commonest metals used are galvanised steel and stainless steel. There are limitations on the materials allowed as fill to these structures – argillaceous rock is specifically excluded, for example – and on their chemical properties. Key references are Table B.1 of BS EN 14475: 2006, Execution of special geotechnical works – Reinforced fill, and Tables 6/1 and 6/3 of the UK Specification for Highway Works (Highways Agency *et al.*, 2009a). Guidance is also given in the UK Design Manual for Roads and Bridges in BD12/01 Design of Corrugated Steel Buried Structures with Spans Greater than 0.9 Metres and up to 8 Metres (Highways Agency *et al.*, 2001) and BD70/03 Strengthened/reinforced soil and other fills for retaining walls and bridge abutments (Use of BS 8006: 1995) (Highways Agency *et al.*, 2003). The recently published revision of BS 8006 (British Standards Institution, 2010) refers to Table B.1 of BS EN 14475 for electrochemical properties of fills used with metallic reinforcement.

It has been known for many years that some materials, such as colliery spoil and argillaceous rocks in general, are liable to cause corrosion of structural metallic elements (West and O'Reilly, 1986; Winter *et al.*, 2002), and these materials are not permitted as backfill to reinforced soil, anchored earth or corrugated steel buried structures in the Specification for Highway Works (Highways Agency *et al.*, 2009a). However, there have been problems with other materials such as alluvial sands and gravels (Reid *et al.*, 2005); acid generated by pyrite oxidation caused corrosion of a number of corrugated steel buried culverts in a highway scheme in England (**Figure 37.7**). The culverts and the backfill material had to be replaced before the road could be opened, causing considerable delay and expense.

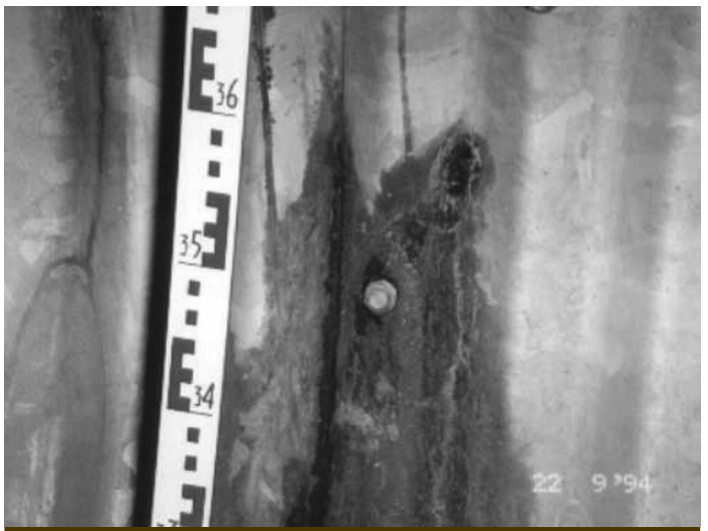


Figure 37.7 Corrosion of corrugated steel buried culvert due to pyrite in alluvial sand and gravel used as backfill

		Within 500 mm of structural metallic elements (Clause 601.15, UK Specification for Highway Works)	
Parameter	**Units**	**Galvanised steel**	**Stainless steel**
WS	mg/l SO_4	300	600
OS	% SO_4	0.06	0.12
Min pH	Units	6	5
Max pH	Units	9	10

Table 37.3 Limiting values for backfill to structural metallic elements

The limiting values for backfill vary slightly between galvanised and stainless steel, and are shown in **Table 37.3**.

Limiting values are also given for chloride content, organic content, resistivity, redox potential and microbial activity index. These values do not apply to backfill to ancillary metallic structures such as manholes and drain covers. Where the metallic elements are encased in concrete, the limiting values for backfill to concrete should be used (see section 37.4.1). The limiting values are very tight because of the sensitivity of the applications, and should only be applied to backfill to structural metallic elements, not to more general applications such as unbound subbase, capping or general fill.

A number of natural and recycled aggregates have been found to have OS values in excess of those in **Table 37.3**, although they have been used successfully for backfill to structural metallic elements in the past. The investigation carried out by Reid and Avery (2009) using an accelerated weathering test showed that five commercially available aggregates did not appear to contain reactive sulfides (see section 37.3.7), although measurement of the OS values showed that three of the five exceeded the limiting values in **Table 37.3**. The accelerated weathering test, thus, suggests that these materials would be suitable as backfill to structural metallic elements. Problems are unlikely to arise with aggregates from sources with a long history of production, as they are likely to have been thoroughly tested and any problems would have been identified. Problems are more likely where new sources are being developed and with borrow pit materials, which may be more variable and less well investigated. The problems with the alluvial sand and gravel discussed by Reid *et al*. (2005) were in this category; in this case a borrow material obtained on site was used rather than an aggregate supplied from an external quarry.

37.4.3 Attack on sheet pile walls

Steel sheet piles are widely used in civil engineering and are susceptible to corrosion from acid, sulfates and sulfides. In oxidising situations, e.g. above the water table in granular soils, corrosion will result from the mobilisation of sulfates and acidity in accordance with the reactions set out in section 37.2.2. However, in situations where oxygen is scarce or absent, a different set of reactions may occur; sulfates may be reduced to sulfides, leading in some cases to generation of highly corrosive monosulfides of iron or gaseous hydrogen sulfide. These reactions can occur in organic silty sediments in estuarine areas, which are often locations where extensive sheet piling is employed in harbour works. The high salinity associated with such environments, plus the diurnal change of water level due to tides, leads to conditions particularly suited to the formation of these reactive monosulfides. Tiller (1997) describes the mechanisms of corrosion specific to these environments together with various methods for combating them.

37.4.4 Degradation of bulk fill materials; geotechnical and environmental aspects

The consequences of pyrite oxidation in bulk fill materials in terms of potential attacks on concrete and structural metallic elements have already been described in sections 37.4.1 and 37.4.2. However, these reactions can also have significant effects on the fill material itself, both in terms of the long-term geotechnical properties and the leaching of polluted drainage waters into nearby watercourses.

The materials most susceptible to degradation of their geotechnical properties are weak rocks such as mudstones and shales, and materials such as colliery spoil, which contain large quantities of these primary components. These materials are widely used as bulk fill for highway and other embankments and for embankment dams; the latter, in particular, can require several million tonnes of fill and can reach heights of up to 40 m. They are also expected to have a service life of 200 years or more, so any potential long-term deterioration in their geotechnical properties is important, as this could lead to a decrease in the factor of safety and loss of serviceability. Embankment dams are generally constructed using the materials available within the footprint of the proposed reservoir, as

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it is seldom practicable or economic in the UK to import large volumes of fill material. It is, therefore, particularly important to understand any potential degradation of the fill materials and design the embankment to cope with it.

These issues were investigated thoroughly for two major embankment dams in the UK during the 1980s and 1990s. The dams were Carsington in Derbyshire and Roadford in Devon; both were constructed using locally available weathered mudstones of Carboniferous age, which contained significant quantities of sulfide and sulfate minerals. Among the potential issues identified were:

- loss of shear strength of the fill materials due to leaching by sulfuric acid;
- chemical attack on drainage materials by sulfuric acid;
- clogging of drains by precipitation of gypsum or ochre;
- increased compressibility of the fill due to leaching with sulfuric acid;
- attacks on concrete and metal; and
- pollution of downstream watercourses by acidic drainage waters with high concentrations of metals and sulfates.

The results of the investigations and the records of measures taken during construction are described by Chalmers *et al.* (1993) for Carsington and Wilson and Evans (1990) and Davies and Reid (1997) for Roadford. It was concluded that chemical weathering was unlikely to lead to a significant loss of strength or performance of the fill, but an allowance was made for a drop of 1.5° in the peak drained friction angle (ø′) in the long term. Potential attacks on concrete and steel were avoided by taking appropriate protective measures in view of the likely sulfate and pH conditions. The biggest concerns were potential chemical attacks on drains and the treatment of drainage waters to protect high-quality watercourses downstream of the embankments.

The original Carsington Dam failed in 1984 when construction was nearly complete (Skempton and Vaughan, 1993); the embankment had included drainage blankets of limestone in the weak mudstone fill, and these had suffered chemical attack, leading to precipitation of gypsum in the drainage blankets (**Figure 37.8**) and the generation of carbon dioxide (see section 37.1). In the reconstruction of the embankment, drainage layers were not used and the basal drain and chimney drain were specified to be of inert quartz gravel to avoid any chemical reactions (Chalmers *et al.*, 1993). The basal drain was designed to be below the water table at all times, to avoid any oxidation of the iron-rich drainage water; consequently, when the drainage water emerged from the drainage outfall, the iron in solution oxidised and precipitated as ochre, an orange deposit of hydrous iron oxides (**Figure 37.3**). The drainage water was treated with sodium hydroxide to raise the pH and precipitate the iron and other metal oxides before being released into the downstream watercourse.



Figure 37.8 Limestone drainage blanket in acidic mudstone fill, Carsington Dam, showing precipitation of gypsum in the drain

At Roadford Dam, it was discovered at a late stage that the material proposed for the basal drain was calcareous and could potentially be attacked by acid draining from the embankment fill of weathered pyritic mudstone (Davies and Reid, 1997). As it was not practical to arrange for an inert fill material given the remote location of the site, investigations were carried out to assess the potential effect on the drainage blanket (Davies and Reid, 1997). It was found that the loss of permeability due to precipitation of gypsum would be minor due to the design of the drain, which again was permanently below the water table with a continuous flow of water through it. The drainage water was monitored during construction and found to have neutral pH and a high sulfate content, whereas drainage from the embankment fill had markedly acidic pH and even higher sulfate content. Chemical reactions between the drainage from the embankment fill and the drainage blanket were clearly occurring, but the drain has continued to function successfully for over 20 years since construction (Hopkins *et al.*, 2010).

The drainage water also had a high content of iron, manganese and suspended solids, and it has to be treated prior to release to the River Wolf downstream. The chemistry of the drainage water has been monitored for over 20 years and shows an annual cycle, with a peak in sulfate concentrations in the winter months when drainage from the embankment is greatest (**Figure 37.9**). The pH value shows the opposite trend, being lower in winter and higher in summer, but has remained above 7.0 except for a short period during the exceptionally wet winter of 2000/01 (**Figure 37.9**). The calcium concentration remains almost constant, indicating that the drainage water is saturated with calcium carbonate due to dissolution of calcite in the drainage blanket.

For both Carsington and Roadford Dams, a lot of useful information on the chemical reactions that could occur was obtained from observations of trial embankments constructed during the ground investigations. These were primarily designed to give

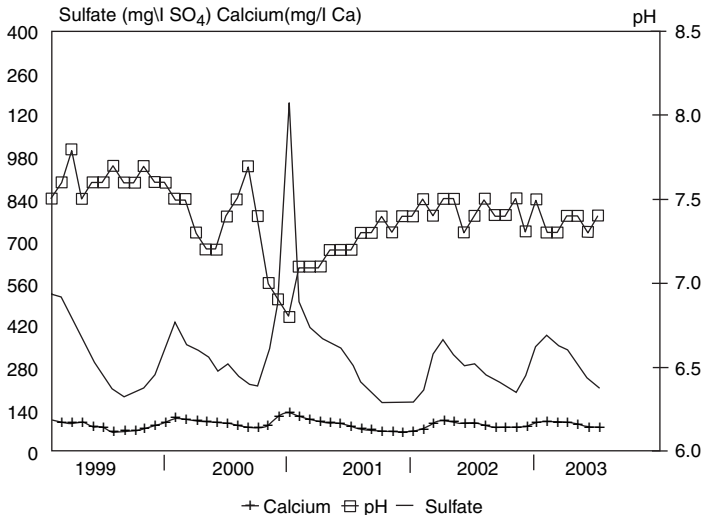


Figure 37.9 Concentrations of sulfate, calcium and pH value in drainage water from Roadford Dam
Reproduced by permission of South West Water from Hopkins *et al.* (2010)

information about the compaction behaviour of the various fill materials but also yielded much information about the nature and scale of the chemical reactions. Observations of precipitates of ochre and gypsum in drainage pools at the toes of the embankments and precipitates of white powdery gypsum on the surface of the embankments a few weeks after construction were a very visible indication of the chemical reactions that occurred when the materials were excavated and exposed to air and water. Chemical analyses of the drainage waters gave an indication of the scale of the problem and enabled design decisions to be made to deal with it. Much of this might have been missed had it not been for the sharp eyes and enquiring minds of the inspectors and engineers supervising these trial embankments. It is important for engineering geologists and geotechnical engineers to be able to recognise the signs of chemical reactions and understand their significance. Trial embankments are a particularly useful way of assessing the potential chemical reactivity of proposed fill materials, and provision should always be made for recording observations of any changes in the materials and chemical analysis of drainage waters.

Where it is not possible to construct trial embankments, engineers should try to examine embankments of similar materials and obtain information about their performance, including any issues with poor quality of drainage waters. Predictions can then be made for the performance of the proposed embankment material. The accelerated weathering test described in section 37.3.7 may also help to assess the likely scale of reactions in the full embankment.

A material that is particularly susceptible to chemical reactions is colliery spoil. In the aftermath of the Aberfan tragedy, a lot of research was carried out to assess the geotechnical properties of colliery spoil, including the effects of chemical weathering on shear strength; the results are summarised by Taylor (1984). The worst-case long-term residual shear strength was estimated to be an apparent effective cohesion (c') of zero (kN/m^2) and internal friction angle (ϕ') of 22°. The specific effect of acid leaching due to sulfide oxidation on the shear strength of mudstone was investigated by Anderson and Cripps (1993). Colliery spoil can be used successfully for bulk fill in embankments, as illustrated by the two examples of highway schemes in section 37.4.1, provided that its properties are known and the works are designed to ensure that they do not cause geotechnical or environmental problems.

37.4.5 Heave of stabilised soils

The treatment of soils such as sands, silts and clays with binders such as lime, pulverised fuel ash, slag or cement is often carried out to improve the geotechnical properties of the materials. This may simply involve drying out wet soils to make them suitable for use as bulk fill materials, but may also include creating stronger layers *in situ* for use as capping or as an unbound subbase in place of imported granular materials. This topic is covered in more detail in the Design Manual for Roads and Bridges HA74/07, Treatment of fill and capping materials using either lime or cement or both (Highways Agency *et al.*, 2007). Reactions between the binders and the soil result in the formation of cementitious compounds in a strongly alkaline environment at pH values in excess of 12. These extreme conditions can result in various chemical reactions with adverse consequences for the treated materials.

At these high pH values, sulfates in the soil react with the cementitious compounds to form a highly expansive calcium sulfo-aluminate hydrate called ettringite ($3CaO.Al_2O_3.3CaSO_4.31H_2O$) and gypsum; these minerals also cause expansive attacks on concrete (section 37.4.1). This is compounded because the high pH conditions accelerate the oxidation of pyrite, thus, generating even more sulfate to react with the stabilised soil. The reactions often do not occur immediately, but several months later, possibly due to seasonal changes in the moisture content supplying the water required for the formation of the expansive minerals. By this time, the treated layer has been overlain by pavement layers, foundations or floor slabs, and heave of the stabilised layers causes disruption to the pavement or building. This is then very expensive to remediate, as the overlying structure has to be broken out to replace the expansive material below. An example of the reactions that can occur and the remedial works that were required for a major highway scheme in the UK are given by Snedker and Temporal (1990).

Guidance on how to assess materials for potential swelling is given in HA 74/07 (Highways Agency *et al.*, 2007) for bulk fill and capping applications. Rather than setting specific limiting values, engineers are required to go through a process of trial mixtures, which are then allowed to soak for 28 days and the swell measured; if the swell exceeds acceptable limits, the material should not be used. Materials that are particularly susceptible to these reactions include overconsolidated clays,

drift deposits derived from them, and colliery spoil (see section 37.2.3). Where soil is to be stabilised *in situ* to form a hydraulically bound subbase layer, a limit of 0.25% as the SO_4 content for the total potential sulfate (TPS) is set in the Specification for Highway Works (Highways Agency *et al*., 2009a, Clause 840.3). Soil with a TPS equal to or greater than 0.25% can only be treated where this has been agreed with the overseeing organisation.

Unlike use as bulk fill (section 37.4.4), where the potential issues can be worked through in many cases, there are few options available for stabilisation if the material to be treated contains significant quantities of sulfates or sulfides. There is some evidence that the use of granulated ground blast-furnace slag leads to less swelling compared to cement or lime (Higgins *et al*., 2002). However, great care should be taken when assessing the potential for stabilisation of clay soils to ensure that they will not be susceptible to swelling.

37.4.6 Heave of floor slabs

Heave of floor slabs can also occur due to precipitation of gypsum in the underlying fill material under certain conditions. This can occur where the underlying material contains high sulfate levels and there is a high water table. As the interior of the building is heated, groundwater is drawn into the fill material below the floor slab by capillary action. The water evaporates, leading to crystallisation of gypsum in the void spaces in the fill material. This generates very high swelling pressures, which cause heave and disruption of the floor slab.

This type of problem is particularly common where buildings are founded on shales, mudstones or overconsolidated clays that contain significant concentrations of sulfides and sulfates. The disturbance due to construction works allows air to oxidise some of the sulfides, which are then transported by capillary action to the fill immediately below the floor slab, where they precipitate as gypsum. Examples from the UK have been reported by Hawkins and Pinches (1987) and Wilson (1987). The problem has been recognised for a long time and guidance was given by Nixon (1978), suggesting a limit of 1% FeS_2 in fill material below floor slabs.

While the majority of incidents have been due to construction directly on sulfate- and sulfide-rich strata with a high water table, similar problems can also occur if fill materials for use below floor slabs contain significant sulfates, sulfides or other expansive minerals. Guidance issued by the BRE (1983) specifies that all materials for use below floor slabs must be non-expansive.

37.5 Conclusions

The chapter has indicated how sulfur compounds can be a hazard in geotechnical engineering works, potentially leading to corrosion of concrete and steel, clogging of drains, loss of shear strength, heave of pavements and floor slabs and pollution of watercourses. These hazards can be avoided if they are recognised at an early stage and appropriate measures taken, either to adopt mitigating measures or to avoid use of unsuitable materials. Guidance is available to deal with most of the situations discussed; the main concern is for engineers to be aware of the potential dangers and to ensure that they are assessed properly during the ground investigation and design stages of the project. This does not just involve carrying out laboratory tests and comparing the results against various sets of limiting values, but understanding the materials and how they will behave when used in the proposed engineering works. In other words, exactly the same approach as in all geotechnical engineering design and construction work.

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37.6.1 Further reading

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37.6.2 Useful websites

The UK Specification for Highway Works and Design Manual for Roads and Bridges; www.dft.gov.uk/ha/standards.

It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 40 *The ground as a hazard*
- Chapter 48 *Geo-environmental testing*
- Chapter 49 *Sampling and laboratory testing*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 38
Soluble ground

Tony Waltham Engineering geologist, Nottingham, UK

Limestones, gypsum, salt and varieties of these are the rock materials soluble in natural water, which can, therefore, be eroded to create ground cavities, which in turn can be the cause of ground failure and surface subsidence. These are features of karst terrains. The dominant karst geohazard is the development of new sinkholes within soil profiles overlying cavernous rock, where the rock remains stable but the soil is washed downwards into the open fissures. Most of these are induced by engineering activity, therefore they are preventable. Engineering problems are also caused by highly irregular rockhead profiles, and by cavities that may underlie foundations. Assessment of karst ground is very difficult, but the broad characteristics of karst are now well known and should be appreciated in practice. Less well known is the karst hazard in sabkhas, with increasing instances of ground failure encountered, but poorly documented, in the construction boom in the Middle East.

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38.1 Introduction

Naturally occurring soluble ground materials are limestones (including dolomite and chalk), gypsum (and also anhydrite) and salt (halite), in order of decreasing abundance and increasing solubility. Exposure of all of these to rainwater, stream water and groundwater commonly leads to the development of subsurface cavities, with all the attendant consequences of collapse and ground subsidence. Long-term underground erosion creates cave networks, which can take all of the surface drainage. The essentially streamless landscapes, with their distinctive suites of landforms, are known as karst. Any karst terrain, on any of these rocks, presents some degree of geohazard, which may include slow ground subsidence, catastrophic collapses and destructive sinkhole development.

38.2 Soluble ground and karst

Soluble rocks present a variety of geohazards where they lie within a few tens of metres of the ground surface (and sometimes also at greater depths). Direct dissolution of the rock is only rapid enough to undermine a built structure within its lifetime in the case of salt, and to a lesser extent gypsum; in both cases any significant loss of ground would take place only where it is exposed to a significant flow of aggressive water. Limestone dissolution is so slow that it would take thousands of years to impact on a structure founded upon it. The hazards for limestone, and gypsum, are created by open fissures and cavities that are dissolved out of the bedrock over geological time scales. Soil-filled fissures within a buried rockhead commonly form very difficult ground for founding structures, but collapses of rock over open caves are rare. The major hazard is presented by bedrock voids, which are capable of swallowing large volumes of unconsolidated soil cover possibly within 12 hours or days; the consequence is the formation of the most common type of sinkhole.

38.2.1 Sinkholes

Sinkholes are closed depressions, 1–100 m in diameter and depth, that are the diagnostic landforms of karst terrains (Waltham *et al.*, 2005); they are correctly known as dolines by geologists, but the sinkhole term dominates in the American and the engineering literature. The term implies that the ground has sunk; water does normally sink into them too, but few have visible streams sinking into them. It is important to recognise the contrasting types of sinkhole (see **Figure 38.1**). Small numbers of collapse sinkholes, formed where the rock collapses into a cave below, are found in most karst terrains, but the chances of a new failure are so low at any site that they present a negligible risk to engineering. Nearly all large collapse sinkholes have expanded in sequences of failure events, further minimising the risk of massively destructive ground collapse.

The main karst geohazard is the formation of subsidence sinkholes in the soil cover over a fissured and cavernous limestone (**Figure 38.2**). Not only are these the most abundant, but new subsidence sinkholes can form by rapid movement of the soil cover and, therefore, have the greatest impact on engineered structures. The suffosion and dropout types differ due to the properties of their soil cover and, therefore, in their mode of failure – either the slow ravelling in a sandy soil or an arch failure into an undermined void in a more fine-grained soil. A soil-filled buried sinkhole constitutes an extreme form of rockhead relief, perhaps described as a soft spot, and sometimes developing a shallow compaction sinkhole in the surface above. Sinkholes may also develop purely by rock dissolution, but these are long-term erosional features comparable to valleys in non-karst

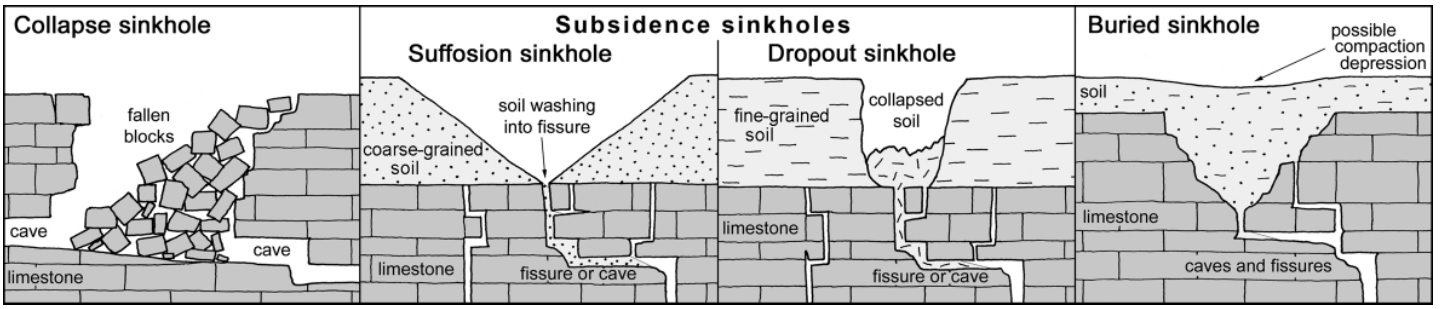


Figure 38.1 The main types of sinkhole developed in or above soluble rocks in karst terrains
Modified from Waltham and Fookes (2003)



Figure 38.2 A recently developed subsidence sinkhole in clay-rich alluvial soils in Turkey; this formed by a rapid collapse in the style of a dropout, but its slopes are already degrading to a wider profile; the bedrock is gypsum, which is visible at the outcrop in the background

terrains, and have little engineering significance except that the ground beneath is more likely to be more cavernous than adjacent ground. A shallow bowl, rather like a compaction sinkhole, can also be formed by localised rockhead dissolution, especially of salt. The engineering significance of both subsidence sinkholes and rock collapse are described in sections 38.3 and 38.4 below.

38.2.2 Distribution

Soluble rocks are widely distributed in all sedimentary sequences. Britain offers an assemblage of ground conditions that reasonably represents the proportions of the land area with soluble ground geohazards that can be found across the world (**Figure 38.3**). Limestones and varieties of carbonate rocks are the most widespread, with outcrops in almost every country in the world, but these do present considerable variation in the scale of their karst geohazards. It is notable in the British example that only some of the limestones are old, strong and cavernous, and, thereby, have widespread karst geohazards, while the weaker limestones rarely provide major difficulties for construction. Also, chalk is a special case, where rock strength and weathering are generally more significant than karstic conditions (Lord *et al.*, 2002). Worldwide, limestone karst causes the most extensive engineering difficulties in huge swathes of southern China, in large parts of the eastern USA and in the Dinaric karst that extends across the nations of the former Yugoslavia.

Gypsum is far less widespread than limestone. The small outcrops in the Midlands and northern England mean that Britain is perhaps under-represented when comparisons are made worldwide; the American mid-West and the Ukraine are the two large regions most impacted by their gypsum karst. Salt is of even more restricted extent, both in Britain and worldwide; its geohazards are well known in the developed lands of Britain and America, but it is the Middle East of south-western Asia that has the greatest areas directly underlain by salt.

38.3 Influences on the geohazard of limestone karst

Dissolution causes the most conspicuous karst features in pure, strong limestones (unconfined compressive strength, UCS > 70 MPa) where extensive fractures can be enlarged to create fissures and caves between blocks of intact strong rock. Weaker and softer limestones, such as England's Cotswold oolites, are more porous, so have more diffuse groundwater flows through micro-fissures; chalk also has diffuse flows, but much of its groundwater is transmitted through open fissures, and caves can occur (Lord *et al.*, 2002). On a broad scale, the age of the limestone is irrelevant; Carboniferous limestones in England, Jurassic limestones in France and Tertiary limestones in South East Asia have identical engineering properties. Dolomites and dolomitic limestones are rather less prone to dissolution, so generally have a reduced scale of karst features. Larger cavities develop where streams of aggressive water enter the limestone from adjacent impermeable rock outcrops, so there is a tendency for increased karst development adjacent to geological boundaries. Beyond that, rock structure and lithology influence cavity development, which is mainly at large fractures and chemically favourable inception horizons, and the guiding features can be recognised in most mapped cavities. But the distribution, pattern and positions of caves cannot be predicted within the hugely variable structure of natural ground conditions.

The scale of karst development may vary considerably, as recognised by a broad classification of karst ground conditions (see **Figure 38.4**). Any description and assessment of karst for engineering purposes should not only define the limestone

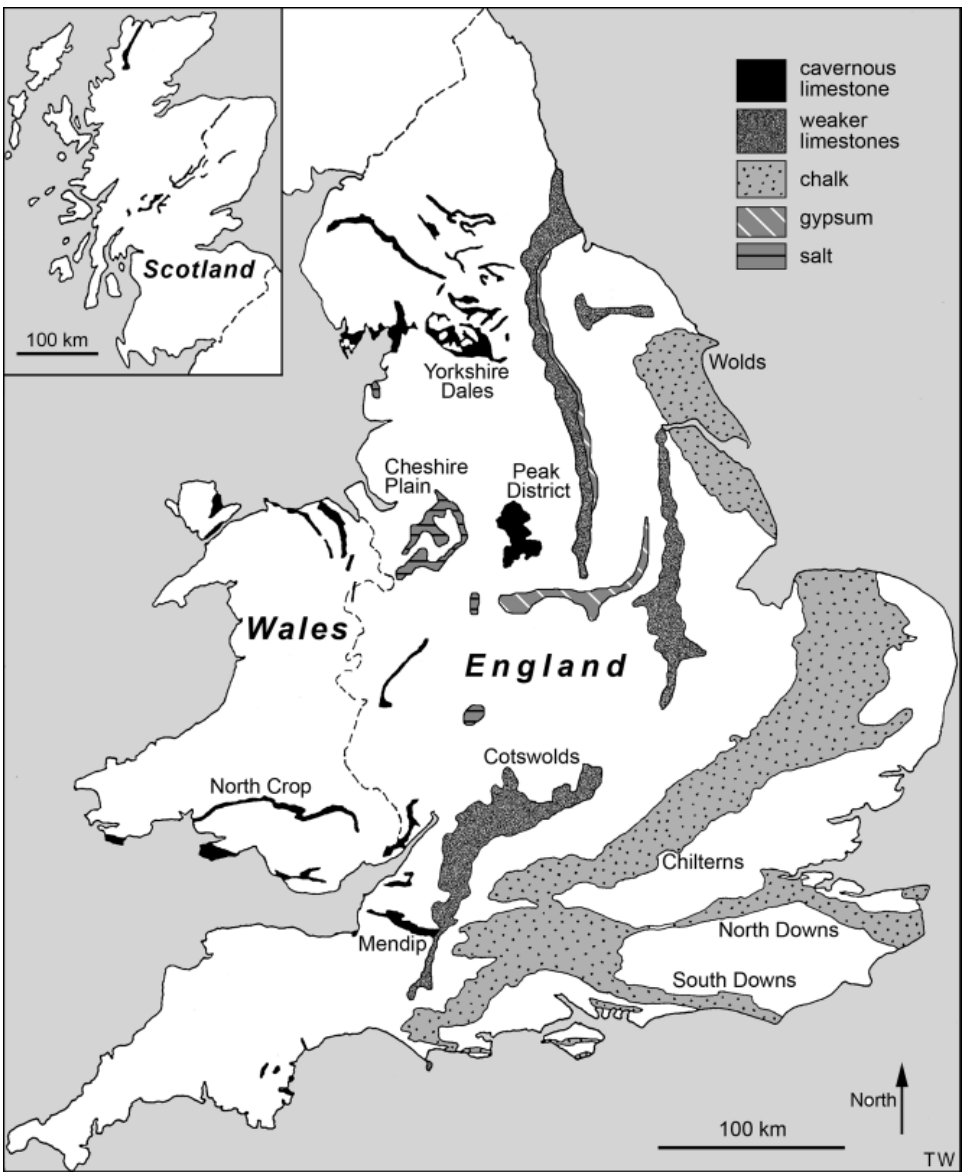


Figure 38.3 The distribution of soluble ground in Britain; all these areas are prone to karst geohazards, but recognisable karst landscapes are only widely developed on the cavernous limestones and on some of the chalk outcrops

lithology and the karst class, but should also determine or estimate approximate values for the three key parameters: typical cave size, frequency of new sinkholes and rockhead relief (Waltham and Fookes, 2003). Limestone dissolution in water is dependent on carbon dioxide to create soluble bicarbonate ions, and most dissolved carbon dioxide is derived from biogenic sources within soil profiles. Consequently, the scale of karst features, and, therefore, the karst class, is closely related to plant cover and, therefore, to the climate, and also to past climates. Karst in the colder climates of high latitudes and altitudes, such as in Britain and Canada, typically shows restricted development, to classes kII or kIII, with cave passages, rockhead fissures and rare new sinkholes all typically measuring less than 10 m. This contrasts with the humid tropics, such as in South East Asia or the Caribbean, where very well-developed karst terrains of classes kIV and kV have giant caves, pinnacled rockheads and numerous sinkholes, all with dimensions that approach or exceed 100 m. Limestones in hot and dry environments, such as in Australia and the Middle East, have modern karst development restricted by low rainfall, but commonly have isolated large caves and other karst landforms relict from wetter climates during the Pleistocene.

Though the natural environments dictate the overall scale of karst development, it is important to recognise that individual events of ground subsidence or collapse are commonly generated by man's activities that bring a sudden change to equilibrium situations. Engineering works that either remove or introduce water, or add an imposed load, all happen at far greater rates

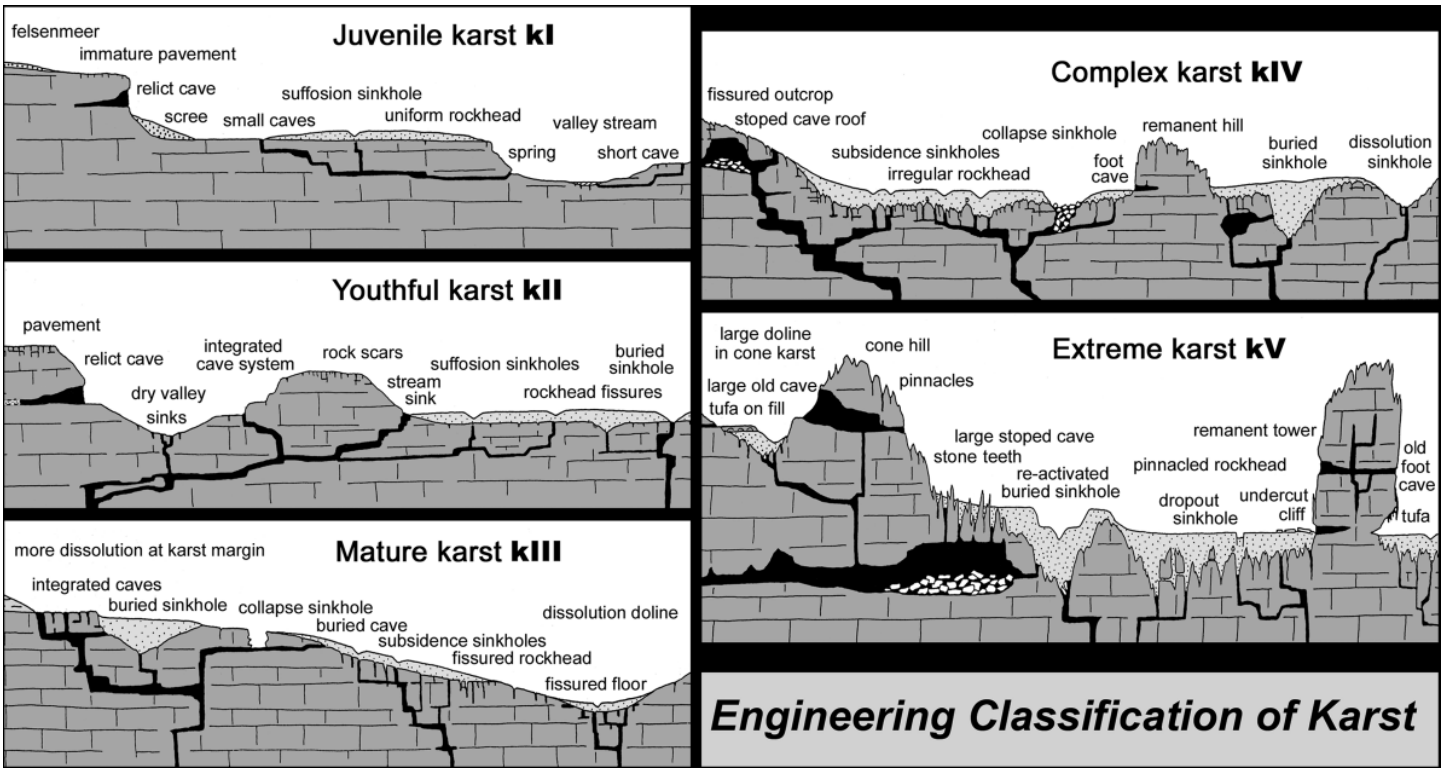


Figure 38.4 The five classes of karst that broadly demonstrate the variety and scale of landforms and ground conditions relevant to engineering
Modified from Waltham and Fookes (2003)

than normal geological evolution. In a natural and undisturbed karst, ground movements and collapses do occur, but typically as isolated events separated by hundreds or thousands of years. But any such event may be triggered prematurely by inappropriate engineering activity. A karst geohazard may be regarded as an event waiting to happen, unless appropriate precautions are exercised to avoid undue disturbance of the existing environment; these include thorough control of the drainage to avoid accelerated soil loss (see section 38.4.1), and the avoidance of excessive loads imposed on unstable rock (see section 38.5.1).

38.4 Engineering works on soil-covered limestones

The major geohazard of karst is the development of new subsidence sinkholes within the soil cover over fissured bedrock, because the process can develop very rapidly, well within structure lifetimes and even within construction periods (**Figure 38.5**), and without any imposed loading. Any unconsolidated soil lying over karst bedrock is prone to loss by downward migration, known as suffosion or ravelling, into the bedrock voids. Clean sand flows with ease and can, therefore, cause a slow lowering of the surface until the stable profile of a suffosion sinkhole is reached, with a throat at a bedrock opening (**Figure 38.1**). A soil with almost any clay content loses ground first from immediately above the rockhead, and it has the capability of developing a large soil void beneath an unstable soil arch. There is, therefore, no surface indication of the impending failure, until the thinned soil arch fails, instantly producing a dropout sinkhole (**Figure 38.1**). In reality, most soils have some degree of apparent cohesion, created in part by the negative pore water pressures, so they can develop soil voids and produce surface failures that vary between instantaneous and progressive; the profiles of most dropout sinkholes degrade into those of suffosion sinkholes either within a few days of the wall slumping or over longer periods. A thick soil mantle tends to reduce infiltration, such that the number of new sinkholes is greatly reduced in soils much more than about 10 m thick, but this is not an absolute limit, and ground failures have been recorded where the cavernous rock lies more than 100 m below the surface level.

38.4.1 Drainage and induced sinkholes

The suffosional removal of soil takes place almost entirely as downwashing by percolating water. It is, therefore, crudely predictable that many new sinkholes will develop during or soon after major rainstorms. However, the locations of new sinkholes are not predictable, as they will be located above open fissures that are unseen in the soil-mantled rockhead until the sinkhole develops (**Figure 38.6**). It is, however, significant that subsidence sinkholes are most likely to develop where and when there is a change to an existing equilibrium in rainfall infiltration and groundwater flow. Such a change



Figure 38.5 Collapse of a road on a bridge approach undermined by a new subsidence sinkhole in alluvial soils in Pennsylvania; this was one of many ground failures that developed within the zone of the decline of the water table around a deep quarry that was kept dry by continuous pumping



Figure 38.6 Numerous small subsidence sinkholes in a thin soil of glacial till over limestone in the Yorkshire Dales; there is no pattern to the sinkholes, as each has developed over an open fissure in the buried limestone, comparable to the exposed fissures in the bare patches of the limestone pavement

may be increased input at any point, either by concentrated run-off from a built structure, by inappropriate use of a soakaway or infiltration pond, or from a fractured pipeline. Equally, the change may be the increased drawdown of surface water due to a water table that has declined within an aquifer that is over-pumped. More than 90% of new sinkhole appearances are related to the disturbance of the drainage equilibrium by civil engineering activities (Newton, 1987; Waltham *et al.*, 2005).

Because most new sinkholes and karstic ground failures are induced by man's activities, the key feature to good engineering practice is to minimise or eliminate the hazard by control of the drainage. This includes the total collection of run-off from all built structures and areas of sealed ground, the proper disposal of that run-off directly into the bedrock or away from the site, the sound maintenance of all pipelines and stream channels, and the avoidance of increased infiltration when ground is exposed or replaced with granular fill during construction works. Failure or inadequate actions for any of these will almost inevitably lead to soil loss and sinkhole development, which will require far more costly remediation at some future date. Where sinkholes are induced by a decline of the water table, there may be no simple remedy that can be applied within the confines of a site or construction project. If that decline is due to groundwater pumping for municipal or private supply, to dewater ground to keep a quarry or mine dry, or temporary dewatering for a construction project, the costs of subsidence damage may simply have to be factored into that operation's budget, unless the economics determine a replacement water supply, a quarry closure or an alternative method of constructing deep foundations.

The placing of structural foundations within the soil profile over karst limestone is inevitable in the case of most roads, and also for many small, built structures over thick soil profiles. They can be perfectly appropriate where the proper drainage measures ensure that the ground is not unduly disturbed. Each project should be treated individually; foundations should be provided appropriate to that particular site (Sowers, 1996; Waltham *et al.*, 2005). A host of alternatives may involve engineered soils, geogrids, rafts and mattresses, partial excavations, replacement soils, load transfer to bedrock pinnacles, and reinforced foundations; all of these measures should be designed to minimise soil ravelling and to bridge any small voids that may develop subsequently (useful examples are provided by Vandevelde and Schmitt, 1988; Lei and Liang, 2005).

Grouting can be appropriate, but the sealing of all fissures within a karst limestone is likely to prove very expensive because huge quantities of fluid grout can be lost into large but unseen open voids. Compaction grouting, within the soil just above the rockhead, can effectively prevent the downward loss of soil, which is densified in the same process, and this commonly yields better results (Henry, 1987; Stapleton *et al.*, 1995).

Where an existing structure exhibits settlement damage following a history of stability, the key to remediation is to determine and rectify the feature that caused the new movement. In many cases this will be a drain or pipeline failure, and repairs may be all that is required to recover the integrity of the structure. Pressure grouting within the soil may be appropriate if action is not been taken before settlement damage becomes too severe. New open sinkholes commonly require repair, and this should entail backfilling with selected sizes of materials so that the sinkhole is choked to prevent further soil loss but it may be required to still drain surface water safely into bedrock fissures (Waltham *et al.*, 2005).

38.5 Engineering works on limestone bedrock

Foundations carried through to the bedrock eliminate the major dangers of soil movement and sinkhole development within the overburden, but karst ground buried beneath soil cover is

very variable and not easily assessed. It offers two major difficulties: an uneven, fissured or pinnacled rockhead, and the threat of open caves lying just beneath any footings.

Dissolution down limestone joints, which may occur before soil cover is emplaced but also matures beneath the soil cover, creates open or soil-filled fissures within the buried karstic surface that is the rockhead. These fissures may be widely spaced, irregular or closely spaced, leaving blocks or pinnacles of bedrock between them; a pinnacled rockhead has a forest of narrow rock pinnacles between networks of wide fissures (**Figure 38.7**). Individual blocks or pinnacles may be loose, due to dissolutional undermining along sub-horizontal fractures or beddings, and are only held in place by the surrounding soil; completely loose blocks within the soil are known as floaters. The depths of fissures, and the heights of pinnacles, are commonly some metres in any karst, and may be many tens of metres in tropical karsts of classes kIV and kV. It is, therefore, not abnormal to find depths to the rockhead varying by tens of metres at adjacent investigation boreholes or structural piles that are only a few metres apart. On a larger scale, rockhead profiles may include buried sinkholes where bedrock depressions up to 100 m or more across are completely filled with breakdown, sediment and soil. These may cause small surface settlements in the form of compaction sinkholes over the soft soil fills, and may have floors of highly fissured bedrock that offer no easily definable sound footing for deep foundations.



Figure 38.7 Pinnacled rockhead in mature limestone karst (of class kIV), exposed on a construction site in southern China; some pinnacles have already been broken down (with sledge hammers) to create a solid and almost level footing for a new hotel; the grey top of the pinnacle on the right originally projected above the soil level, as do some undisturbed pinnacles in the background

Most deeply fissured and cavernous karst limestones are strong materials, so structural loads may be carried safely on pinnacles that have been proven to be of adequate width and are not disconnected from the underlying bedrock. In such cases reinforced foundations can bear on the pinnacles and span the intervening soil-filled fissures and buried sinkholes. Driven piles with any significant end-bearing on rockheads of strong limestone (UCS > 70 MPa) present their own difficulties (see **Figure 38.8**); it can be very difficult to gain a safe seating on steeply sloping bedrock surfaces, so it is normally necessary to drill sockets into the rock, and this can also be difficult through a steeply inclined interface. In chalk terrains, the ground profile generally lacks any sharp contrast between soft soil and

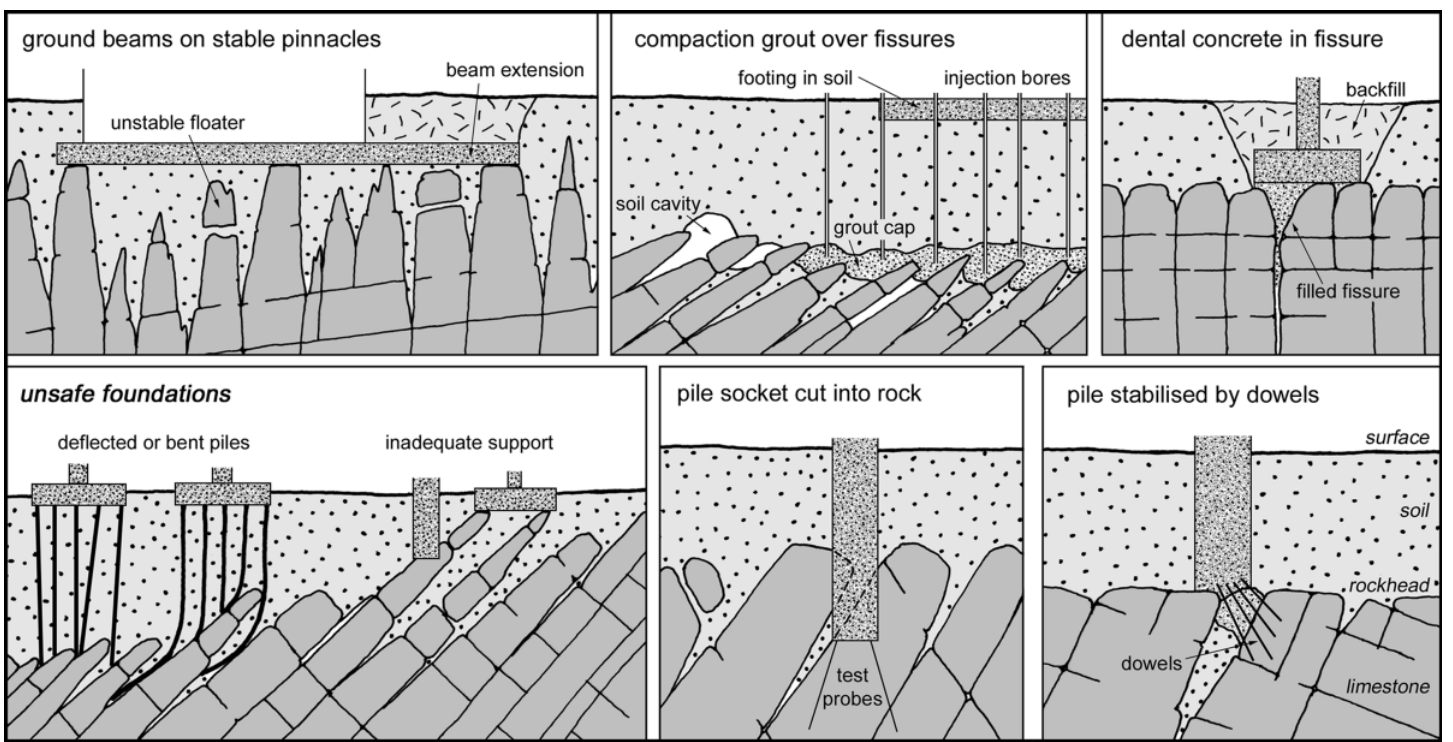


Figure 38.8 Various examples of good and bad foundations on soil-covered limestone karst (Waltham *et al.*, 2005), with due credit to the ideas and experience of the late George Sowers

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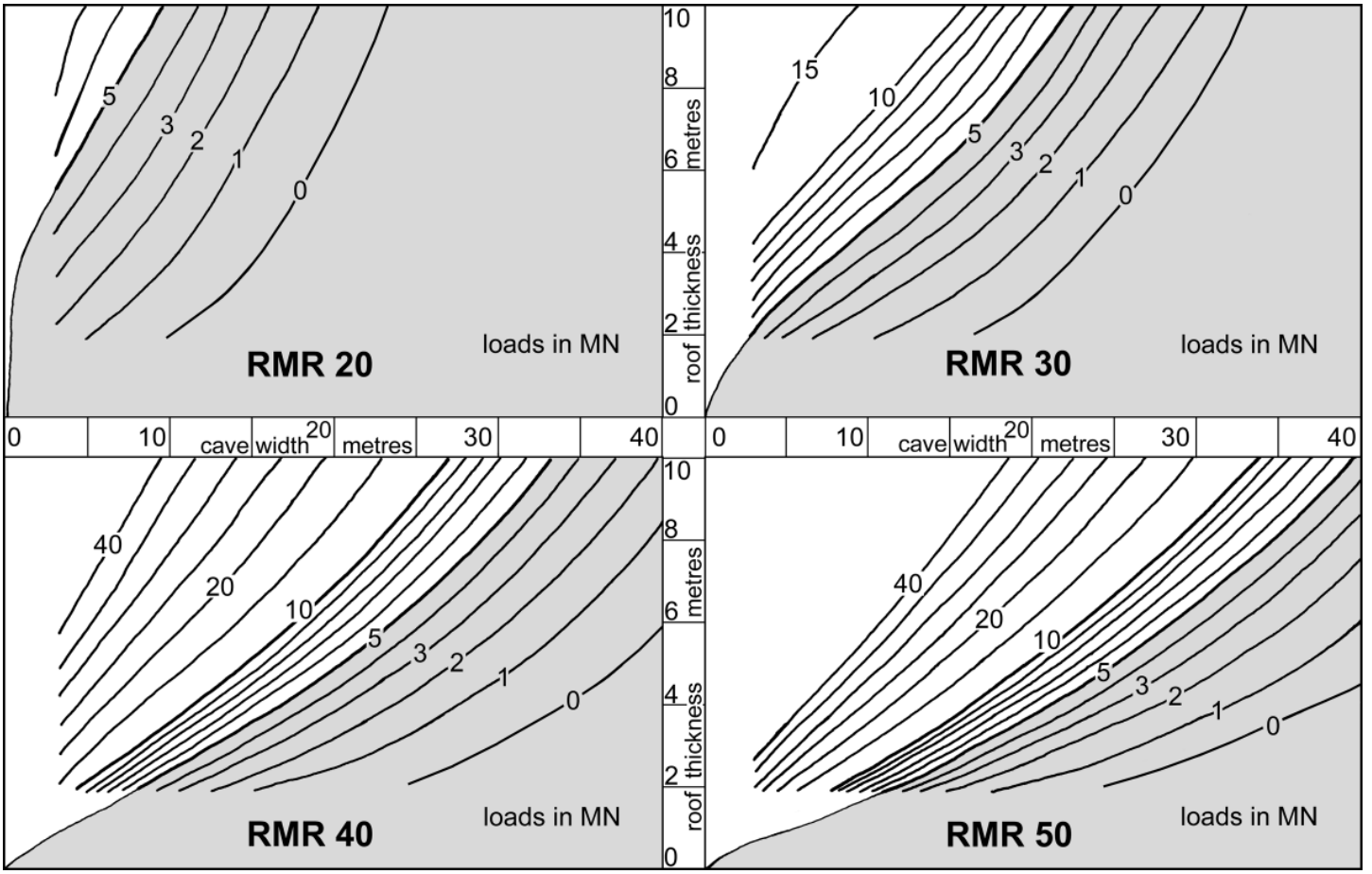


Figure 38.9 Nomograms that relate failure loads to cave width and roof thickness in ground of various rock mass ratings; the pink or grey shaded areas represent situations that should be regarded as unsafe for footings with imposed loads of 1 MN that lie directly over the caves
Modified from Waltham and Lu (2007)

strong rock, and a zone of weathered chalk further softens the rockhead, so foundations are simply taken to whatever depth is required to find adequate bearing capacity. Pinnacle loading on chalk is inappropriate, and details of the rockhead profile are less significant, except that buried sinkholes and filled solution pipes within chalk are frequently the sites of surface subsidence whenever soil movement is re-activated due to modified drainage or leaking pipelines (Edmonds, 2008).

38.5.1 The hazard of unseen caves

Where structural loads are carried down into bedrock limestone, the remaining concern is the presence of an unseen cave directly beneath. In nearly all karst terrains, there is only an extremely small statistical chance of a building being threatened by a cave that is directly beneath, that is large enough and that lies beneath a critically thin rock cover. Many structures have been inadvertently placed over large caves in tropical terrains, but collapse has not been instigated by the imposed loads, which are commonly very modest compared to the self-loads of the rock. The main geohazard from open caves in karst is to end-bearing piles with high point loads, and particularly to any heavily loaded bored piles or caissons. Guidelines exist for the thickness of roof required in limestones of various qualities in order to safely span caves of various sizes when loads are applied to them (Waltham and Lu, 2007); a cave generally has to be significantly wider than its cover thickness to create a threat (**Figure 38.9**). This means that, except in tropical karsts where large caves are typical, only a few metres of sound rock cover are generally required to provide integrity and render any deeper cave irrelevant to structural loading. Cases of rock failure in karst, as opposed to soil failure over karst, are extremely rare, and have only occurred where ground investigations have been grossly inadequate (Waltham, 2008).

Dam foundation and reservoir impoundment attract a whole series of problems when carried out on, or partially on, karstic limestone. Most limestones are strong enough to bear the loads imposed, but the problems of leakage can be massive and very complex, and are beyond the scope of this account. Both foundation problems and hydrological situations are comprehensively reviewed by Milanovic (2004), based on extensive

experience in the Dinaric karst. The Kalecik Dam in Turkey provides an accessible case history of karst leakage and its remediation (Turkmen, 2003).

38.6 Ground investigation and assessment of karst

Because karst is so extremely variable, each ground investigation is almost unique and has to be assessed in the light of local conditions and the available data. An overview of the local karst is essential for broadly evaluating the key parameters of sinkhole frequency, cave size and rockhead relief that may be anticipated; interpretation of these factors usually benefits from wider experience in karst terrains. Beyond that, the design of suitable structures and foundations can only be based on a sensible assessment of the immediate ground conditions in light of the recognised hazards and perceived risks. Numerical modelling of karst ground is likely to be unrealistic, because too many features and factors will always remain unseen or unknown. The morphological complexity of karst fissures and cavities means that any ground assessment based on borehole logs will inevitably be oversimplified; this applies to both unseen caves and rockhead profiles (**Figure 38.10**). Following even the most extensive borehole investigation, unforeseen cavities will almost certainly be revealed by any extensive ground excavation in karst; these will best be remediated on the spot by filling, sealing or spanning by means that can only be assessed after they are revealed. There is no simple answer to how many boreholes should be drilled to assess a karst terrain; the number required is as many as it takes to give the engineer confidence that the likely ground conditions are known or understood to a level that is adequate and appropriate for the risks involved in his particular project. Too few boreholes can create an unacceptable risk, while an excess of boreholes can be a frustrating necessity (Waltham *et al.*, 1986). Extra care may be needed where water flush can induce suffosional soil loss over limestone, or can cause rapid dissolution of gypsum and salt; drill rigs have been known to fall into self-induced sinkholes.

Though boreholes and probes do provide valuable insights into ground conditions, almost every investigative drilling into karst will intersect some extent of open voids or soil fills, most of which will have little or no influence on the overlying ground integrity. The length of a void, or the soil fill, down a single borehole is rarely critical; if it is down a narrow fissure, it is almost irrelevant. The critical factor is the void width in relation to the rock cover, and this can only be assessed from a number of closely spaced probes, or perhaps from a down-hole camera. The depths to which investigation boreholes should be taken into bedrock is indicated by the safe cover thickness that is required (**Table 38.1**). It is normally necessary to prove every pile site or loaded rockhead pinnacle with one or more probes, treating each one as an individual ground investigation.

The overall extent of bedrock voids encountered by boreholes may be used to indicate the scale of potential soil loss by suffosion from a soil cover, though such losses should be regarded as possible in almost any karst. The dominant factor in soil loss and sinkhole development is always the drainage, and this must be properly controlled on all construction

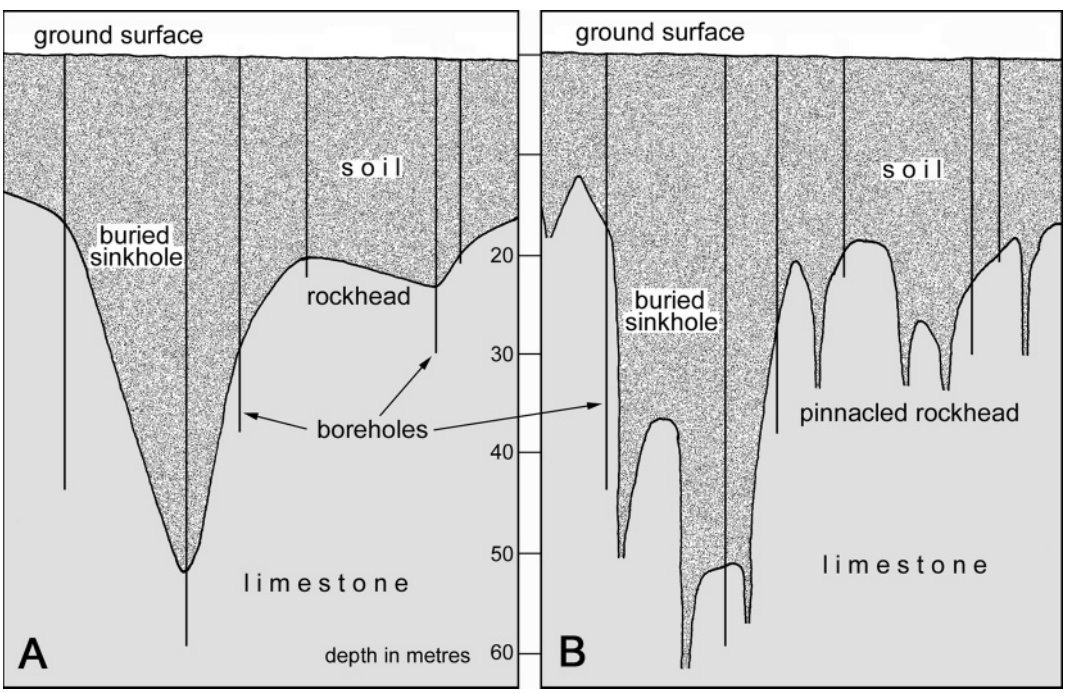


Figure 38.10 Two interpretations of the rockhead profile investigated beneath the site of a new tower block in Kuala Lumpur, Malaysia; (a) is the basic interpretation from the borehole data; (b) is a more likely interpretation based on an understanding of the pinnacled rockheads that are well known in the area

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Rock	Imposed load (kPa)	Karst class	Cave width, likely maximum (m)	Safe roof thickness (m)
Strong karstic limestone	2000	kI – kIII	5	3
		kIV	5–10	5
		kV	> 10	7
Weak limestone and chalk	750		5	5
Gypsum	500		5	5

Table 38.1 Safe roof thicknesses for various cave situations, and, therefore, a guideline to the depths to be proven by probing prior to construction
Data taken from Waltham *et al.* (2005)

projects; the borehole data may only offer a broad indication of how much infiltration of rainfall directly into soil areas may be tolerated close to or within a developed site.

A construction project over a large area of karst ground may reap benefits from appropriate geophysical surveys that can focus attention on the areas of most fissured or cavernous ground. The various methods all require expert interpretation, and even then can only identify ground anomalies that must be validated individually by boreholes or excavation; the proving may be more difficult where the anomalies are offset from the causative features. Only microgravity surveys directly indicate ground voids as missing mass, but these may fail to distinguish between networks of narrow fissures and more hazardous large voids. Seismic or electrical tomography in 2D or 3D can provide more useful data where boreholes are available (Waltham *et al.*, 2005).

Karst ground conditions are probably the most variable that a civil engineer may encounter, and may, therefore, be the most difficult where responses to their extreme and unpredictable variability have to be within a reasonable budget. Major errors, unwarranted expenses and possible catastrophes occur where the rather special nature of karst ground conditions are not recognised at an early stage within a project. Those with no experience or understanding of karst can make the most elementary of errors. All too often, karst is only considered on a site after the first new sinkhole has appeared amid the construction works. Yet if karst is recognised early, if it is given due consideration, and if all site drainage is properly controlled, its geohazard may be marginalised to a point where risks are reduced to acceptable levels. Because most karst ground failures are induced by engineering activities, the geohazard should be largely eliminated by good practice.

38.7 Geohazards on gypsum terrains

Gypsum karsts are very similar to those on limestone, except that there are no climatic constraints on the extent and distribution of gypsum dissolution; they also lack the extreme landforms of vertical cliffs and tall towers, because the rock is mechanically weaker. The most widespread geohazard on gypsum is the development of new sinkholes within the soil cover, and in this respect the conditions, the processes, the extents and their assessment are very comparable to those in a limestone karst (Johnson and Neal, 2003).

Gypsum differs from limestone in that it is much more rapidly dissolved in natural water, so that rock removal by dissolution can become a factor within the lifetime of an engineered structure (though dissolutional loss is still hugely slower than soil movement with reference to the hazard of new sinkholes). As in the case of limestone, proper drainage control is the key to surface stability in gypsum karst. However, the simple disposal of large flows of water into clean bedrock fissures may not be appropriate in gypsum, where a raised level of dissolution activity may rapidly create new cavities and modify the existing drainage with negative side effects. Similarly, remedial sealing or blocking of any open voids or known sinkholes within gypsum may only deflect future processes, problems and subsidence to adjacent ground. The rapidity of gypsum dissolution makes it especially susceptible to environmental change due to engineering works, and this is a widespread hazard in the sabkha terrains of the Middle East (see section 38.8). Reservoirs impounded over gypsum have a significant failure rate because leakage is generated not only by the washing of sediment fills out of the bedrock fissures, as in any karst, but is then exacerbated due to dissolutional erosion by the large induced flows. Abstraction for water supply can also cause accelerated dissolution, and ground subsidence, by drawing in flows of unsaturated water, though on a much smaller scale than in salt terrains.

Caves and open bedrock voids are generally not as large in gypsum as they are in limestone, because gypsum is a weaker material within which caves collapse at an earlier stage. On the other hand, the weakness of bedded gypsum and its changes to and from anhydrite allow more rapid upward cavity migration by progressive roof failure (**Figure 38.11**), so small rock collapse features are more common. Collapse sinkholes more than 100 m across, known in some gypsum karsts, have developed by multiple failures that have extended the collapsed ground laterally, and not in single large collapse events. Ground failures can also be induced by engineering loads imposed on the gypsum bedrock. Buried cavities are generally small but they require a proportionately thicker cover to ensure the integrity of overlying structures, for even the most modest loadings, which is appropriate for a rock weaker than most limestones; this determines the depth of probes required to prove stable ground in gypsum karst (**Table 38.1**).

38.8 Geohazards in salt terrains

Salt (also known as rock salt or halite) is an evaporite material widespread in sedimentary sequences that have accumulated in arid environments. Because it is so rapidly soluble in rainwater, its occurrence and geohazard are restricted to three

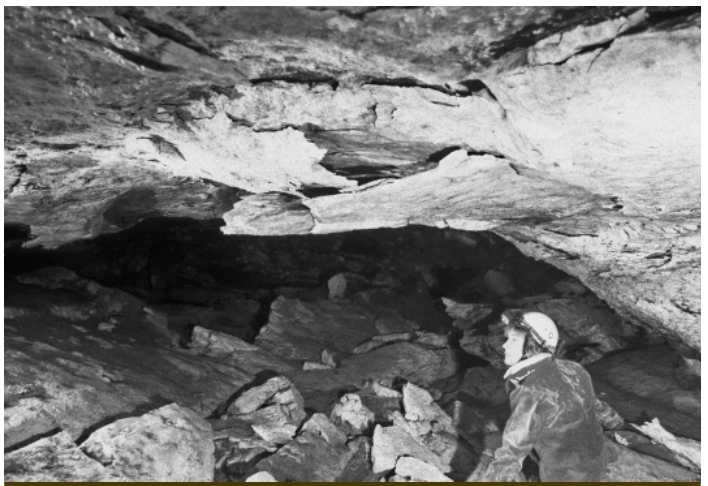


Figure 38.11 Cavity migration by progressive roof failure over a cave passage in massive gypsum at Pinega, Russia; flakes that are peeling away from the roof deform and sag before they break off



Figure 38.12 Collapsing ground on a salt dome in southern Iran; the person is standing at the edge of an active sinkhole in about 4 m of residual soil that is being undermined by rapid dissolution of the heavily eroded and cavernous salt beneath

environments: modern desert salars, salt domes in semi-arid terrains and buried rock sequences elsewhere.

Salars, salt pans, playas and continental sabkhas are areas of newly deposited evaporite salt, commonly with other minerals in thicker sequences. Groundwater is normally saturated, thereby restricting dissolution. Karstic cavities some metres across, which lie at shallow depths and may be revealed by smaller collapse openings at the surface, are generally restricted to marginal areas where the salt has been reached by aggressive surface run-off or groundwater input from adjacent hills. As active salt areas are largely prone to seasonal inundation, construction on them is very limited, but roads and driving routes that cross them can be threatened by the concealed cavities close to their margins.

Salt diapirs, or salt domes, rise from stratiform salt at depths of some kilometres. The salt is generally lost to rainfall at an outcrop, so that only caps of less soluble gypsum, anhydrite and clay survive at an outcrop. Actively rising salt diapirs form significant salt mountains, with or without salt glaciers flowing from them, but there are few beyond the many in the semi-arid Zagros Mountains of southern Iran (Talbot and Aftabi, 2004). These constitute mobile and cavernous terrains that are unsuitable for any development (Bosak *et al.*, 1999). Their thick mantles of residual soil are pitted by closely packed subsidence sinkholes that are actively collapsing and ravelling into dissolution cavities within the underlying salt (**Figure 38.12**). Cave chambers are up to 15 m across, with block failure and roof migration continuing at rates that are orders of magnitude faster than in limestone caves, so that bedrock collapses impact the ground surface as frequent events. Only when a diapir's rise virtually ceases, does surface lowering dominate, and then the residual mantle can become thick enough to prevent most rainfall reaching the salt, and its surface, therefore, will approach stability. By that stage, the salt sub-crop lies beneath a lowland terrain with chaotic sinkhole topography, but it is stable enough to be crossed by highways with minimal subsidence problems.

38.8.1 Subsidence over buried salt

Within environments of significant rainfall, salt does not appear at outcrops, but only survives beneath a cap of residual soil. In England's Cheshire Plain, a glacial drift 10–50 m thick overlies dissolution breccia that is around 50 m thick and consists of collapsed blocks of the mudstone that was originally interbedded with salt (Waltham, 1989). Underneath, the thick salt beds remain *in situ*, creating wide sub-crops due to their low angles of dip. Ground stability is ensured where any voids within the salt are filled with saturated brine, but subsidence takes place over any sites where the input of freshwater allows renewed dissolution of the salt. This is most extensive along linear subsidences (typically 1–10 m deep, 100–400 m wide and 1–5 km long) that develop over 'brine streams' of concentrated groundwater flow through the permeable dissolution breccia at the rockhead.

Subsidence is hugely accelerated when brine is pumped from these zones on an industrial scale, thereby drawing fresh water into the breccia. Even more destructive ground collapses and large sinkholes develop where brine is pumped from old flooded mine workings, so that support pillars are dissolved by the input of replacement water. These styles of brine-pumping have now been stopped in Cheshire, so the catastrophic collapses and rapid movements in the linear subsidences have virtually ceased, but comparable practices continue to cause surface damage in other parts of the world. Beds of salt with no outcrops can also create major surface subsidences where water and brine are able to flow through them. Some events are due to poorly managed brining operations, but the USA has many cases of large collapse sinkholes that developed after nearby wells, drilled for fresh water or petroleum through salt beds, were left uncased or poorly cased, thereby allowing brine outflow into aquifers that previously had no connection (Johnson and Neal, 2003).

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Flows from brine springs, in Cheshire and elsewhere, indicate that dissolution of buried salt can continue in an undisturbed natural environment. The result is ground subsidence, albeit on a modest scale, that can only be described as natural and uncontrollable. An appropriate engineering response may be the construction of houses on rafts, which prevent structural damage and are prone only to tilting; these may then be jacked back to horizontal, in the manner that was common on a large scale in past times of active brining in Cheshire, though current movements are generally too small to warrant such an operation. Natural subsidence, generally on a small scale but locally with sinkhole development, is widespread over salt beds in the USA (Johnson, 2005), and can extend to the rare formation of large collapse sinkholes as at the McCauley Sinks in remote country in Arizona (Neal and Johnson, 2003).

38.9 Karst geohazards on sabkha

Geohazards within the sabkha environment of arid coastal plains have grown in importance with the huge increase of construction activity on the lands fringing the Arabian Gulf. Coastal sabkhas are supratidal flats up to about 10 km wide, formed of evaporites and carbonates typically with a thin cover of aeolian sand (Warren, 2006). Continental sabkhas in inland basins are dominated by halite, with the attendant hazards due to dissolution (see section 38.8). Coastal sabkha lithologies are normally only a few metres thick, but can accumulate to significant thicknesses along the margins of subsiding basins; when subsequently uplifted, the carbonate and sulphate materials are subject to erosion and the consequent development of the more conventional karst geohazards.

The special concern for sabkha relates to the modern sediments that immediately underlie the coastal plains. These include relict and buried sabkha horizons within sequences dominated by aeolian sands that extend some kilometres inland from the active sabkhas. Sediments include carbonate sands, algal mats and dolomitised facies, or are dominated by clastic quartz with variable amounts of carbonate. Typically these are poorly consolidated to depths of 5–10 m; sandy materials have weak cements of anhydrite, gypsum and calcite, which are prone to loss by dissolution when the groundwater regime is changed. Denser, more competent and more lithified sediments persist at greater depths, where any halite in the coastal sabkhas may have been lost by dissolution during diagenesis.

The primary source of a sabkha's karst geohazard is due to gypsum that occurs at shallow depths within the poorly lithified sequence. This is deposited both within the capillary zone and beneath the shallow water table. Beds of almost pure gypsum are generally no more than a metre thick; they contain nodules of anhydrite and have an open texture with small cavities (vugs) and high primary permeability. Halite is locally present, but is not in massive beds; carbonates are present, but only in some of the sand grains. Groundwater in a sabkha is dominated by brines leaking upward from buried aquifers, and these are normally saturated with respect to gypsum. Consequently, evaporation causes interstitial precipitation of gypsum, which converts easily to anhydrite with the characteristic chicken-wire texture, producing ground with locally reduced bearing capacity.

38.9.1 Sinkholes on sabkha

Natural ground subsidence on sabkha appears to be rare, as there are few known or documented examples, and natural ground cavities are almost unknown. This is because dissolution is almost impossible in the undisturbed environment of solute precipitation from saturated waters, and the minimal rainfall is too low to disturb this equilibrium. Karstic ground subsidence, including sinkhole development, is, however, widespread throughout the Gulf coastal regions, where there have been reports of multiple sinkholes developing on construction sites. Most new sinkholes are no more than 5 m across and 2 m deep, appearing in ground at the perimeter of a construction works but rarely impacting the built structures, so they have been rapidly filled and forgotten. Some much larger sinkholes have also occurred.

All recorded new sinkholes on sabkha appear to have developed as a consequence of engineering activities that include leaking pipelines, uncontrolled drainage disposal, site dewatering and drilling with water flush that is not sulphate saturated. Many of these appear to have developed from dissolutional cavities in the underlying, gypsum-rich, Neogene sequences and not in the thin sabkha cover itself. It remains uncertain as to how much cavitation is due to dissolution of sabkha gypsum horizons at multiple levels within the Quaternary sequences. But it does appear that all have been the consequences of rapid dissolution in gypsum horizons, mostly within a few metres of the surface, when stable groundwater was replaced by the input of unsaturated water subsequent on engineering works. Suffosion of the unconsolidated cover sands then produced the sinkholes (**Figure 38.13**).

38.9.2 Sabkha karst in the Gulf region

A second subsidence mechanism occurs where the new input of unsaturated water dissolves the sulphate or halite cement of the sabkha sands, allowing localised compaction and displacement of the loose material. Of the many undocumented 'collapse settlements' in Saudi Arabia, one event at a large, steel-framed desalination plant near Jubail involved a number of column bases, founded on pads 1 m below the surface level, each sinking by about 50 mm (Sabtan, 2005); this was the result of leakage from old, corroded and fractured pipelines, which had been observed for some time before the implications were appreciated. The structure was remediated with mini-piles, 15 m long, driven into stable ground. The collapse potential is high in some of these sabkha silts, but they do not show instantaneous compaction in the style of hydrocollapse, because time is required for the dissolution of the natural cement, so conventional oedometer testing interrupted by inundation is not indicative.

As the dissolution of sabkha soils and their subjacent gypsum sequences is largely or entirely at shallow depths,



Figure 38.13 A small new sinkhole, already partly backfilled, which developed within a construction site on the coastal sabkha of the Arabian Gulf, after engineering works appeared to have induced dissolution of the gypsum that lies within either the bedrock sequence or the Quaternary cover
Photo courtesy of Laurance Donnelly, Halcrow

geohazards may be avoided by the use of deep pile foundations. This is normal practice for large built structures in the Gulf region, which have, therefore, not been affected by the sinkholes that their construction has undoubtedly triggered in adjacent ground. But the sinkhole hazard persists for roads and infrastructure that lack deep foundations, and where there is any failure to control all water movement. While it appears that direct rainfall can infiltrate the ground without disturbing the chemical equilibrium, any points of localised run-off from large built structures and areas of hard standing may displace saturated groundwater and permit new dissolutional activity after rare rainfall events. Soakaways and infiltration basins, which may be appropriate on some types of permeable ground, cannot be used for rainwater disposal on sabkha and the associated coastal plains. Where cavities are detected or become apparent, remedial action may include induced compaction, sinkhole sealing and spread footings.

It may be significant that sinkholes have developed, and ground cavities have then been revealed, on sites where prior investigation drilling had found no voids or karstic features. Small ground cavities, followed by larger surface sinkholes, may have developed only after disturbance and hydrological change were induced by the construction activity. Such a timescale is likely where halite is present, and it is possible in sulphate lithologies, though it is impossible in carbonate rocks. It is also possible that borehole disturbance exacerbated the geohazard, either with chemically aggressive water flush or by linking aquifers and changing the groundwater flow within the sabkha horizons. Sealing an area of ground with grout may merely deflect dissolutional activity into adjacent ground with soluble materials, unless the causative input of water is at the same time controlled and disposed of properly. Remediation of subsided ground by grout injection may also run certain risks when water-based grouts are used in such rapidly soluble ground.

Karst processes in sabkha soils are still not completely understood, and are subordinate to flooding, settlement and sulphate attack in terms of the geohazard. But it does appear possible that all sinkhole events on sabkha and its underlying gypsum are induced by engineering activity, so the established karst mantra of 'control the drainage' is even more critical than usual.

38.10 Acknowledgements

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It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 40 *The ground as a hazard*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Section 4: Site investigation

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Chapter 39

Introduction to Section 4

Anthony Bracegirdle Geotechnical Consulting Group, London, UK

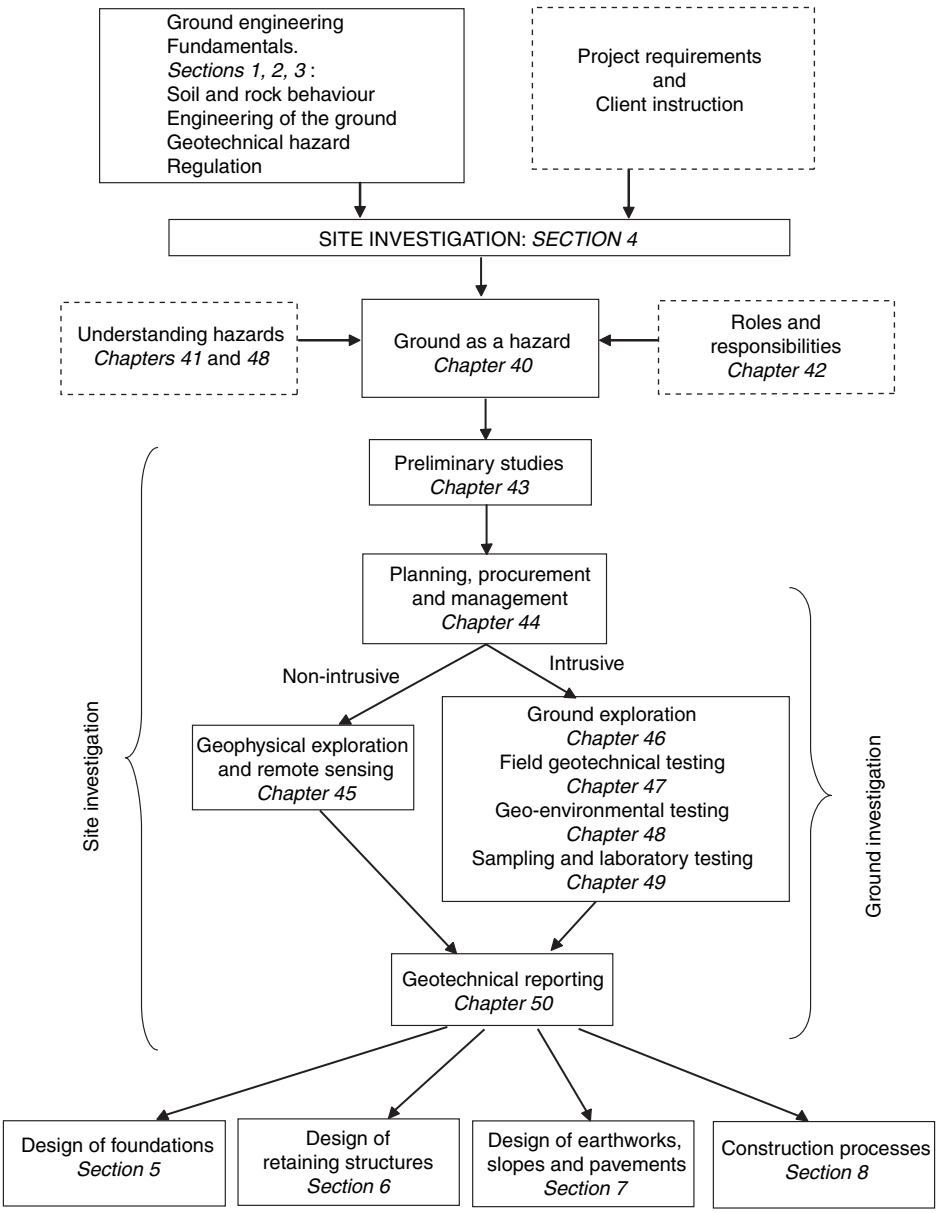


Figure 39.1 The structure of Section 4 and its relationship to other Sections

Site investigation provides a bridge to link the requirements of a specific project with its successful design and execution. Knowledge of the fundamental principles of material behaviour, ground hazards and construction risk are necessary at the outset, and these are discussed in the preceding sections.

In preparing this section, the authors have focussed on providing general guidance on principles, without overly delving into detail. For example, general classifications systems for soils and rock are provided in standard texts such as BS EN ISO 14688 (British Standards Institution, 2002, 2004) and 14689 (British Standards Institution, 2003) and reference to other detailed texts may be required for material-specific classification systems. The emphasis here is on the importance of thought and knowledge to site investigation practice and the section begins with an appeal for its application.

An understanding of land processes, topography, geology and the influence of human activity on the environment are essential. As discussed in Chapters 40 *The ground as a hazard*, 41 *Man-made hazards and obstructions* and 48 *Geo-environmental testing*, practitioners should have a rudimentary knowledge of potential ground hazards, including natural hazards, man-made hazards and contamination. The legal framework and the roles and responsibilities of all parties should be clearly defined and understood and this subject is outlined in Chapter 42 *Roles and responsibilities*.

As shown in **Figure 39.1**, the term 'site investigation' describes a process that encompasses a desk study and geotechnical reporting. The term 'ground investigation' refers to the physical activity of carrying out intrusive or non-intrusive investigations: laboratory and field testing are a part of this process. Desk studies are described in Chapter 43 *Preliminary studies* and planning and procurement in Chapter 44 *Planning, procurement and management*.

Chapters 45 *Geophysical exploration and remote sensing and geophysical exploration* to 49 *Sampling and laboratory testing* deal with the mechanics of ground investigation: geophysics, intrusive exploration, geotechnical and environmental field testing, sampling and laboratory testing. The product of the site investigation process is, of course, the geotechnical report, which, as discussed in Chapter 50 *Geotechnical reporting*, must be clear and unambiguous. The report should be as easily understood and as informative years after writing as it should be immediately after writing. To this end, it is important that any standards or classification systems used are clearly defined; not only should there be a description of what was done, there should also be an explanation of what could not be achieved and why, together with highlighted areas of uncertainty or potential error.

All too often, it is the failure of this important process that has led to compromised safety and economic loss. It hoped that the discussion within this section will be of interest to readers and help practitioners cross the bridge between aspiration and the successful execution of projects.

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Chapter 40

The ground as a hazard

An appeal for an intelligent view of the ground

Jackie A. Skipper Geotechnical Consulting Group, London, UK

How humans interact with hazardous ground conditions governs the resultant ground risk. Potentially hazardous ground conditions frequently attract engineering development because of important aesthetic or economic considerations. Many are successfully undertaken, but success requires sufficient investment, sound ground knowledge and an appropriate design. Yet in many projects, the ground is the least well-understood material, and this can lead to expensive problems being discovered during the construction phase.

This chapter encourages the reader to become actively knowledgeable about the ground of their project by first discussing the main types of geological hazard which may be encountered on a UK engineering site or project. Secondly, sources of ground information (many of which are now freely available online) are discussed, along with a system by which these can be interrogated to increase knowledge of ground conditions and design an effective site investigation. Finally, the use and communication of an intelligent understanding of these conditions has been shown to be extremely important in the success of engineering projects, ensuring that the engineer avoids becoming a victim of potentially hazardous ground.

40.1 Introduction

Humans often see the ground as hazardous. However, the Earth is simply influenced by gravity, water, heat or lack of it, plants and animals, and the sun. Humans often make the ground hazardous (or rather, it is how we behave that increases risk) by where we have chosen to live and work. For example, crumbly cliffs may be hazardous, but they do not represent a risk until humans decide they like the view from the steep crumbly cliffs overlooking the sea, and make their home there. The success of humans as a species has propelled us to develop, live and work in areas which were previously uninhabited (e.g. steep mountains, bogs), often for good economic or social reasons. We have remarkably short memories of risks related to hazardous ground events and we frequently emotionally weight an advantage (e.g. a lovely view) over a disadvantage (unstable ground). Fortunately, because of modern advances in civil engineering, there are few environments that we cannot engineer if enough money, a good understanding of the ground and a suitable design are invoked. Ultimately, ground hazards only become risks if they are unforeseen and unmitigated (for detail on ground profiles see Chapter 13 *The ground profile and its genesis*). This introduction is therefore an encouragement to understand the ground better before attempting to engineer it.

Geotechnical engineering is one of the most challenging of engineering specialities because the ground is far more diverse and variable than any man-made material. Considering how important the ground is in the success of most construction projects (amazingly, it frequently keeps buildings upright, sometimes for many hundreds of years), it is one of the poorest understood of the materials we utilise on a regular basis. On the majority of projects, for example, we have a far better sampling and testing regime and an infinitely better understanding of the behaviour of concrete and grout mixes on site than we have of the ground – our ultimate material. The exception is, of course, when a contractor wants to make a claim for unexpected ground conditions.

On some projects it is clear that people end up as victims of the ground rather than understanding and mastering it. 'Understanding the ground' is, however, not the same thing as doing a 'preliminary investigation' (see Chapter 43 *Preliminary studies*) and a 'design investigation' (see Chapter 44 *Planning, procurement and management*). Understanding is the process by which ground information is made into a ground model, augmented if necessary, communicated to all the relevant parties in the project, and if possible, added to throughout the project.

Why bother understanding the ground better? Why not, as an engineer once asked, just drill a couple of boreholes, test what comes up and base your design on that? My response was to use an analogy: if the ground is represented by a wedding cake at the bottom of a black plastic bag, and the borehole is your hand reaching into the black plastic bag, completely blind, to grab a piece, you may end up basing your design on a rosebud. Boreholes represent a miniscule fraction of the ground and, moreover, the ground frequently does not consist of straight lines between two boreholes. Hence the ground in between the boreholes will not necessarily be the same as that recovered in the boreholes.

Many companies try to save money by having a 'cheap' site investigation, and this is a process in which data may be lost. As Chapman amply discusses in Chapter 7 *Geotechnical risks and their context for the whole project*, saving money with a 'cheap' site investigation is practically the worst investment

in the construction industry over the long term – hence the inverted commas. However, despite improvements in standards of site investigation over the past few years (see Chapters 42 *Roles and responsibilities* to 48 *Geo-environmental testing*), there will always be economic downturns. 'Cheap' site investigations will always be sought and may not be avoidable, but can miss important changes in the ground. This is not only because fewer boreholes, maybe of a cheaper type, will give you less data, but because low-paid staff are frequently poorly trained in what they are logging – taking samples for testing, and the testing itself. Even with a full EC7-approved site investigation it is possible for important information about the ground to slip through the cracks between the specification and the interpretative report. So it is important to make the most of every piece of available information.

In geotechnical engineering we can compensate for less adequate site investigations by adopting a conservative design – by taking no risks, or alternatively by taking the basic minimal information available and taking the risk that the ground will be fine. However, those extremes have been shown repeatedly (Chapman and Marcetteau, 2004) not to be the smartest and most cost-effective solutions. Understanding the ground to the best of our ability, and designing with the ground in mind, makes the best economic sense (see also Chapter 2 *Foundations and other geotechnical elements in context – their role*).

40.2 Ground hazards in the UK

Most of the common types of hazardous ground in the UK are dealt with in this volume and the relevant sections are linked. These ground hazards can generally be grouped into three categories: geological hazards, geomorphological and topographical hazards, and anthropomorphic hazards, and are discussed below.

40.2.1 Geological hazards

This category includes the results of events that occur naturally on Earth which we consider unreasonable. Examples include dramatic geohazards such as volcanic activity – resulting in debris or mudflows, and earthquakes – resulting in the possibliliy of tsunamis. In the UK we are generally considered low risk for these hazards since we are a long way from tectonically active centres. However, we do have occasional earthquakes in certain areas and have (in the past) had tsunamis due to earthquakes or underwater landslide failures. Seismic hazards do therefore need to be considered for long-life projects, sensitive projects (such as large tunnels, nuclear power stations) and coastal projects. Volcanoes are even less common in the UK – the last ones to erupt on the UK mainland were probably about 55–60 million years ago. However, bear in mind that *many* of our soils and rocks contain layers of volcanic ash laid down or reworked within them, and these ash deposits frequently contain swelling clays which may precipitate or exacerbate landslips, slow down tunnelling operations, and impede soil handling and re-use (see section 40.2.2). If in doubt, the clay minerals need to be analysed.

The British Geological Survey (BGS) has very useful information, historical data and risk maps for these tectonism geohazards on their website.

The second part of this category includes ground types which are harder, weaker, softer, looser, have more and bigger holes or caves in them, are more variable, less stable, more aggressive, or contain water or gas at higher pressures than would be desirable. These types of ground are discussed in great detail in Section 3: Problematic soils and their issues, while folded or faulted rocks are dealt with in Chapter 18 *Rock behaviour*.

In an ideal engineering world, all geological deposits would be laterally extensive, of uniform and predictable thickness, and homogeneous in nature. Unfortunately the ground is usually variable. While it is possible to be exceptionally lucky and encounter tens of meters of thickness of well-behaved, unweathered, unfaulted, horizontally-bedded strata across an entire project site, it is actually not the norm. The reason for this is the variability of processes which contribute to the deposition of soils and rocks, and which alter them afterwards (see Chapter 13 *The ground profile and its genesis* for more detail). Generally the most uniform, homogenous and laterally extensive rocks are marine sediments, which can be deposited as essentially the same type of soil or rock over hundreds of thousands of kilometers; but do not be lulled into a false sense of security. Even amazingly thick, fully marine deposits sediments like Chalk, Carboniferous Limestone and London Clay vary vertically in strength, texture and permeability due to changes in water depth while they were being deposited. The nearer to the shore we get with environments of deposition, the more variable the sediments get (think Carboniferous Coal Measures, which were deposited in swamp to delta environments, or the Mercia Mudstone Group, which varied from desert to floodplain in origin). In addition, with worldwide fluctuations in sea level and the extremes of climate variation over geological time, we can end up with a stack of potentially very variable sediments indeed. Once we superimpose on this the deposition or intrusion of igneous rocks, or start to look at the processes of folding, faulting and metamorphosis, this variability increases further. However, it is important to remember that variability is still only a potential risk if it is not anticipated.

40.2.2 Geomorphological and topographical hazards

This category includes all the hazards which are primarily concerned with how gravity and erosion affect the ground, and how water interacts with the ground near or above its surface. Landslips and landslides, mudslides, coastal erosion and flood risk all come into this category. Tectonism (the action of the Earth's tectonic plates moving in relation to each other, resulting in, amongst other things, mountain-building events) has a lot to answer for. Topographically higher ground is (in geological terms) just waiting for gravity to act on it, to be weathered and fall down, to be talus until it falls further and gets washed into a river, and eventually it breaks down into sand and gets deposited in the sea as sediment. It is important to recognise once more that humans love to build in high places, near

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the sea and everywhere in between – but that hilly areas can fall down, and that coastal and low-lying areas often flood.

40.2.3 Anthropomorphic hazards

In the last few thousand years, humans have had an enormous impact on the surface and sub-surface of the Earth. In the last 200 years we have changed the planet more than in all the previous several thousand years put together. Our skills as humans have, in this short time, enabled us to use fossil fuels to drive transport and industry (often leaving large holes in the ground where they were abstracted from) and our skills in civil engineering have enabled us to master many formerly forbidding environments and locations.

An awareness of how an area has changed over time may reveal a wide range of human activities, many of which will have implications for particular projects. For example, in the area of east London where I used to live there was, in chronological order: a Bronze Age trackway development (archaeology), a coal gas production plant (probable contamination), Joseph Bazalgette's Northern Sewage Outfall (Victorian tunnels and obstructions), and past and present industry associated with the London Docks (archaeology, pollution *and* obstructions). Superimposed on these layers and issues were the abundance of munitions dropped in the area during the Second World War 1939–1945 (possible unexploded ordnance), the relatively new Dockland Light Railway and its tunnels, and normal everyday services. There are many companies who will, for a reasonable fee, help steer a way through this potential multiplicity of man-made hazards, provide site-specific environmental risk information and historical mapping for a given site area.

40.3 Predicting what the ground may have in store

Using personal computers, we are fortunate to be able to access more information about the ground of a potential site faster than ever before. Programs such as Google Earth can give us aerial imagery which previously required long waits or searches, and possibly special viewing equipment. Not only can we look up our site in seconds, but in one click we can discover what volcanoes or earthquakes have occurred nearby (try ticking Gallery in the Layers folder and then go to Old Hawkinge in Kent if you want to find an earthquake in the UK), the approximate elevation, and distances between interesting points. Using Street View within Google we can get a close-up of topography and buildings in the area. Using the Historical Imagery facility (in the View dropdown menu) we can even see if the area has changed over the previous few years – useful for landslip or redevelopment projects.

Armed with Google Earth and a geological map of your site area, it is possible to assemble a great deal of quality information to assist your understanding of the ground in your project area within an hour or so, using the topography–water–anything odd (TWA) system:

Topography Running the cursor over the aerial view of the site and the surrounding area will give you an idea of variations in topography. The next question to ask is – does the topography make sense and, if not, why is it like this? If you look at the geological map, is there an obvious reason?

Water Where is the water in this area? Is it in rivers, streams, lakes, marshes, an estuary, the sea? Does it make sense where the water is, or is there poor drainage due to impermeable strata? Is the site in an obvious flood risk area (i.e. at or near sea level, or at the same level as the local rivers run)? Is there no water at all in this area? Why? Where could it have gone?

Anything odd? Is there anything that does not seem quite right, or any colours or shapes on the ground that do not make immediate sense in terms of natural or human activity? Are there any areas where buildings 'should be' but are not? Road names can be useful indicators of geological or former engineering hazards. Are there road or lane names near your site which include such words as: Watermeadow, Flood, Spring, Cave, Swallow or Swallowhole (possible flood or solution hazards), Brick, Kiln, Mine or Quarry (former mining or quarrying), Undercliff, Zigzag, Slip (possible ground instability problems)? Cave or Dene can also indicate natural or man-made cavities. In Google Earth, Street View can be another useful source of information, picking up uneven ground surfaces, major cracks in walls, and repeatedly re-made roads.

If, for any reason, you cannot use Google Earth, there are an increasing number of other remote viewers which are free to use such as Yell.com and Bing Maps, both of which allow 3D inspection of built sites in UK cities.

40.4 Geological maps

Geological maps can, to the average engineer, be slightly confusing. They use bizarre colours and odd symbols. However they do contain vast amounts of useful information if you have a little patience. If you want help, ask a geologist or put 'how to read geological maps' into a search engine. A note of caution: engineers often consider geological maps to be 'gospel'. They are not – they are produced by clever geologists who do a lot of field work, incorporate as many good quality boreholes as possible, and then extrapolate their findings. Without X-ray vision, their geological maps, while a very good best guess of what the ground consists of, can occasionally be wrong. Another important point to mention here is that just because a geological formation is called Kimmeridge Clay, or Lias Clay, or Gault Clay, etc., it does not mean that the formation is only clay. Likewise, formation names with 'sand' at the end (e.g. Lower Greensand, Arden Sandstone) rarely consist entirely of sand. Soils and rocks of any age or name usually contain naturally occurring harder layers, or weaker layers, or indeed sandier layers – be prepared and read the geological map carefully. Again, the BGS has very useful tools on their website such as the lexicon of named rock units (also useful for looking up soils – 'rock' means rock *or* soil in this instance). The BGS

onshore borehole historic database is now largely free, too, and in combination with the lexicon will probably tell you far more than you ever thought you needed to know about the strata in the area of your project, and this information will act as the basis for a refined ground model based on a well-designed site investigation.

40.5 Conclusions

Having a better understanding of the ground is a huge leap forward and will help enormously in planning and designing for a project, but what appears to make the greatest difference is communicating this understanding to others (Skipper, 2008). I first experienced this principle in 2003 when working on the Dublin Port Tunnel, where I found that communication of the ground model to *all* levels of staff (from site investigation staff to designers and foremen to site workers) made a huge difference to their collaboration, communication and feedback. This improved level of understanding allowed the use of the observational method in what was complex and challenging geology in a very sensitive location (Long *et al*., 2003). Since then I have been involved in the use of this principle in a wide range of projects from the small to the very large. I have seen it result in a wide range of improvements, from better specified site investigations to improved ground descriptions and interpretations, in turn leading to optimised design and better risk management. So to conclude, an intelligent understanding of the ground makes the best technical and economic sense, and communicating this understanding with, and to, others, maximises this value to engineering projects.

40.6 References

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Skipper, J. A. (2008). Project specific geological training – a new tool for geotechnical risk remediation? *The Proceedings of Euroengeo 2008*, International Association of Engineering Geology Conference, Madrid.

40.6.1 Further reading

Bryant, E. (2004). *Natural Hazards*. Cambridge, UK: Cambridge University Press, 328 pp.

Griffiths, J. S. (2002) (Comp). *Mapping in Engineering Geology*. London, UK: The Geological Society, 287 pp.

Mitchell, C. and Mitchell, P. (2007). *Landform and Terrain, the Physical Geography of Landscape*. Birmingham: Brailsford, 248 pp.

40.6.2 Useful websites

British Geological Survey, borehole record viewer; www.bgs.ac.uk/data/boreholescans/home.html

British Geological Survey, Earth hazards information and contacts; www.bgs.ac.uk/research/earth_hazards.html

British Geological Survey, lexicon of named rock units; www.bgs.ac.uk/lexicon/

Google Earth; www.google.com/earth/index.html

Ground viewers/remote sensed imagery; www.yell.com/map/ and www.bing.com/maps/

It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 8 *Health and safety in geotechnical engineering*
- Chapter 13 *The ground profile and its genesis*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 41

Man-made hazards and obstructions

John Davis Geotechnical Consulting Group, London, UK
Clive Edmonds Peter Brett Associates LLP, Reading, UK

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Man-made hazards in the ground take many forms. The principal forms discussed here are voids, contamination, unexploded weapons, obstructions and buried services. Many different geological materials have been mined in the UK over thousands of years. These include materials used for fuel (coal and oil shale), a wide variety of metal ores, non-metals and stone itself. Voids left by mining activity occur in a wide range of geological settings and both the mining methods and the nature of the voids themselves also are similarly varied. References are made to methods of void detection and treatment. Man-made contamination in the ground is common in previously developed sites, especially those with an industrial history. A legacy of war is the risk of encountering unexploded munitions in the ground. Processes for minimising the risks associated with contamination and unexploded weapons are discussed in this chapter. Man-made obstructions in the form of buried structures or parts of structures are commonly encountered in ground investigations and ground works. These can be particularly hazardous if they contain voids. Underground services are an almost ever-present hazard for ground investigations. Risk minimisation techniques are also discussed for these classes of hazard.

41.1 Introduction

This chapter briefly describes man-made hazards and obstructions that might delay or compromise the implementation of ground investigations. These man-made hazards and obstructions may well also affect subsequent design or construction, creating obstructions to penetration of foundations, hard areas or variability. This is discussed in only a limited way here.

In the context of ground investigations a hazard is taken here to be something man-made within the ground that has the capacity to cause harm in an occupational health and safety sense. Above-ground operational health and safety hazards are not discussed.

An obstruction is taken here to be something man-made within the ground that has the capacity to delay or prevent progress of a ground investigation. Some examples of pre-ground-investigation bureaucratic processes that can cause delay if not addressed are also given.

The possible occurrence of both the physical and bureaucratic issues raised here should be addressed in the desk study, and the scope and form of any subsequent ground investigation should be adjusted accordingly. See Chapter 43 *Preliminary studies* for more detail on the scope and content of desk studies.

41.2 Mining

Great Britain has a long and rich history of mineral extraction using mining techniques, dating back to Roman and even Neolithic times. Man's desire to obtain minerals of special value and use has driven the development of mining activity through time. As a result significant areas of land contain mined ground and a legacy of ground instability.

Historical mine workings present a significant subsidence hazard to existing and new-build construction as well as a risk of personal injury or death. From a health and safety perspective it is therefore necessary to identify where the mine workings are, the nature and extent of the void space and the degree of risk posed to the surface.

For coal mining areas the investigation and treatment of workings and mine entries needs to be notified and agreed in advance with the Coal Authority. Outside of coalfields it is usual to agree such matters with the local council, who may wish to involve the Mineral Valuer and the Health & Safety Executive. Awareness of the legal framework, such as the Mines and Quarries Act 1954, is important as well.

41.2.1 Mineral types

From a historical mining perspective it is convenient to think of mineral types in terms of coal and related minerals (e.g. fireclay and gannister, also iron ore and oil shale) extracted within the many coalfield areas of Great Britain (see **Figure 41.1**) and those minerals that were mined outside of coalfield areas.

Minerals that were extracted in non-coalfield areas include metal ores (e.g. iron, lead, zinc, copper, silver, gold, tin); non-metals (e.g. fluorspar, calcite, barytes); evaporites (e.g. gypsum and salt); and stone (e.g. slate, sandstone, limestone, chalk). For reference, the recorded spatial distribution of non-coal mining is also shown in **Figure 41.1**.

Currently mining is restricted to coal, salt, potash, gypsum and others (silica sand, limestone, barytes, fluorspar, slate and haematite). Further details can be obtained from publications such as the *United Kingdom Minerals Yearbook* (Bide *et al.*, 2008).

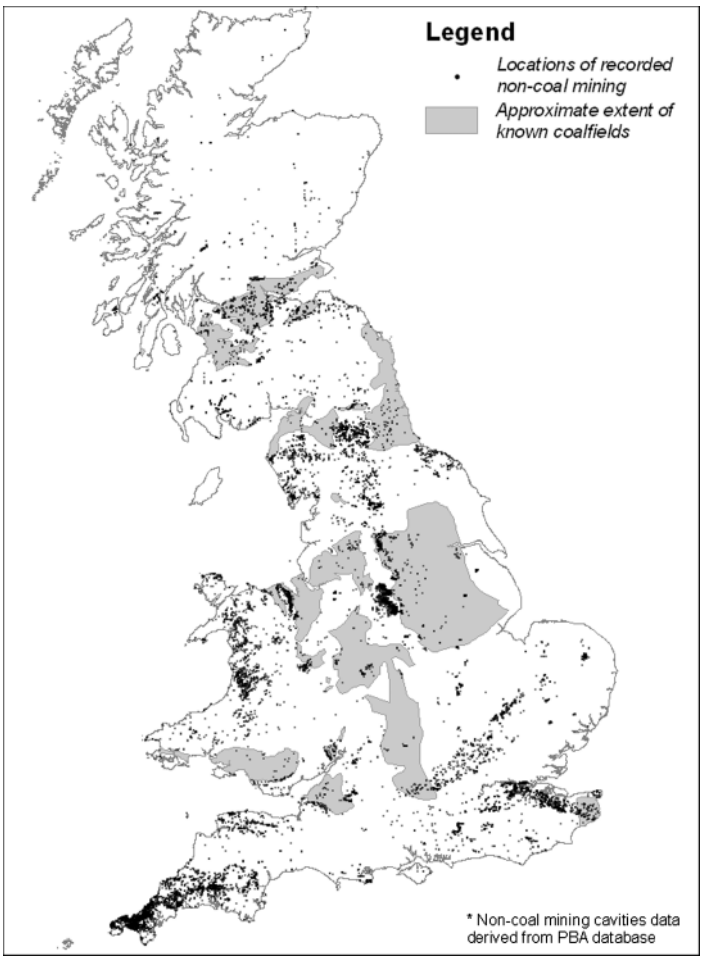


Figure 41.1 Spatial overview of mining in Great Britain

41.2.2 Methods of mining

Methods of mining have changed with time and technology, but also reflect differing geological settings.

A mineral at outcrop would be extracted by quarrying. However, as the overburden rock/soil thickness that needs to be removed from above the mineral layer increases, extraction by adits driven through the quarry face becomes more efficient for continuing the extraction underground without the need to remove the overburden (see **Figure 41.2**). For minerals that occur at shallow level in layers that do not dip steeply the next simplest style of mining is the 'bellpit'. A narrow, unsupported shaft was dug down to the mineral layer and the extraction continued outwards from the shaft base radially to form a 'bell shape' (see **Figure 41.2**). Other regional variants were also developed, for example in the chalk of southern and eastern England, such as deneholes, chalkwells and chalkangles (Edmonds *et al*., 1990).

Later, more advanced forms of mining referred to as 'pillar-and-stall workings' were developed. The workings, accessed via shafts and adits, comprised a series of tunnels (stalls) excavated outwards from the entry point, linked by cross-tunnels to create a grid of intervening pillars to support the mine roof (see **Figure 41.2**). For coal extraction the pattern of tunnels and pillars that were created, and the names referring to them, varied regionally (Littlejohn, 1979).

For many minerals (e.g. limestone, chalk and gypsum) this form of mining continued as the preferred method of working. However, for coal, the mining style was further developed into the 'longwall' method. This technique involved coal being completely extracted along a 'longwall' face, progressing forwards below large areas. Access to the working face and removal of the mined coal was achieved via connecting, perpendicular roadways. Propping was used to support the working face. In the areas between the roadways the roof was allowed to progressively fail and subside as the working face moved forward. The original mines were hand-dug, but in time the process became mechanised (see Healy and Head, 1984).

Other methods of mining were developed to suit the particular characteristics and geological setting of the mineral. Where a mineral exists in the form of an irregular ore body or steeply dipping vein it is common to follow the depositional pattern extending through the host rock. In limestones the ore or vein may be accessed via naturally occurring karstic cavities. Larger ore bodies or veins can be mined upwards (stoping). These forms of past mining are typical in areas like the Pennines, North and South Wales, Peak District, Lake District, Mendip Hills and Cornwall.

The soluble nature of salt has been used to extract it by brine pumping (solution mining), particularly in parts of Cheshire (Bell, 1975), Lancashire, Somerset, Staffordshire and Worcestershire.

41.2.3 Mining history and information sources

Many of the mining techniques described above date from the Middle Ages and earlier; however, mining activities notably intensified from the late 1600s, through the 1700s and 1800s, continuing into the 1900s as well. It is evident that mining activities may have started and ceased in an area before publication of the first-edition Ordnance Survey (OS) maps (typically dating from the 1870s/1880s). Consequently, reliance on the use of historical OS maps alone should be treated with caution when researching the history of a site in a mining area.

Information on coal and non-coal mining is available from a wide variety of sources, often not mainstream or easily accessible. Specialist knowledge may be required to fully interpret and understand the data. However, since the early 1990s a number of national and regional studies have been published (e.g. Arup Geotechnics, 1990), making information more accessible. The recorded mining data have been captured digitally and made available by the British Geological Survey and Peter Brett Associates LLP. For coalfield areas a large digital mining archive is held by the Coal Authority. Other smaller collections of data are held by various councils, and caving and mining-history organisations. Obtaining relevant mining information from these sources is essential for a desk study to check for the potential for current and past mining activities.

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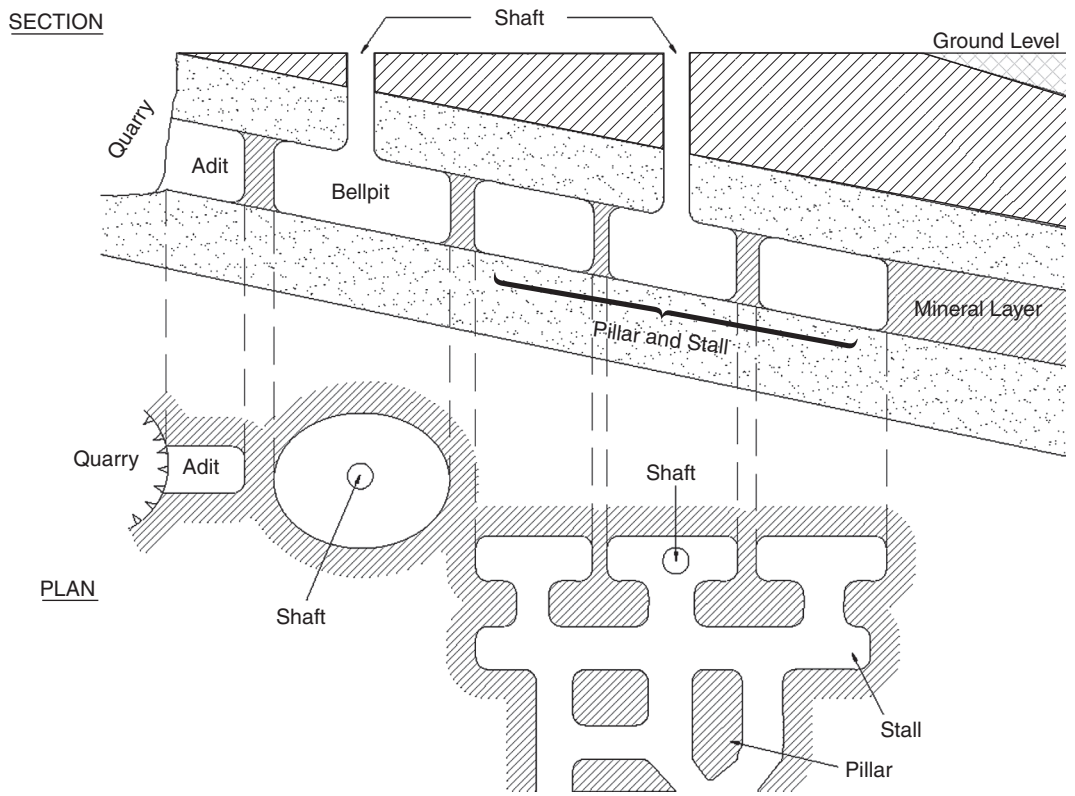


Figure 41.2 Illustration of mineral extraction techniques used as mineral layer deepens from outcrop



Figure 41.3 An example of the results of collapse of old shaft entries
Courtesy of Peter Brett Associates LLP

41.2.4 Ground instability hazard

Collapses over abandoned mines pose a ground instability hazard. Common subsidence triggers include leaking water-carrying services and soakaways, changes in water table level, heavy rainfall, disturbance by excavation and surface loading.

The most common form of instability relates to the collapse of old shaft entries, often left poorly sealed and backfilled. Large ground movements can result if a shaft has deteriorated, and failing ground can move both vertically down and laterally into open, connected void space at depth (see **Figure 41.3**). Subsidence may also occur over shallow workings and adit entrances, but again depends on the failure mechanism and the extent of remnant interconnected mine void space at depth.

Large-scale mining of a succession of coal seams can produce widespread subsidence troughs, problematic for properties and infrastructure at the surface (see Healy and Head, 1984). Other forms of longer-term ground movement over coal mining areas can be associated with fault lines and major discontinuities where stress relief may be concentrated (e.g. Donnelly *et al*., 2008).

A range of ground instability problems caused by abandoned mines associated with a variety of other mineral types is presented in the Technical Review Case Study Reports prepared by Arup Geotechnics (1990) as part of a national review study.

Discussion of how local authorities and developers should take account of ground instability relating to mining, in the context of planning and development, is set out in PPG 14: Development on Unstable Land (Department of the Environment, 1990), Annex 1 (Department of the Environment, 1996) and Annex 2 (Department of the Environment, Transport and the Regions, 2000).

41.2.5 Investigating mine workings

A wide variety of direct and indirect investigation techniques exist for detecting mine workings. For investigations of coal mine workings advance permission must be obtained from the Coal Authority.

Indirect techniques include the use of aerial photography and satellite imagery to look for shafts, adits or associated surface excavations, spoil heaps and infrastructure. The viewing of historical images as well as utilising different parts of the electromagnetic spectrum can be useful. There are many texts describing the types of analogue/digital data available together with methods of image manipulation and interpretation (e.g. Lillesand *et al*., 2008).

Other forms of indirect techniques comprise geophysical surveying. In a mining context the techniques need to be sensitive to the detection of open voids, partially to fully collapsed voids, water-filled voids, broken or collapsing rock/soil sequences, Made Ground backfills, brick or metal shaft linings, etc. Depth to the mine void and the surface environmental setting of the survey both play a part in the final choice of survey – if a survey is even feasible. There are many texts describing the theory and practice of geophysical surveying (e.g. Reynolds, 1997).

Again, there is a range of direct investigation techniques that can be used to detect mine workings. Considerations for the choice of method depend on a number of factors, such as, geology, depth of mineral, style of mining, whether intact or collapsed, access, and health and safety risks. The potential for hazardous gas or polluted water escapes also needs to be evaluated. In the case of soluble strata or collapse situations the choice of drilling flush is important. Inaccessible open voids may be viewed directly using down-hole closed circuit television (CCTV) cameras, which allow the style of mining and condition of the workings to be evaluated. In circumstances where there is a need to measure the size and orientation of open workings, down-hole laser survey techniques can prove useful (see **Figure 41.4**). Where access is possible and safe, direct visual inspection can then be undertaken (see **Figure 41.5**). The various investigation techniques are described in detail elsewhere in this manual.

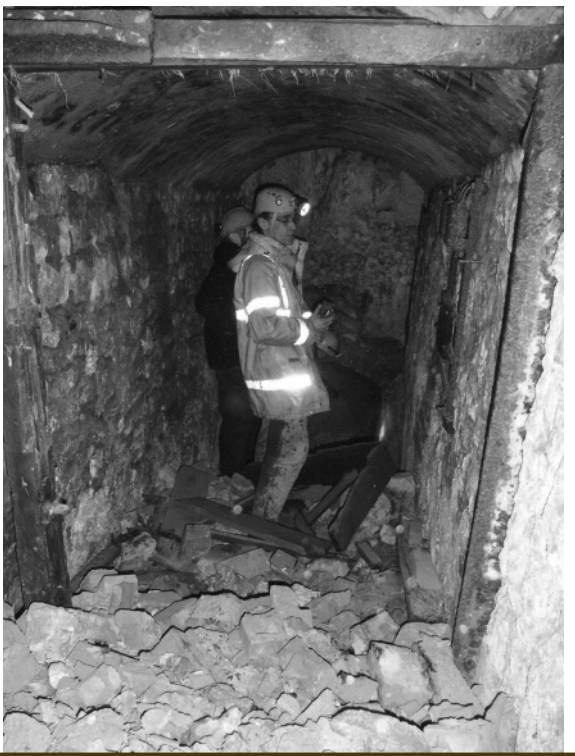


Figure 41.5 Direct visualisation of mine workings
Courtesy of Peter Brett Associates LLP

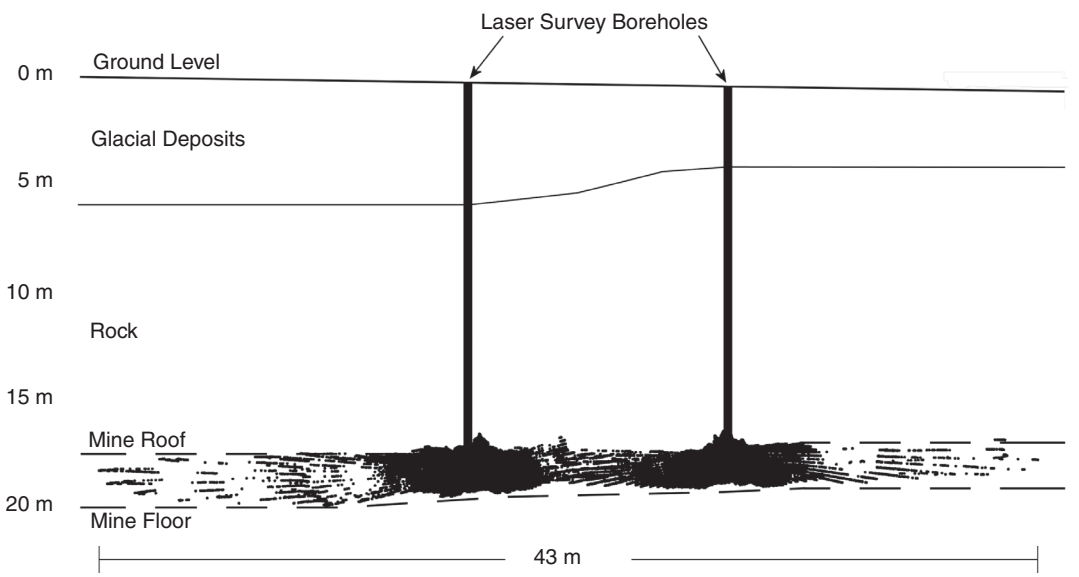


Figure 41.4 Results of down-hole laser survey techniques

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41.2.6 Mine working investigation case history

In order to illustrate how a combination of indirect and direct investigation techniques can be used effectively to detect historical mine workings, the following account briefly describes the investigation approach at Field Road, Coley, Reading (SU710728 area).

A major ground subsidence occurred on 4 January 2000, causing the collapse and loss of the fronts of two houses, their gardens, the pavement and adjacent road. The collapse measured about 9 m across and was several metres deep (see **Figure 41.6**).

After initial infilling of the collapse a surrounding ring of close-centred dynamic probes was undertaken to establish the likely cause of the collapse. The results revealed a series of open voids that were interpreted to be old chalk mine workings, having a mine roof level at about 7.5 m to 11 m below ground level (bgl) and a mine floor level at about 13 m to 14 m bgl. A typical probing profile is shown in **Figure 41.7**. The ring of probes was extended outwards in a grid pattern that revealed yet more mine workings forming an irregular pillar-and-stall style of working. A smaller number of light cable percussion boreholes were positioned strategically to provide sampled geological profiles to confirm the interpretations being made for the probing profiles. A good geological correlation was achieved (see **Figure 41.7**).

To check how far the mine workings might extend, a microgravity geophysical survey was conducted initially as a trial along the road, south of the collapse. The trial was successful in detecting the pattern of mine workings as a series of Bouguer anomalies, and so the survey was extended further along the road in a southerly direction and outwards into the adjacent garden areas. The survey interpretation was then checked by carrying out additional dynamic probes. **Figure 41.8** shows an example of the microgravity survey results compared with the actual mapped mine workings detected by dynamic probing. The geophysical survey results demonstrated that the microgravity technique provides a good indication of where mine workings are to be found, so useful in a reconnaissance sense, though locating the exact mine tunnel positions was less precise.

When faced with the task of mapping out the pattern and extent of larger-scale pillar-and-stall mine workings there is a tendency to use a grid pattern of exploratory holes. If the mine workings are in an advanced state of collapse, or have collapsed, then this approach may be unavoidable. However, where there are sections of open, intact mine workings it is possible to reduce the number of exploratory holes being bored.



Figure 41.6 A major ground subsidence, 4 January 2000
Courtesy of Peter Brett Associates LLP

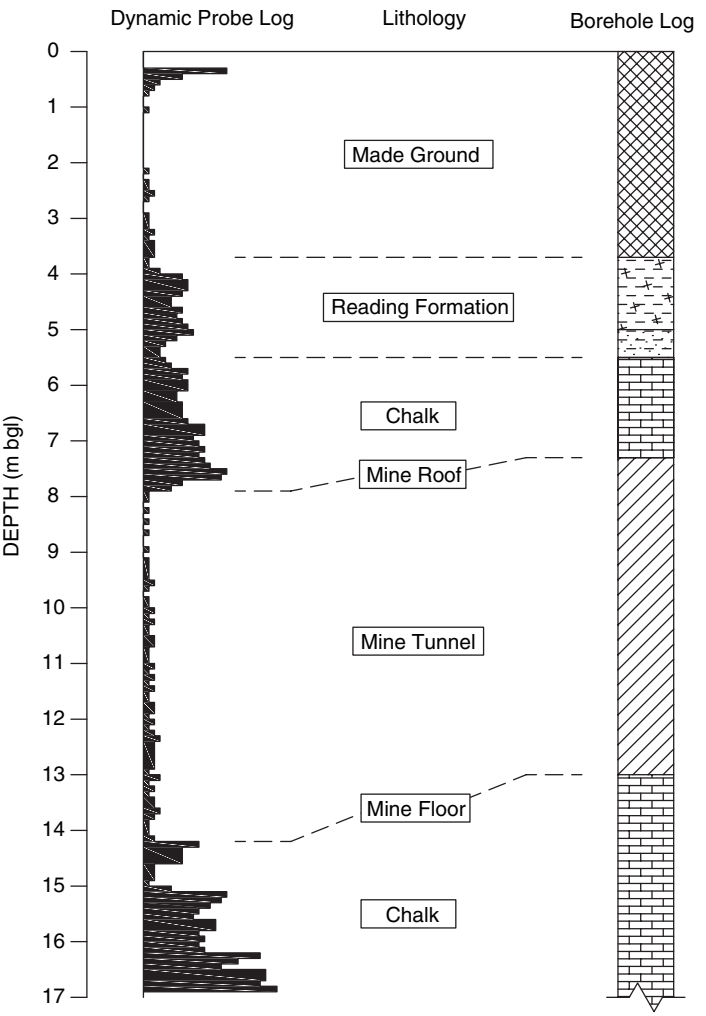


Figure 41.7 A typical probing profile

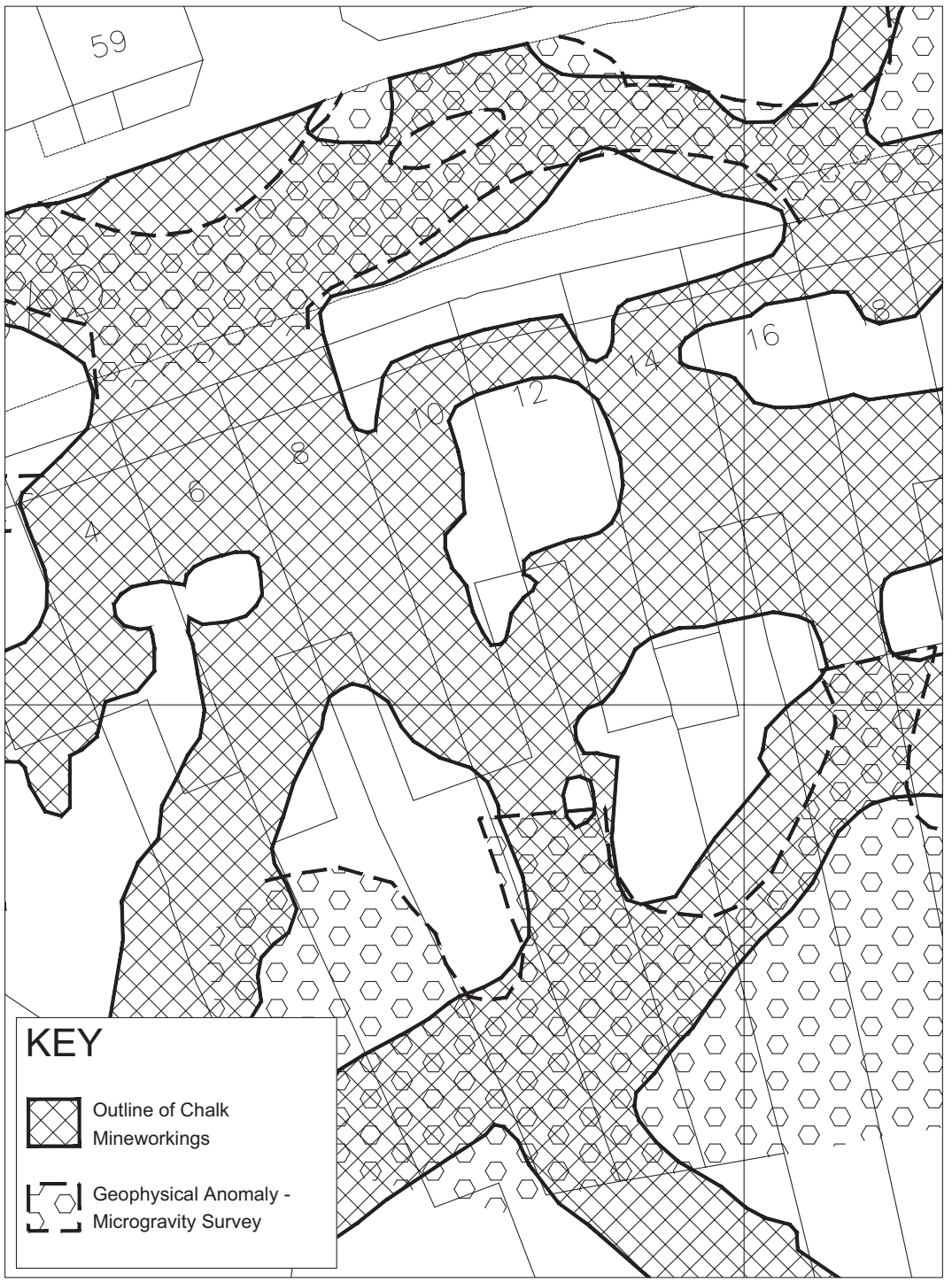


Figure 41.8 An example of the microgravity survey results compared with the actual mapped mine workings detected by dynamic probing

At Field Road, for example, if a probe encountered a void, the access to the void was enlarged using a window sampler borehole and plastic casing inserted into the hole to the roof level of the void. A small diameter borehole CCTV camera was then introduced via the casing, and a visual survey of the mine void was undertaken (see **Figure 41.9**). The direction of the mine tunnel was inferred from the survey, and the next probes were positioned accordingly, some 5–10 m away to intersect the tunnel further along, and the survey process was repeated again. This offered a cost-effective way of mapping the mine tunnels present, greatly reducing the numbers of probes required compared with adopting a conventional grid-pattern approach.

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Figure 41.9 Visual survey of a mine void using a small diameter borehole CCTV camera

Where dynamic probes cannot penetrate to sufficient depth to map mine workings, or angled holes beneath a building are required, rotary drilled boreholes can be used instead (see **Figure 41.10**).



Figure 41.10 An example of the use of rotary drilled boreholes
Courtesy of Peter Brett Associates LLP

41.2.7 Treatment of mine workings

Depending upon the nature and extent of the mine workings defined by the ground investigation, there are a variety of remedial mine treatment options. It is often the case that historical mine workings will comprise a mix of open voids, collapsed ground and further disturbed/broken ground over and around the mine workings.

Sometimes access to the mine workings may be possible from ground level, providing a means to infill and stabilise the workings using grouting techniques, e.g. progressive infilling behind shuttering (see **Figure 41.11**).

Often, though, it is unsafe to enter the mine workings, in which case the infilling works have to be undertaken from the surface, involving grout injection via boreholes. Open voids can be stabilised using bulk infilling methods (e.g. pumped as grout or another filler such as sand or pulverised fuel ash (PFA) washed in using water). Broken rock sequences overlying collapsed workings can be strengthened using pressure grouting techniques (high slump grouts) to seek out and infill dispersed voids or within unconsolidated sequences using compaction grouting techniques (low slump grout) to compact the loose material.

Where shaft entries have collapsed they may be infilled with a granular material, which is then solidified with a cementitious grout as necessary to counter settlement and loss at the base with time. The shaft top is then capped with a reinforced slab, whose depth below surface and size depends upon the nature of the near-surface ground conditions, shaft diameter and construction (see Healy and Head, 1984).



Figure 41.11 Infill and stabilisation of mine workings using grouting techniques
Courtesy of Peter Brett Associates LLP

Often raft foundations are utilised in areas of past mining to protect the structure from subsidence damage. However, if there is still a risk of voids rising to the surface, consideration needs to be given to the design span and cantilever of the raft versus the likely size of crown hole. In situations where the near-surface ground conditions are of poor quality but the risk of subsidence is deemed to be low, then piled foundations can be considered.

For shallow workings it might be feasible to pile down through the mine floor level, end-bearing into the stable ground below. Such piles need to be sleeved through the ground above the mine floor level to resist negative skin friction as the ground settles/collapses and to free-stand through voids if necessary.

In mined areas consideration also needs to be given to protecting services and infrastructure against subsidence damage. Some additional protection can be given by placing high-tensile-strength geogrid within the road pavement construction and below service runs.

The various ground treatment techniques and foundation types are described in detail elsewhere in this manual.

41.3 Contamination

Contamination of the ground by pollutants can be both a hazard and an obstruction. The nature of any possible contamination should be addressed in the desk study and this should be carried through to construction design and management (CDM) risk assessments for site activities by the investigation designer, regardless of the status of the investigation in CDM terms. The ICE Ground Investigation Specification (Site Investigation Steering Group, 1993) has a simple but somewhat dated site health and safety classification scheme for contaminants. This is useful in a contractual sense and should be completed, but it does not replace the need for a comprehensive designer's risk assessment. See Chapter 48 *Geo-environmental testing* for further discussion of the philosophy, sampling and testing of potentially contaminated ground.

If it is not addressed directly, contamination can also be a significant bureaucratic hazard in the UK. Where ground investigations are for projects involved in the Local Authority Planning process then third parties (the Local Authority and the Environment Agency (EA), or SEPA in Scotland) have a significant interest in the content of ground investigations and can control their content by the use of Planning Conditions. See Planning Policy Statement 23 Annex 2, which is available online (see section 41.11).

Failure to consider the needs of the Local Authority or the EA/SEPA or consult with them when designing a ground investigation could result in further investigation being required by planning condition. This is a significant matter. Such conditions would typically require an additional investigation to be completed and assessed before development could be commenced. The text of typical conditions is given in appendix 2B of Planning Policy Statement 23 Annex 2. An example is given below; it is clear that failure to discharge this condition could lead to a great deal of delay. This type of condition would apply to any type of site, even those where the apparent risk of 'contamination' appears to be very low. What the Local Authority seeks is a demonstration that the risk of contamination is negligible, or measures to deal with any confirmed source–pathway–target linkages.

> 'The development hereby permitted shall not begin until a scheme to deal with contamination of land and/or groundwater has been submitted and approved by the LPA and until the measures approved in that scheme have been fully implemented. The scheme shall include all of the following measures unless the LPA dispenses with any such requirement specifically and in writing:
>
> 1. A desk-top study carried out by a competent person to identify and evaluate all potential sources and impacts of land and/or groundwater contamination relevant to the site. The requirements of the LPA shall be fully established before the desktop study is commenced and it shall conform to any such requirements. Two full copies of the desk-top study and a non-technical summary shall be submitted to the LPA without delay upon completion.
>
> 2. A site investigation shall be carried out by a competent person to fully and effectively characterise the nature and extent of any land and/or groundwater contamination and its implications. The site investigation shall not be commenced until:
>
> (i) a desk-top study has been completed satisfying the requirements of paragraph (1) above;
>
> (ii) The requirements of the LPA for site investigations have been fully established; and
>
> (iii) The extent and methodology have been agreed in writing with the LPA. Two full copies of a report on the completed site investigation shall be submitted to the LPA without delay on completion.
>
> 3. A written method statement for the remediation of land and/or groundwater contamination affecting the site shall be agreed in writing with the LPA prior to commencement and all requirements shall be implemented and completed to the satisfaction of the LPA by a competent person. No deviation shall be made from this scheme without the express written agreement of the LPA. Two full copies of a full completion report confirming the objectives, methods, results and conclusions of all remediation works shall be submitted to the LPA.'
>
> Appendix 2B, Planning Policy Statement 23 Annex 2

Ground investigations have the ability to promote the transport of contaminants. Great care needs to be given to the design of ground investigations where mobile contaminants exist adjacent to aquifers or surface waters. Prosecution can be the result of a careless approach. See Chapter 48 *Geo-environmental testing* for more detail.

41.4 Archaeology

The possibility of archaeological remains being present on a site should be addressed in the desk study, preferably by approaching the Local Authority Planning Department in the UK. Where archaeological remains are likely to be found the Local Authority is likely to seek to influence the content and form of the ground investigation. This may have a significant impact on the duration, content and conduct of the investigation, and early consultation is strongly recommended. Specific archaeological investigations may be required, either as part of the ground investigation or as a separate exercise. See Chapter 43 *Preliminary studies* for more detail on desk studies and archaeology.

The archaeological content of combined archaeological and geotechnical investigations could range from a watching

brief for trial pits and boreholes, where an archaeologist examines the near-surface deposits as the geotechnical investigation passes through them, to specific boreholes and trial pits. Where archaeology-specific boreholes are required they usually involve continuous sampling of near-surface deposits. The destructive examination of these continuous samples, often off-site, usually precludes the use of these parts of the borehole for any geotechnical use. The Institution of Civil Engineers produces a useful standard specification and contract terms and conditions for archaeological investigations.

Finding archaeological remains, especially human bones or burial grounds, can cause significant delay to a ground investigation. Where bones are found that are suspected to be human the police should be called immediately. In these cases the police are likely to initially consider the site to be a crime scene. This will cause delay; however, the consequences of not contacting the police could be significantly more onerous. Testing for specific bacteria and viruses may be required, depending on the context of the find.

Archaeology can present significant restrictions to design and construction works. In a number of instances the archaeology must be left in place, and either bridged over or minimally disturbed by piling. See the English Heritage (2007) guidance note on piling and archaeology. Construction may be subject to delays in a similar fashion to the site investigation.

41.5 Ordnance and unexploded ordnance (UXO)

There are obvious hazards and potential delays associated with inadvertently finding buried explosives in a ground investigation. Guidance on assessing such risks and on mitigation techniques is given in CIRIA Report C681 (Stone *et al*., 2009). Higher risk areas are typically those in urban and industrial areas of major cities that, because of their historic situation, are such that the entry points of unexploded bombs might have gone unnoticed. Examples would be inter-tidal areas or areas of already demolished or damaged buildings. Desk-study information that can be used to assess UXO risks includes:

- historical maps (comparison of pre- and post-war maps);
- wartime bomb damage maps;
- wartime records of 'abandoned' unexploded bombs.

Detailed guidance is given in CIRIA Report C681 (Stone *et al*., 2009).

Unexploded bombs can move significant distances laterally below ground at the time of impact. Unexploded bombs can also move significant distances after impact where they have fallen into large tidal rivers. There are even known instances of unexploded bombs being unknowingly picked up, placed in fill, and then dug up again several years later.

Backfilled explosion craters also raise the possibility of otherwise unexpected ground conditions and the selection of inappropriate ground investigation techniques. These issues should be considered at the desk-study stage. Where backfilled craters are expected, low-cost simple techniques that can be used at relatively close centres can be used to identify variations in backfill thickness. These would include trial pits in general and window sampling for pits filled with the local natural soil if that is generally suitable for this technique. Buried craters in urban areas are more likely to be backfilled with coarse demolition rubble. Craters in more rural settings, such as airfields, are more likely to be backfilled with the soil displaced by the impact and explosion. Geophysical techniques may also be of use in this scenario. The choice of technique will depend on the nature of the backfill compared to the natural soil.

Where there is considered to be a higher than usual risk of encountering UXO in a borehole, the usual mitigation choices are to move the borehole, to abandon the borehole, or to employ surface or more usually down-hole magnetometer techniques to assess the presence of a bomb before the borehole reaches it. This activity requires the use of stainless steel casing and, as such, must be pre-planned.

As the site investigation is the first intrusion on to a site, the requirements of the investigation are often more onerous than during later construction. However, probing or further magnetometer surveys may be required during piling, and strict procedures followed when employing bulk excavation.

41.6 Buried obstructions and structures

Made Ground can contain man-made obstructions in the form of cobbles and boulders of natural and man-made materials, either placed as fill or as a result of *in situ* demolition. Typically, any large clasts like these are going to be stronger materials (rock, concrete, brick, steel and timber). These are likely to cause some degree of difficulty for any drilling or probing technique and possibly for trial pitting using smaller machines. The use of larger machines may be constrained on-site by space constraints and off-site by difficulties in accessing the site with low loaders. The desk study can indicate to a modest degree the likelihood of encountering these types of obstructions.

There are many different types of buried structure that a ground investigation might experience as an obstruction. Examples include foundations, old floors, basements, tunnels, culverts, machine chambers and membranes. These structures can be solid or can contain space filled with other solid material, liquids or gases, possibly under pressure. Consideration of these matters is a basic part of a desk study, and identification of significant obstructions would normally be a defined aim of ground investigations. In addition to probing or other intrusive methods of locating these, various geophysical techniques can be used (see Chapter 45 *Geophysical exploration and remote sensing*).

When considering this issue during a desk study it is useful to classify buried structures in one of two ways, those in use and those that are disused. Buried structures that are in use typically have owners who are keen to protect them. Their advice should be sought. This advice may significantly constrain the form and content of the investigation, for example restrictions

on drilling close to tunnels. Examples of such structures would be major sewers, major pipelines, communications ducts or tunnels, railway and road tunnels, buried storage tanks, culverts and penstocks.

Disused and 'ownerless' underground structures are rare and are necessarily harder to indentify. The types of structure that might be encountered are all those listed above plus air-raid shelters and other historical military and civil defence infrastructure.

Since many people are very interested in these types of underground spaces the internet is a useful source of data. In the UK, Subterranea Britannica can be a good source of information and further links, particularly for disused military and civil defence infrastructure (see section 41.11).

The desk study and walkover (Chapter 43 *Preliminary studies*) present the best opportunity to identify disused buried structures before ground investigation commences. This issue should be actively considered in each and every desk study. The ground investigation has the potential to significantly damage a buried structure and in so doing create hazards by instigating collapse or by releasing liquids or gases. The desk study should address the likelihood of encountering buried structures and identify strategies for dealing with that occurrence. Buried structures can also be affected by ground investigations even though the ground investigations do not directly encounter the structure. Examples of this possibility would include excavations leading to settlement damage, excavations damaging tree roots, and pumping of groundwater leading to settlement of nearby structures.

Where ground investigations have finding buried structures as a specific aim then the choice of technique will depend on safety considerations. Questions about whether the structure can afford to be damaged, and consideration of the consequences of any damage, need to be addressed before a technique can be selected. Machine-dug trial pitting is often used for shallow robust structures. Shallow structures which cannot be damaged will often be investigated using hand-dug pits with suitable support. If the structures contain voids, non-intrusive geophysical techniques such as ground probing radar may be useful (see Chapter 45 *Geophysical exploration and remote sensing*).

41.7 Services

Buried services constitute one of the greatest health and safety hazards in ground investigation. If encountered and damaged they can also be the source of significant delay and expense and can instigate pollution of the ground. All ground investigations, especially those on brownfield and urban sites, should incorporate some or all of:

- risk assessment;
- searches with utility providers;
- surveys to locate and trace services;
- hand-dug starter pits.

Contractual responsibilities for these activities should be clear, and sufficient time and money should be allowed for their completion. See Chapters 42 *Roles and responsibilities* and 44 *Planning, procurement and management* for more information on roles and responsibilities.

Since searches with utility providers can take several weeks to complete, this activity should be started as early as possible in the desk-study process. Clients and designers have responsibilities in this area, particularly under CDM regulations (see Chapter 42 *Roles and responsibilities* for more detail). Consideration should be given to carrying out pre-investigation service traces. The cost of this can be mitigated if this work is combined with topographic surveys in cases where the project requires such surveys anyway. Ideally such surveys will involve lifting manhole covers and inducing signals along services that can be reliably traced at the surface. This information can then be rechecked on-site at the time of the ground investigation using portable 'Cat Scanning' equipment. This equipment, however, is not effective with all services and great care should be used when relying on its use in the absence of other information or surveys.

Disused and derelict sites may contain both live and dead services. Dead or disused services can still be hazardous and should be treated with the same caution as live services. Disused gas pipes can still contain explosive gas, fuel pipes can contain fuel, and so on. Older high voltage (HV) cables often contain asbestos paper wraps and can contain toxic oil-filled jackets. It is often useful to consider the former and current uses of a site. Was HV required and where? How were structures heated? Was fuel (gas/oil) distributed around the site and how?

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41.8.3 Useful websites

British Geological Society; www.bgs.ac.uk
The Coal Authority; www.coal.gov.uk
Cornish Mining World Heritage; www.cornish-mining.org.uk
Environment Agency; www.environment-agency.gov.uk
History of Bathstone Quarrying; www.choghole.co.uk/Main%20page.htm
Kent Underground Research Group; www.kurg.org.uk
Mine-Explorer, The Home of UK Disused Mine Exploration; www.mine-explorer.co.uk
Planning Policy Statement 23: Planning and Pollution Control – Annex 2; www.communities.gov.uk/publications/planningandbuilding/pps23annex2
Scottish Environment Protection Agency (SEPA); www.sepa.org.uk
The Slate Industry of North and Mid Wales; www.penmorfa.com/Slate
Subterranea Britannica, a good source of information and further links, particularly for disused military and civil defence infrastructure; www.subbrit.org.uk
UK Minerals Yearbook; www.bgs.ac.uk/downloads/browse.cfm?sec=12&cat=132

It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 8 *Health and safety in geotechnical engineering*
- Chapter 48 *Geo-environmental testing*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

Chapter 42
Roles and responsibilities

Jim Cook Buro Happold Ltd, London, UK

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This chapter provides guidance on the roles and responsibilities of the participants involved in site investigations. The guidance is based upon key documents, which have been published by various organisations involved in this industry. These documents are referenced and it is recommended that they be read.

The chapter focuses on the non-technical issues regarding site investigation work. Comments are given on the Institution of Engineers Site Investigation Steering Group guides and on the obligations under the Construction (Design and Management) Regulations 2007, Corporate Manslaughter and Health and Safety.

The section on conditions of engagement provides commentary on tendering requirements and submissions, including comments on the current status of conditions of contract.

The timing of when ground investigations ought to be carried out is discussed and comparisons are made between the design stages recommended by the Association for Consultancy and Engineering and the Royal Institute of British Architects.

Commercial issues relating to consultants and ground investigations are discussed with comments on the traditional methods of procurement, the one-stop shop approach and the situation when the consultant carries out the ground investigation.

Underground services are also mentioned and the chapter concludes with guidance on commercial issues that need to be considered, such as insurance.

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42.1 Introduction to site investigation guides

It is important to understand the elements of work that make up a site investigation and which people are going to do what and their individual responsibilities. It is usual that the work elements of a site investigation are undertaken by separate organisations. There are usually two entities: a consultant and a specialist contractor, both of whom work on behalf of the client (the employer).

The client, aside from their normal obligations, is usually responsible for obtaining all the appropriate planning and building regulation approvals and possession of the site prior to the commencement of any site inspection or intrusive investigation work.

It is incumbent upon the organisation (usually the consultant) that is producing the site investigation tender documentation to provide all available information including client-supplied information or advise where it can be obtained or viewed.

The objective and scope of a site investigation should reflect the requirements of the intended structure or building. It is important that the briefing section of the tender document provides all relevant information and any particular requirements or potential hazards in undertaking an intrusive investigation, e.g. boreholes or trial pits constructed close to or near to existing structures that may be above or below ground.

The method to be adopted for carrying out a site investigation should be appropriate to the ground conditions at the site and the ground-engineering testing and sampling requirements. These methods are usually specified by the consultant, who would approve the specialist contractor's method statement.

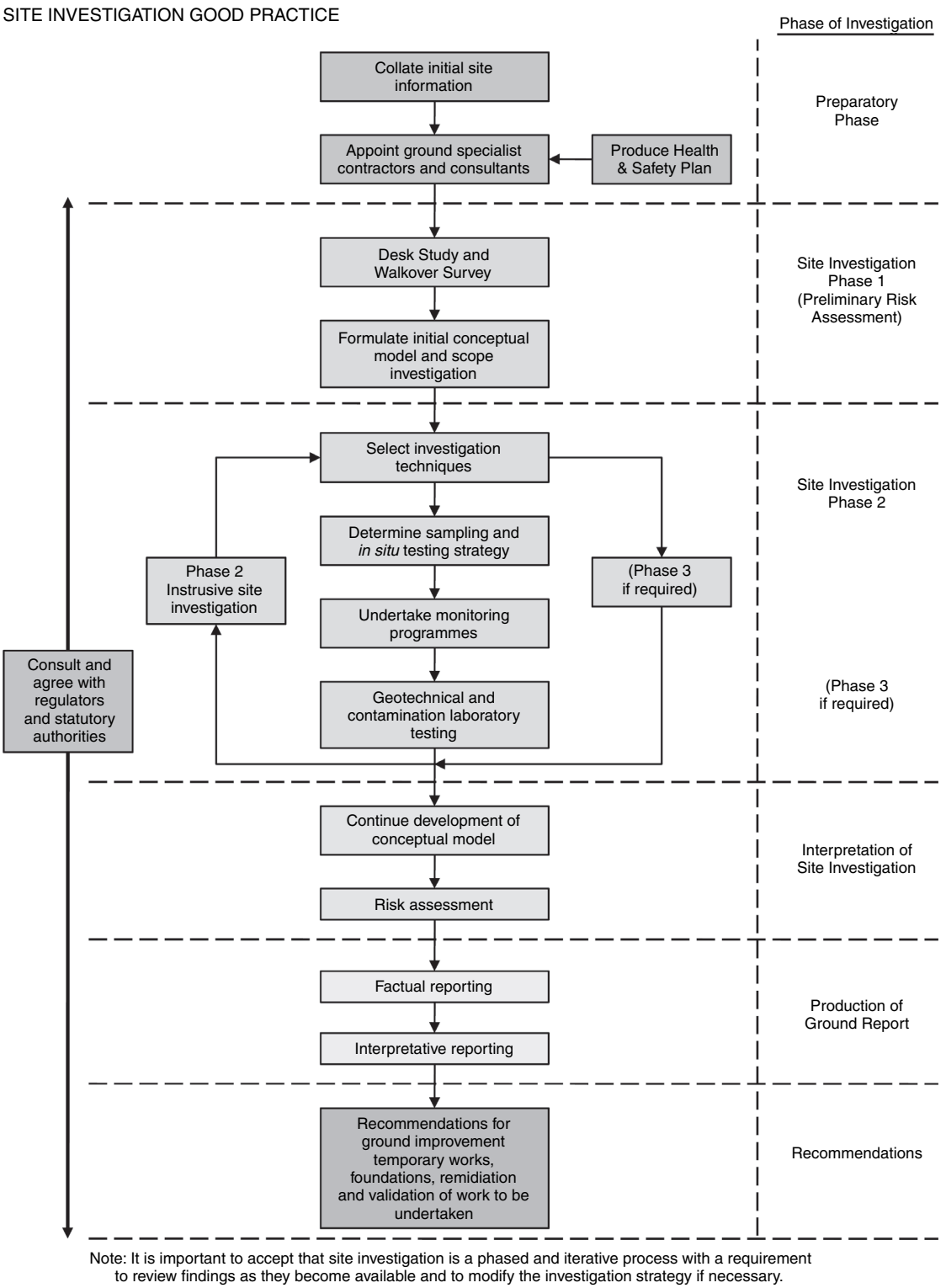
Cottington and Akenhead (1984) provide useful guidance on the contractual responsibilities of the parties involved in carrying out a site investigation.

Figure 42.1 shows the sequence of activities for a site investigation. Notice that it is an iterative process.

First, a desk study is produced, which is usually undertaken by a consultant. This is followed by fieldwork, which is usually called the ground investigation, and this is then followed by laboratory testing and factual reporting. The ground investigation can be intrusive or non-intrusive (or a combination of both) and it may be carried out in several phases. This element of the investigation is usually undertaken by a specialist contractor. The factual report provided by the specialist contractor is generally reviewed by the consultant and the interpretative report and recommendations are produced by the consultant.

There are variations on the above, generally dependent upon the size of the project, the client's requirements and so on.

A basic responsibility for the consultant, who has been appointed to arrange and procure the investigation on behalf of the client, is to ensure the work is undertaken in accordance with the appropriate specification, standards and conditions of engagement. All work should be carried out in accordance with what is known as 'best practice'. The Institution of Civil Engineers (ICE) has produced a specification and conditions of contract document and the Association of Geotechnical and Geoenvironmental Specialists (AGS) provides very good guides on these matters (details are given in Further reading). Details on codes and standards and their relevance can be found in Chapter 10 *Codes and standards and their relevance*.



Note: It is important to accept that site investigation is a phased and iterative process with a requirement to review findings as they become available and to modify the investigation strategy if necessary.

Figure 42.1 Site investigation activities
Reproduced from AGS (2006)

ICE has produced a series of documents, the *Site Investigation in Construction Series* (ICE, 1993), under the guidance of the ICE Site Investigation Steering Group (SISG), a body which includes various learned societies and trade associations. These documents were supported and endorsed by many major organisations such as the Association of Consultancy and Engineering (ACE), the British Geotechnical Association (BGA), the Institution of Civil Engineers (ICE), the Department for Transport (DfT), the Department of the Environment, Transport and the Regions (DETR), the Construction Industry Research and Information Association (CIRIA), then National House Building Council (NHBC), to name but a few. The above documents quickly became accepted as the industry norm but since their publication the methods and techniques of undertaking a ground investigation have changed and geo-environmental aspects have come more to the fore in view of the increase in brownfield development. As a result, the series is currently being revised by the steering group to take these changes into account and will be published by ICE Publishing in 2011/12. It is expected that the revised *Site Investigation in Construction Series* will quickly become key industry guides.

42.2 CDM regulations (2007), corporate manslaughter and health and safety

Site investigation is an investigation into the ground conditions at a particular site, which could be a greenfield site, where no development has previously been carried out, or, more likely in the United Kingdom, a brownfield site, where the land is to be redeveloped. Any work carried out on site, be it a simple walkover survey as part of a desk study or a full intrusive investigation, involves varying degrees of risk (see Chapter 43 *Preliminary studies*).

The Construction (Design and Management) (CDM) Regulations 2007 apply to all construction projects which include a site investigation. Section 2.1 of the regulations details their application and gives the responsibilities required, and these are discussed below.

42.2.1 Application of the CDM Regulations (2007)

The application and obligations of the client regarding ground investigations are fully described in AGS's *Client's Guide to Construction (Design and Management) Regulations (CDM) 2007 for Ground Investigations* (AGS, 2008). This guide was published subsequent to the CDM regulations that came into force on 6 April 2007. The following section is mainly based upon the AGS guide, which should be read, with supporting comments from the author.

It is important to note that under the CDM regulations, the client has the ultimate responsibility even if the investigation is procured or planned by a third party. In the case of a ground investigation any intrusive fieldwork, such as boreholes or trial pits, is considered as construction work.

The CDM regulations are applicable to all ground investigations that involve any form of intrusive work in the ground. A key issue concerning CDM regulations is whether a ground investigation is notifiable to the Health and Safety Executive (HSE). A ground investigation is notifiable if the construction period is likely to comprise 500 person-days or more, or the construction work is likely to last 30 project working days or more. The client is required to appoint a competent CDM coordinator and principal contractor for a notifiable ground investigation project. If the ground investigation is included within an existing or intended project, it will be notifiable as part of that particular project.

For a ground investigation where no specific development is planned and the work is wholly non-intrusive then the CDM Regulations 2007 probably do not apply. However, if site visits or walkover surveys are to be undertaken, due consideration must be given to health and safety issues for personnel undertaking this type of work, i.e. site visit risk assessments and method statements will be required anyway.

For site investigations that are to be carried out in potentially high-risk environments, e.g. over water, on or adjacent to railways and highways or underground, specialist training is likely to be required within a more rigorous health and safety regime.

42.2.1.1 Client's obligations

It should be noted that the client's responsibilities under the latest CDM regulations have increased compared to the previous regulations of 1994. A key issue is statutory obligations, any breach of which is a criminal offence. The client cannot abdicate their responsibilities but can delegate particular tasks to others, including third parties. For notifiable projects the CDM coordinator is obliged to advise and aid the client to comply with their duties.

The client's key responsibility is to establish the health and safety regime. This is achieved by undertaking the following for all projects:

- engage suitability qualified designers and contractors, who have been fully assessed for competency;
- ensure all relevant parties (including designers and contractors) receive appropriate pre-construction information as soon as it becomes available;
- ensure that all parties (including the client) have sufficient resources at all stages of the project;
- ensure the project timeline is appropriate for each stage of the project.

If a project is deemed to be notifiable, additional duties are required:

- Appoint a suitably qualified CDM coordinator and principal contractor. This should be carried out either at or (ideally) before the design stage.
- Ensure the CDM coordinator is given all relevant project information as soon as it is available.
- Ensure a health and safety plan for the site work is in place. This should be produced in liaison with the CDM coordinator and client. Site work should not commence until this is in place.

- Ensure adequate staff facilities are in place on site before work commences.
- Ensure the CDM coordinator has produced a health and safety file. This should be passed to the client at the end of the project, and will require updating if modifications to the structures are carried out.

When a client fails to appoint a CDM coordinator or principal contractor, the duties of these parties default back to the client. The designer also has an obligation to make the client aware of their duties with regard to the CDM regulations. The designer also has obligations under the CDM regulations for any design decisions undertaken by the client.

42.2.1.2 Advice to clients

Where intrusive work is undertaken as part of a ground investigation, the CDM regulations apply even if the project is not notifiable. Clients should carry out the following:

- Provide pre-construction information to designers, contractors or others to allow them to plan and manage their work effectively and comply with Regulation 10 (which deals with the client's duty in relation to information) as follows:
  - a description of the project including the programme, key dates and arrangements for managing health and safety;
  - information available from the existing health and safety file (if there is one);
  - details on existing statutory services (and private services if available);
  - details of existing structures (including drawings, specifications, etc.) if available;
  - details of any environmental restrictions and any existing on-site risks;
  - details of activities on or adjacent to the site that might impact on the ground investigation.
- Confirm whether or not a project is notifiable to the HSE.
- Ensure competent designers and contractors are always appointed. This also applies to CDM coordinators and principal contractors where appropriate.
- Ensure reasonable management arrangements are in place throughout the project to ensure construction work can be carried out, so far as reasonably practicable, safely and without risk to health.
- Ensure suitable welfare facilities are to be provided from the start and throughout the construction phase by the contractor or principal contractor, as appropriate.
- On notifiable projects appoint, in writing, a CDM coordinator and principal contractor.
- Ensure construction work does not start until a suitable construction plan has been prepared and welfare arrangements are in place.
- Ensure the health and safety file is retained and updated if the structures described in it are modified.

42.3 Corporate manslaughter

Corporate manslaughter is now on the United Kingdom statute books through The Corporate Manslaughter and Corporate Homicide Act 2007. This is a landmark in law and came into force on 6 April 2008. Under this act, companies can be found guilty of corporate manslaughter (or corporate homicide in Scotland, as there is no such thing as 'manslaughter' under Scottish law) as a result of serious management failures resulting in a death or deaths as a direct result of a gross breach of their duty of care. The act clarifies the criminal liabilities of companies including large organisations where serious failures in the management of health and safety result in a fatality.

An organisation is guilty of corporate manslaughter if the way in which its activities are managed or organised causes a death and amounts to a gross breach of a relevant duty of care to the deceased. Companies and organisations should keep their health and safety management systems under review, in particular, the way in which their activities are managed and organised by senior management.

Sadly the first conviction under this act occurred in February 2011, when a geotechnical company was deemed to be responsible for the death of a young geologist who was killed whilst working inside an unsupported excavation. Concerning this very important piece of legislation, the reader should refer to *A Guide to the Corporate Manslaughter and Corporate Homicide Act 2007* published by the Ministry of Justice (2007) and the Health and Safety Executive and Crown Prosecution Service websites given in the references.

42.4 Health and safety

The revised ICE *Site Investigation in Construction Series* (ICE, 1993) provides guidance on health and safety issues, some of which is based upon the British Drilling Association (BDA) document *Guidance for Safe Intrusive Activities on Contaminated or Potentially Contaminated Land* (BDA, 2008), which was published in 2008. The BDA document provides health and safety information and recommendations on best practice for ground investigations.

This document describes the rationale and the need for new guidance for ground investigations, mainly due to an increase in statutory and regulatory legislation, changes to working practices and an increase in brownfield site development.

This guide is comprehensive: it has 17 chapters ranging from appropriate legislation, including four acts and 11 regulations, followed by chapters on competence, training and qualifications, managing health and safety, risk assessment and so forth.

Further information regarding health and safety can be found in Chapter 8 *Health and safety in geotechnical engineering* of this manual.

42.5 Conditions of engagement

A ground-investigation specialist can be engaged using bespoke terms, which are drawn up specifically for a particular

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project, or using the terms and conditions of contract produced by public bodies such as the Highways Agency.

ICE has produced two sets of *ICE Conditions of Contract for Ground Investigation*, the first set was published in 1983 and the latest set (the second edition) was published in November 2003 together with a set of guidance notes (ICE & CECA, 2003).

ICE, ACE and the Civil Engineering Contractors Association (CECA) are sponsoring bodies of the Conditions of Contract Standing Joint Committee (CCSJC). Representatives of members of CCSJC and members of the BGA and AGS all provided input and guidance into the second edition.

The second edition has been specifically written to focus on the contractual requirements of ground investigation and in particular on fieldwork, including laboratory testing and factual reporting. The second edition is not considered to be appropriate for producing interpretative reports and, therefore, a different form of agreement should be used for this type of work. A form of ACE agreement could be used for interpretative reports.

It is recommended that where possible the second edition is used for ground-investigation contracts.

The ICE conditions of contract can be used for all types and sizes of ground investigation. Normally for building projects, the engineer, who would be acting on behalf of a client, would have an 'in-house' specialist ground-engineering team. This team would review the client's requirements with the structural engineer and architect and then design an appropriate site investigation including the ground investigation.

Ground investigation includes fieldwork, laboratory testing and factual reporting. Management, site supervision, technical reviewing and all engineering reporting would be carried out by the engineer's ground-engineering team.

This specialist team would produce a ground-investigation tender document, which would comprise a specification, bill of quantities and a commercial package based upon the *ICE Conditions of Contract for Ground Investigation Revision 2*. It is good practice to include supporting appendices, which may include assessments of potentially hazardous sites, a CDM hazard questionnaire and any pertinent client-provided information on site constraints, e.g. buried services.

The commercial package would include all aspects deemed necessary on behalf of the client including: cost, programme, insurance, damages, liabilities, payment terms and retention as appropriate. The tender document, including the conditions of contract, should be agreed with the client prior to being issued.

The tender would then be sent out to an agreed number of client-approved ground-investigation specialist contractors. As part of the tender return submission, and in addition to completion of the commercial package, the specialist contractor should also submit appropriate documentation concerning how the work is going to be undertaken, e.g. method statements. Most codes recommend a phased approach; the RIBA stages (see section 42.6) are a useful guide to the requirements of each stage.

The above process is also a suitable basis for civil engineering-type projects as well as building-type projects.

In situations where the main engineering assignment for the client is carried out by the lead engineer under the ACE Conditions of Contract, there are two elements of site investigation work deemed to be included within the overall fee package. These are a desk study and the production of a ground-investigation specification. Any other requirements of the site investigation, such as site supervision, fieldwork, laboratory testing and interpretative reporting, are items of additional work, which need to be priced and included in the ACE agreement. Geotechnical design, including foundation specification, is deemed to be included within the overall agreed fee.

For smaller ground-investigation projects, it is not uncommon for a simple exchange of letters, between either an architect or a structural engineer acting on behalf of the client and a specialist site investigation contractor, to form the basis of the contract.

In this situation it is important that the specialist site investigation contractor has the appropriate level of resource, competence and experience to undertake all the work required, including a desk study, designing a specification, fieldwork, laboratory testing and producing an interpretative ground-engineering report.

It should be noted that support for the ICE Conditions of Contract has been withdrawn by ICE and support for these conditions is expected to be taken over by ACE and CECA. It is also likely that the New Engineering Contract (NEC3) conditions of engagement (Institution of Civil Engineers, 2005) will be developed to include ground-investigation work. The ground-engineering industry together with NEC will need to decide if one of the main options in the NEC3 conditions of contract is adopted or alternatively whether *Engineering and Construction NEC3 Short Contract* (ECSC; ICE, 2005) can be used and tailored to suit this specialist type of work.

42.6 When should a ground investigation be carried out?

The intrusive part of a site investigation can be carried out during a single site visit or over several phases of fieldwork. It is usual that at least a desk study is undertaken for a project that is going to be submitted for planning approval, as required by ODPM PPS 23 (ODPM, 2004). This is called a phase 1 site investigation.

Subject to project requirements, an initial ground investigation is undertaken focusing on geo-environmental concerns for brownfield sites.

The phase 1 site investigation work could also include geotechnical as well as geo-environmental aspects and, thus, it is generally more prudent and economic to have a combined phase 1 site investigation focusing on both aspects.

AGS are currently proposing to the building regulators that a combined ground investigation should be mandatory during planning approval.

Ground investigations should be carried out on a phased basis and need to take into account the design process. There

are several design phases in building design and these generally comprise:

1. feasibility and concept design;
2. scheme design;
3. design development;
4. final (detailed) design;
5. production information and tender;
6. construction to practical completion;
7. after practical completion.

RIBA phasing has design stages A, B, C, D to L while ACE has design stages C1, C2, C3, C4 to C8. These are given in the following references: RIBA Outline Plan of Work 2007 Amended 2008 and ACE Agreement B (1) 2002 (revised 2004) Civil/Structural Engineering Non Lead Consultant.

The above ACE 2004 document was revised in 2009 and renamed ACE Schedule of Services Part G (a) Civil and Structural Engineering Single Consultant or Non Lead Consultant for Use with ACE Agreement 1 – Design 2009. These revised work phases are given in the fourth column in **Figure 42.2**. In current practice the earlier 2004 version is still in extensive use in the industry. The different stages and services are compared in **Figure 42.2**. The ground-engineering input required in the RIBA stages is shown in **Figure 42.3**.

From **Figure 42.3** it can be seen that the site investigation spans the early stages of design for RIBA (stages A to C). A phase 1 ground investigation would typically be undertaken during stages A or B during the site appraisal and the phase 2 detailed ground investigation would be completed ideally before stage D commences.

Phase	RIBA Stage	ACE B1 2002 rev 2004	ACE Part G(a) 2009
Feasibility	A (Appraisal)	C1	G2.1
	B (Strategic Briefing)	C2	G2.2
Pre-Construction	C (Outline Proposals)	C3	G2.3
	D (Detailed Proposals)	C4	G2.4
	E (Final Proposals)	C5	G2.5
	F (Production Information)	C6	G2.6
	G (Tender Documents)	C7	G2.7
	H (Tender Action)		
Construction	J (Mobilisation)	C8	G2.8
	K (Construction)		
	L (After Practical Completion)		

Figure 42.2 Comparison of RIBA Stages and ACE Services
Courtesy of Buro Happold Ltd

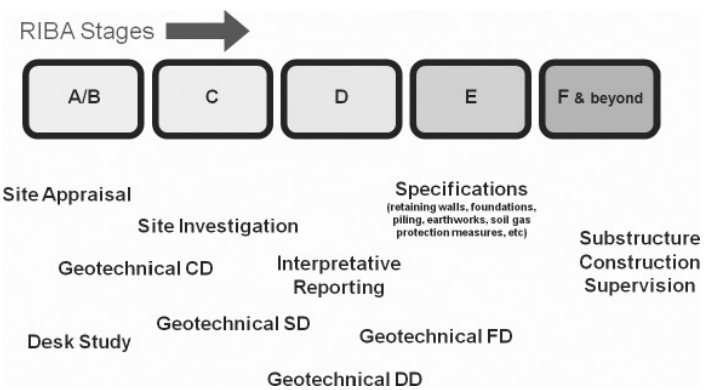


Figure 42.3 Typical ground-engineering input during RIBA stages; CD Concept Design, SD Scheme Design, DD Detailed Design and FD Final Design
Courtesy of Buro Happold Ltd

42.7 Consultants and ground investigations

42.7.1 Background

It is becoming more commonplace that consulting engineers are asked to provide a 'one-stop shop' and to incorporate the provision of ground investigation and laboratory services within the scope of their design contract with the client or the client's agent.

Consequently in this instance the consultant is required to employ a ground-investigation contractor directly or to perform the ground investigation using his own staff with his own or hired equipment. The consultant, therefore, becomes the employer of the specialist ground-investigation contractor and is acting in the role of a contractor (rather than just a consultant) to his client or his agent. For some consultants acting as a contractor is an entirely new situation.

AGS has produced a document, *Consultants Undertaking Ground Investigation Contracting*, within the *AGS Toolkit* suite of guides (AGS, Toolkit), concerning this change of role. The following comments are not an exhaustive description of how to deal with all the issues associated with the changes that consulting engineers must now adopt; instead they are intended to raise awareness of some of the issues that will be faced by consulting engineers so that further guidance, as appropriate, can be obtained.

One of the key issues here is that the consultant bears the financial risk of the cost of the specialist ground-investigation contractor. The guide identifies the need for the development of an appropriate standard contract form for use between the consultant and his client in this scenario.

Traditionally for site investigation work, consultants have acted as the engineer as defined in the ICE Conditions (ICE & CECA, 2003) and other similar contract forms such as ACE (2004). Under this arrangement the consultant has no direct contractual arrangement with the ground-investigation contractor.

In circumstances where the consultant procures the ground-investigation services on behalf of the ultimate client under the contractual scope of supply, there will be obligations to the ground-investigation and laboratory contractor that will need to be properly dealt with in the contract with the client. The

consultant will need to be remunerated for the additional effort and business risks.

Generally the most successful ground investigations have been performed within a contractual relationship that readily allows flexibility in work scope due to variations in anticipated ground conditions.

There has also been a tendency for clients or their agents to request 'lump sum' ground investigations (which include laboratory testing). It is likely that a lump sum form of contract that constrains the ground-investigation work scope, with resulting inflexibility, potentially places the party holding the risk of the lump sum in a conflict between commercial and professional obligations.

The AGS guide (AGS, 2008) provides a background to the way ground investigations have traditionally been procured and comments on the one-stop shop and lump sum approaches.

There are different types of contract that can be used for ground-investigation work and the most popular in recent times has been the *ICE Conditions of Contract for Ground Investigation Version* (ICE & CECA, 2003). These conditions are generally well known to consultants and ground-investigation contractors and have been used for many ground-investigation projects. They may be suitably modified for the one-stop shop role. Other forms of contract such as ICE Minor Works (Institution of Civil Engineers, Association of Consulting Engineers, Civil Engineering Contractors Association, 2001) and NEC can be used.

See also Chapter 44 *Planning, procurement and management*.

42.7.2 Traditional approach

The traditional procurement route of engagement is where the client engages a consultant engineer who undertakes the procurement of a ground-investigation contractor on behalf of the client to undertake a ground investigation. This process is fully described in the AGS guide (AGS, 2008).

There are advantages and disadvantages of this method of appointment, which are well explained in the guide. The risks to the client (e.g. an increase in overall contract costs) and the responsibilities of the client (e.g. prompt payment) are all listed.

The duties of the engineer (e.g. certification of invoicing) and the responsibilities of the contractor (e.g. carrying out the work to the agreed programme) are also given in detail.

The commercial arrangements for the client in this approach mean that there has to be a separate contract for the consultant to deal with the requirements of the ground investigation and there has to be another contract for the ground-investigation contractor.

This two-contract approach may seem cumbersome and this has probably resulted in the adoption of the single appointment, the one-stop shop.

In this arrangement, the agreement between the client and the consultant can be an ACE or ICE-type consultancy agreement, and that between the client and the ground-investigation contractor can be the *ICE Conditions of Contract for Ground Investigation* (ICE & CECA, 2003).

42.7.3 One-stop shop approach

The one-stop shop approach is a non-traditional arrangement that allows the client to have a single agreement with the consultant to deal with the requirements of the ground investigation and the ground-investigation contractor.

This arrangement transfers a certain degree of the client's commercial risk to the consultant, who would usually lead the arrangement. These issues include risks such as responsibility for payment of the ground-investigation contractor, whose input into the project is usually substantially more than the fees attributable to the consultant's input. The risk of default by the client and funding claims for extra work can be substantial.

The additional responsibilities that the consultant takes on need careful consideration and may be considered as being unfair. There are also possible concerns from the ground-investigation contractor's point of view.

The key issues for both consultants and ground-investigation contractors in undertaking one-stop shop investigations are discussed in the AGS guide (AGS, 2008).

42.7.4 When the consultant undertakes the ground investigation

Consultants have successfully carried out ground investigations for several years without the benefit of a form of contract that has been specifically developed for the combined role. There are risks associated with this form of arrangement, which are the same as those which a ground-investigation contractor may have faced outside those of the normal consulting agreements.

These risks are listed in the AGS guide (AGS, 2008) and include damage to underground services, health and safety issues (see sections 42.2 and 42.4) and loss of samples, to name but a few.

Additional areas of concern for the consultant are given in the guide and key concerns are the provision of necessary and suitable insurance, as standard insurance arrangements may not be adequate. The consultant would be well advised to ensure project cover is in place (typically through a project contractor's all-risks policy).

Regarding the ground investigation, the guide gives three options as to how this may be undertaken, that is, through subcontracting, work packaging and project managing, but all three are led by the consultant.

There are various forms of contract for undertaking this type of assignment but they may require some modification. The use of the ICE form of agreement may be considered but it should be borne in mind that the *ICE Conditions of Contract for Ground Investigation* (ICE & CECA, 2003) were specifically produced for the ground-engineering contractor to carry out the fieldwork, laboratory testing and factual reporting.

42.7.5 Particular issues when the consultant employs a ground-investigation contractor under the ICE contract

There are several fundamental points to be noted when the ICE conditions of contract (ICE & CECA, 2003) are used in the situation where the consultant employs a ground-investigation contractor. The consultant becomes the employer and also the engineer at the same time. This can cause major concerns, in particular the express contractual provision for the engineer to act impartially and regarding the resolution of disputes. These two matters require modification to clauses 2(7) and 66 of the ICE conditions.

There are at least 10 key responsibilities listed in the AGS guide (AGS, 2008) that affect the consultant who is acting as an employer; these include payment, liabilities for unforeseen ground conditions, wayleaves and permission, etc.

The guide gives comments and certain courses of action for the consultant, which may be adopted in setting up the contract, which are likely to minimise the consultant's level of risk. The guide also provides advice and 'health warnings' for the consultant if they propose to use the *ICE Conditions of Contract for Ground Investigation Version* (ICE & CECA, 2003).

42.7.6 Other insurance

In addition to the contractor's all-risks insurance mentioned above, there are three important types of insurance that should be considered by both consultants and ground-investigation contractors: professional indemnity (PI) insurance, public liability (PL) insurance and employer's liability insurance.

PI insurance is usually carried by consultants or ground-investigation contractors to cover liabilities arising from their interpretation, engineering, design and recommendations for a project. For geo-environmental aspects the level of cover offered by insurance companies can be much lower than those offered for geotechnical aspects due to the perceived higher risk of dealing with contaminated ground.

PL insurance is usually taken out by companies to cover the potential affects to third parties caused in carrying out their business activities. This includes bodily harm or death to members of the public or damage to property.

Regarding PI and PL insurance, organisations should consider all potential risks in order to decide upon the level of insurance needed and the amount of insurance cover deemed appropriate. It is important to note that the level of insurance cover offered in a contract does not necessarily limit the degree of exposure (total liability) to the sum insured unless specifically agreed.

The Employer's Liability (Compulsory Insurance) Act 1969 sets out a minimum level of employer's liability insurance cover, which is currently at £5 000 000. This insurance cover is needed as the employer is responsible for the welfare of employees when they are at work, either in the office or on site.

Further information should be obtained from the various insurance companies and brokers, who offer advice and guidance. The Insurance Institute of London provide publications on the above and other types of insurance.

42.8 Underground services and utilities

The likelihood of there being underground services and utilities is extremely high for sites in developed areas and less so for more greenfield sites. It is also worth mentioning that there are above-ground services and utilities that should be considered and these are not dealt with here. In undertaking any form of intrusive ground investigation, albeit trial pits or boreholes, there is a risk of encountering services and utilities.

In general, the client's engineer provides the location plans, showing the existing services and utilities for the project, to the ground-investigation contractor. These may be provided to the engineer by the service or utility-owner company. The ground-investigation contractor would then arrange for a non-intrusive survey (cable and pipe locating) to be carried out to confirm the location on the ground and subsequently excavate a test pit to confirm the existence of the services and utilities for each borehole or trial pit.

The question of responsibility is one which needs to be carefully considered. The recently published Health and Safety Executive document *Avoiding Danger from Underground Services* (Health and Safety Executive, 2000) provides guidance and also outlines dangers and risk reduction methods.

This publication focuses specifically on health and safety but also points out that by considering this there will be a reduced risk of damage to services and property. This guide is targeted at all parties that could be involved including owners, clients, designers, planning supervisors, contractors, operators and employees of the aforementioned.

The dangers of damaging underground services and safe systems of work are mentioned together with a flow diagram. There are sections on training and supervision, planning, design and also where CDM applies includes duties and responsibilities. This document is comprehensive and also provides appendices on legislation and suggested advice for site personnel.

42.9 Contamination

The degree of contamination of a site is normally determined during the first phase of a geo-environmental site investigation, the phase 1 site investigation. This study would usually be carried out early in the project in order to meet the requirements of obtaining outline planning approval, as given in PPS 23 (ODPM, 2004).

Following the phase 1 investigation, if deemed necessary because of the identification of potential contaminants, a more detailed geo-environmental investigation will be required possibly together with a geotechnical site investigation. From the results of the desk studies, site work and laboratory chemical tests, a geo-environmental risk assessment will be made of the site and remediation options recommended if necessary.

In the event that elevated levels of previously unidentified contaminants are discovered on site during construction, the responsibility for dealing with these will be subject to the contractual arrangements between the contractor, consultant and client. The general principle is the 'polluter pays'; however, this is subject to commercial arrangements between the

original owners and the purchaser, and the nature of any land-use changes for the new development.

AGS collaborated with the Environmental Industries Commission (EIC) and ACE to produce a paper *The Terms upon which Contaminated Land Consultants are Employed* (AGS, EIC & ACE, 2007). This document is available from the AGS website. This guide was developed to support the commercial arrangements for studying contaminated land and more specifically to highlight the common contractual issues that should be considered by consultants. It provides guidance on key contractual issues such as the limitation of liability, collateral warranties, strict obligations and fitness for purpose, insurance clauses, indemnities, incorporation of standard forms, assignment and duration of risk.

42.10 Footnote

This chapter is generally based upon literature published by AGS and others, which has been referenced and acknowledged. These documents are generally readily available for download on the organisations' websites. At the time of writing, some of the documents and guidance notes referenced are subject to continual update and change. There have been recent changes in legislation and there are likely to be more in the future.

The recent introduction of the Eurocode 7 parts 1 and 2 for the ground-engineering industry is slowly working its way through the design and implementation processes, which will provide issues of concern as the well-known British Standards are withdrawn.

42.11 Disclaimer

Although every reasonable effort has been made to check the information and validity of the guidance provided in this chapter and the AGS documents referred to, neither the author, any individuals, members of Buro Happold, members of the Working Group, nor the AGS provide any warranty or guarantee as to the accuracy of the contents and do not accept any liability or responsibility whatsoever for any inaccuracy, misstatement or misrepresentation contained herein or for any loss, damage, expense, cost, claim or the like arising directly or indirectly from any reliance upon it, howsoever arising.

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42.12.2 Useful websites

Corporate manslaughter, Health and Safety Executive (HSE); www.hse.gov.uk/corpmanslaughter/

Corporate manslaughter, CPS (England/Wales) Guidance; www.cps.gov.uk/legal/a_to_c/corporate_manslaughter/

Association for Consultancy and Engineering (ACE); www.acenet.co.uk

Association of Geotechnical & Geo-environmental Specialists (AGS); www.ags.org.uk
AGS Client Guides; www.ags.org.uk/site/clientguides/clientguides.cfm
British Geotechnical Association (BGA); http://bga.city.ac.uk/cms
CIRIA (Construction Industry Research and Information Association); www.ciria.org
Civil Engineering Contractors Association (CECA); www.ceca.co.uk
Institution of Engineers (ICE); www.ice.org.uk
Insurance Institute of London; www.iilondon.co.uk
National Housing Building Council (NHBC); www.nhbc.co.uk
Royal Institute of British Architects (RIBA); www.architecture.com

It is recommended this chapter is read in conjunction with

- Chapter 8 *Health and safety in geotechnical engineering*
- Chapter 96 *Technical supervision of site works*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 43
Preliminary studies

Vicki Hope Arup Geotechnics, London, UK

Preliminary geotechnical studies are a key element in managing ground risk in construction. They take the form of a desk study in which relevant available documentary information about the ground at a site is identified, obtained and reviewed. A walkover site survey adds significant value. The preliminary study should be completed before scoping and specifying a ground investigation (GI), to help focus the GI and to maximise its value and relevance.

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43.1 Scope of this guidance

No guidance manual could supply the diversity of knowledge, skills and experience necessary to generate comprehensive preliminary geotechnical studies appropriate for every possible project, at every possible site. This section of the Manual, therefore, sets out to: (i) outline the types and sources of information that should typically be reviewed when writing a preliminary geotechnical study; and (ii) encourage the writers of preliminary studies to regard their reports as the all-important first step in the process of managing ground-related risks in a construction project. Focus is given to preliminary studies for sites in the UK, but much of the guidance is applicable to sites overseas.

43.2 Why do a preliminary geotechnical study?

Preliminary studies, often called desk studies, are an efficient and cost-effective way of gathering and scrutinising available information at the earliest stages of a project with the aim of understanding the ground and ground-related hazards at a site. Within the construction industry, project delays and cost overruns can frequently be linked to encountering 'unforeseen ground conditions' when construction starts on site (Chapter 7 *Geotechnical risks and their context for the whole project*). A well-planned, well-executed preliminary study can assist greatly in reducing – although it can never wholly eliminate – the likelihood of meeting unexpected problems in the ground during construction.

Undertaking a preliminary study is a requirement of all relevant codes in the UK:

- British Standard BS 5930: Clauses 6.1.1 and 6.2 (BSI, 1999);
- Eurocode 7 BS EN 1997–1: Clauses 3.1 and 3.2.2 (BSI, 2004);
- National House Building Council Standards Chapter 4.1 (NHBC, 2011).

The preliminary study should be viewed as the essential first link in the geotechnical project risk management chain (see **Figure 43.1**). The chain progresses through each stage of the geotechnical engineering of a construction project (Chapter 42 *Roles and responsibilities*). In essence:

- The preliminary study identifies likely ground-related hazards.
- The ground investigation explores the hazards and their associated risk (see Box 43.1).
- The geotechnical design reassesses the ground risks and mitigates the associated hazards to appropriate levels.
- Any remaining (known) ground-related hazards that are not mitigated at the design stage can, in liaison with the project management team, be communicated to the construction team via specifications and other contract documents, to help ensure that these issues do not come as a surprise on site during construction.

Box 43.1 Hazards and risks are not the same

A **hazard** is a source of potential harm. The **risk** associated with a hazard is the likelihood of that harm occurring, factored by its impact if it were to occur. For example, flooding is a hazard. The risk would depend on how prone an area is to flooding, and the degree of damage and loss that would be caused by a flood event.

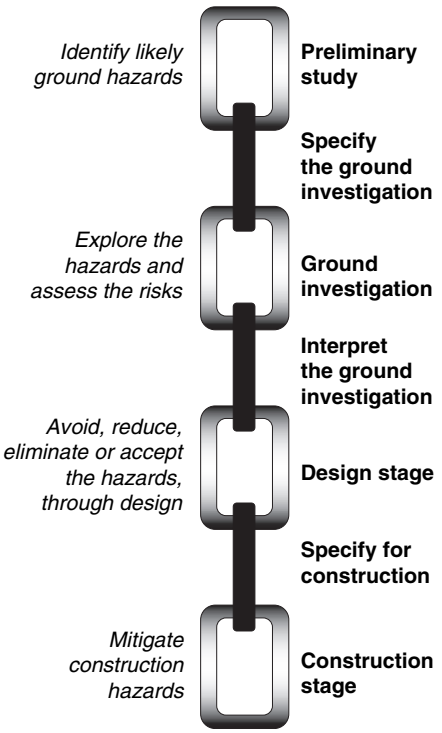


Figure 43.1 The geotechnical risk management chain

43.3 What goes into a preliminary geotechnical study?

The factual core of a preliminary geotechnical study typically encompasses:

- *site details*, including its location, address, grid reference, boundaries, topography, present use, proposed use, access routes, land ownership, neighbouring land usage, etc.;
- *site history*, including an interpretative review of historical maps, ideally extending back to rural fields; aerial photographs and satellite imagery; prior site usage; archaeological potential; listed buildings and scheduled monuments; water courses; tunnels; mine workings (Chapter 41 *Man-made hazards and obstructions*); possible unstable ground (as defined in PPG14, 1990); underground infrastructure; flood records; buried obstructions; old foundations; underground services and utilities; possible sources of contamination from current or previous site activities, and initial assessment of these; historical changes in topography; presence of trees, etc.;
- *site walkover*, including observations of the site and its vicinity;
- *site geology*, including an interpretative review of all available geological maps and memoirs, including old map series; well records and borehole logs; data from previous ground investigations at the site or in its vicinity; an assessment of the likely site stratigraphy; groundwater regime and hydrogeology; seismicity, if relevant, etc.;

plus a variety of project-specific and site-specific variations on these core themes.

Guidance on the archaeological assessment of sites is published in the *Standard and Guidance for Archaeological Desk-Based Assessment* by the Institute for Archaeologists (2008), and this is not repeated here. In relation to site contamination, a preliminary study is termed a Phase 1 study, and a ground investigation is a Phase 2 study. These must follow prescribed legislative requirements. Guidance on the requirements for Phase 1 studies is presented in British Standard BS10175 *Investigation of Potentially Contaminated Sites* (BSI, 2011); Environment Agency CLR11 *Model Procedures for the Management of Contaminated Land* (Environment Agency, 2004); and in the *Institution of Civil Engineers' Design & Practice Guide Contaminated Land, Investigation, Assessment and Remediation* (Strange and Langdon, 2007). Chapter 48 *Geo-environmental testing* addresses Phase 2 studies.

The preliminary geotechnical study report should also cover the following interpretative elements, each of which demands some knowledge of the proposed development, as well as knowledge of the site itself:

- *Recommendations for the ground investigation:* outlining the scale and scope of the proposed ground investigation, and highlighting any special site-specific or project-specific aspects identified in the desk study that should be addressed in the ground investigation.
- *Details of ground-related site constraints:* cataloguing the identified site-specific factors that may affect ground investigation works, foundation options and foundation construction.
- *List of ground-related hazards:* describing and prioritising the identified ground hazards (see Box 43.2 for some examples), and outlining how these could be investigated further and how they could be mitigated. Ground hazards can usefully be treated in three categories: topographic, geological, and man-made. The format of the listing should give priority and emphasis to the site-specific and project-specific geohazards that are particular to the site: these should not be lost or overwhelmed by a long list of generic issues that could apply at any site.

The inclusion of a list of ground-related hazards in a preliminary geotechnical study benefits both the client and the project team because it:

- helps convert the preliminary study findings into risk management information that is of direct use from the earliest stages of a project, by identifying ground-related factors that may affect costs and the programme;
- translates the preliminary study findings into a practical format that is readily understood by non-geotechnical colleagues and clients;
- acknowledges the importance of risk-based methods in contemporary project planning, and integrates the preliminary geotechnical study within that process from the outset of the project;
- brings considerable value to a project, yet makes use of material that the geotechnical team has already gathered and should already have thought about.

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Box 43.2 A partial list of hazards: some ideas to consider…

Topography
slope stability issues
site access
possible GI drill rig locations
potential for flooding

Groundwater
need to dewater? effects?
rising groundwater
tidal variations
potential for aquifer contamination
discharge of pumped groundwater

Voids/soft spots
mining
karsts
backfilled quarries
backfilled basements
old wells

Trees
present on site? – walkover
recently removed? – archive photos
clay soils: shrink/swell potential
species, height and maturity
removal *versus* retention

Buried features
operational and disused tunnels
services and utilities
foundations, basements, tanks
UXO (unexploded ordnance)

Heritage issues
listed buildings
archaeological planning constraints
need for a watching brief during GI
impact on programme of finds
Scheduled Ancient Monuments

Neighbours
proximity to existing structures
sensitivity of existing structures
ground movements at excavations
noise and vibration
need for party wall agreement?

43.4 Who should write a preliminary geotechnical study?

Preliminary geotechnical studies should be written by suitably experienced geotechnical engineers or engineering geologists. One can speculate that if an engineer and a geologist were set to work in isolation of each other on desk studies about the same site, they would probably generate very differently focussed reports. In practice, for all sites, a combination of both technical perspectives is invaluable: each specialist can gain a deeper insight of a site by liaising and collaborating with the other. The balance in the input needed from each specialist will depend largely on the site's location and geology: a pipeline in a remote mountainous region would typically benefit most from an engineering geologist, whereas a building in a city on an alluvial plain would probably benefit most from a geotechnical engineer.

Appropriate knowledge and experience are essential in order to understand fully the significance of the information gathered. The responsibility for a preliminary study, therefore, should not be assigned to the most junior member of the team, although, if properly supervised by an experienced senior colleague, preliminary studies are valuable training grounds for junior engineers and engineering geologists.

Beyond knowledge and experience, useful characteristics for writers of preliminary studies include a high level of observant curiosity, a persistent and creative attitude to seeking data, and a methodical and thorough approach to reviewing data. These attributes are needed because a preliminary study often must draw together disparate – and sometimes incomplete – data to form as complete and insightful a picture as possible of the ground at the site of interest. That cannot be achieved in a routine or formulaic way: it requires inquisitiveness underpinned by a systematic approach to obtaining and assembling relevant information.

43.5 Who should read a preliminary study report?

The report should be circulated to the client and the design team, including the architectural team. If a preliminary study has been undertaken thoroughly and effectively, the resulting report should:

- be essential reading for whoever is tasked with scoping and specifying the ground investigation;
- probably contain some items of useful information that were not previously known to the rest of the design team, and which may well influence their thinking;
- summarise the currently available ground information in one convenient volume, including ground-related hazards;
- engage and interest the client, especially about the history of their property.

The contents, or a summary of the contents, of a preliminary geotechnical study should usually be made available to the construction team (the contractor), alongside the ground investigation factual reports. Any concerns about releasing an interpretative study to a third party can be dealt with via a suitably phrased third-party caveat in the text of the report. In terms of controlling ground risk in a project, it is beneficial that the construction team are as fully informed as possible about a site's history and the anticipated ground conditions at the site. They can then assess the likelihood and potential impact of these factors on their site works.

43.6 How to get started: sources of information in the UK

In the UK, the following are useful starting points for gathering data from which to develop a preliminary geotechnical study.

This list is not exhaustive, and new sources of information – especially online sources – become available each year.

- Geological maps and cross-sections published by the British Geological Survey (BGS, www.bgs.ac.uk) at 1:10 000 and 1:50 000 metric scale. Both drift and solid maps should be reviewed, along with the relevant geological memoir. Older maps can also be highly informative, despite changes in the names of geological formations. Additional information can be gained from mineral extraction maps.
- The GeoRecords service at the National Geoscience Data Centre (www.bgs.ac.uk) can be used to obtain old well records and borehole logs, from the archive held at the BGS. The archive can be searched via an online map.
- Ordnance Survey (OS) maps (www.ordnancesurvey.co.uk), dating from the mid-nineteenth century to the present, can assist greatly in tracking historical changes at a site. Small-scale OS maps (1:1250 and 1:2500 at metric scale) show local detail, including street names (see Box 43.3) and the outlines of individual buildings. OS maps include topographic data, expressed as spot heights and contours, relative to sea level (see Box 43.4).

Box 43.3 Place names

Place names can provide valuable information and insight into the history of a site, its topography and even its hydrogeology. An unusual place name should always trigger an enquiry into the origin of the name, and an assessment of its implications for the proposed works at the site. For example, a street called 'Flask Walk' in Hampstead is close to Well Lane and Well Walk: all three derived their names from the iron-rich mineral waters that were sourced and bottled there from the seventeenth century onwards. Many older place names, especially Anglo-Saxon names, provide information about the natural landscape. Some examples of the many Celtic, Saxon and Viking root words used within place names in the UK include:

- allt: the side of a hill;
- beck, bourne, burn: a stream;
- glan: the bank of a river;
- gill: a valley;
- hamps: a stream that is dry in summer;
- ings: a meadow or marsh;
- keld, kelda: a spring;
- mere: a pool or lake;
- moss: a marsh, bog or swamp;
- pant: a hollow;
- slack: a stream within a valley.

Box 43.4 Liverpool and Newlyn

When using early Ordnance Survey maps or old well logs dating from before the 1920s, be aware that the UK height datum system changed during the 1920s. From 1844, the height elevation datum was at Victoria Dock in Liverpool. After 1921, the UK national height datum system was based on tidal measurements at Newlyn in Cornwall. The difference in heights between the Liverpool and Newlyn systems is not large, but it is just enough to cause confusion unless it is taken into account. The height conversion factor between the two systems differs by differing amounts across the country. The Ordnance Survey has an online conversion calculator at www.ordnancesurvey.co.uk.

- Historical maps were produced by a variety of map-makers. Some maps were semi-pictorial views, but others were more rigorously cartographic, notably those of Roque, Cary, Horwood and Stanford. Copies of old maps are available from online suppliers, and some have been published as books.
- From the late 1830s, church tithe maps showed land parcel boundaries and building outlines, but the maps were of variable quality. Most local history centres hold the tithe maps for their area.
- Maps made for fire-insurance purposes by Charles E. Goad Ltd from the mid-1880s to 1970 show the commercial districts of many UK towns and cities. At a scale of 1:480, they show a building-by-building mapping, with details such as industrial use, construction materials, number of storeys and basements. The Map Library at the British Library (www.bl.uk) has an extensive collection of UK Goad fire maps. Scanned Goad maps are also available commercially from online map suppliers.
- The Coal Authority Mining Reports Office (www.coal.gov.uk) can be used to obtain details of documented mining operations, and mining and brine-extraction ground stability reports (see Chapter 41 *Man-made hazards and obstructions*). Its website includes a gazetteer indicating the areas of Great Britain where mining-related ground stability is likely to be an issue.
- Topographic data can be obtained from several sources, including the OS Land-Form Profile Service (www.ordnancesurvey.co.uk), which covers the UK. The data are presented as x,y,z values (derived from photogrammetry) with an accuracy of approximately ±1 m in plan and ±1.8 m vertically.
- Documentary searches of underground services and utilities plans can be commissioned from commercial search firms (see Box 43.5). As well as shallow utility services, the search should include transportation and utility tunnels.

Box 43.5 Utility searches by commercial agencies

In the UK, several commercial organisations offer 'one-stop shops' for obtaining, collating and reporting documentary searches of utility plans of statutory service undertakers (gas, water, electricity, telecoms, etc.). A request for a documentary utility search should state clearly the footprint and the depth of the proposed development. The searches can take several weeks to complete, as they rely on receiving responses from third parties. Be aware that the documentary search results will show only the approximate location of the buried utilities – not exact positions – and they will not show buried utilities within a private property boundary or within buildings.

- The possible presence of disused historic tunnels should not be overlooked (for example the Post Office railway in London or Williamson's tunnels in Liverpool). Identifying such infrastructure often relies upon local knowledge, or access to it.
- Public libraries usually hold a collection of local history books, including out-of-print publications that may not be readily available elsewhere.
- Certain below-ground infrastructure such as emergency bunkers and strategic telecoms tunnels are, by their nature, not widely advertised. Some disused facilities have been reported by enthusiastic amateurs (such as at Subterranea Britannica www.subbrit.org.uk). For operational facilities, enquiries should be made to the

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relevant bodies to ascertain where any 'hidden' infrastructure is in relation to the proposed development.

- The archives at the Institution of Civil Engineers (www.ice.org.uk) are a useful resource for obtaining information and records of historic engineering structures (Chrimes, 2006).
- Details of grade-listed buildings and Scheduled Ancient Monuments in England are available from English Heritage (www.english-heritage.org.uk) and its searchable online archive of images of grade-listed structures (www.imagesofengland.org.uk).
- Historic aerial photography of England from the 1940s onwards, including wartime images showing bomb damage, can be obtained from the National Monuments Record Centre (English Heritage). The NMR archive in Swindon can be visited in person. Alternatively, a 'priority search cover' can be ordered, which lists the date, location, scale, stereo-pairing and verticality of the aerial images. Detailed interpretation of aerial photographs is a specialist skill, as is the photogrammetric analysis of stereo-pairs of images to quantify dimensions and levels from aerial imagery.
- Comparable services in Wales and Scotland are provided by, respectively, the Royal Commission on the Ancient and Historical Monuments of Wales (www.rcahmw.gov.uk) and the Royal Commission on the Ancient and Historical Monuments of Scotland (www.rcahms.gov.uk).
- Information about wartime bomb damage and unexploded ordnance (UXO) can be obtained from several sources. For London, hand-coloured OS maps were used to record the degree of bomb damage; these are available in book format. The Department of Communities and Local Government (www.communities.gov.uk) maintains a list of known, abandoned UXO. Information about known UXO at a site can be obtained from the department, by emailing details of the site location (address, grid reference, map). The likelihood of UXO being present at a site is partly, but only partly, a function of the density of bombing that occurred in that district. A readily accessible summary of bombing densities across the UK is available at www.zetica.com.

43.7 Using the internet

The internet is a powerful – but not an all-powerful – tool. Internet searches can offer a quick and low-cost way to find information about a site. Maps, satellite imagery (for example at www.google.co.uk), reports, journal papers and images can often be found online and downloaded almost immediately. At its best, the internet can provide, or lead to, useful information that would otherwise be difficult, time-consuming and costly to obtain. At its worst, however, the internet provides low-grade, unverified and sometimes erroneous information.

When using the internet as part of a preliminary study, be conscious that:

- Data available on the internet are often incomplete, undated and unreferenced – and, hence, potentially unreliable. Online data should be checked independently where possible. Be aware that some websites simply recycle material from other online sites, although the material may have been in error in the original source.
- The successful exploitation of internet search engines relies on selecting appropriate and well-chosen keywords, and then refining and re-focussing the search as it progresses.
- Material pre-dating the internet may not be readily revealed by an internet search. Traditional desk study research methods – and some legwork – may still be needed. For example, local history centres and archives nowadays often list their catalogues online, but a visit in person to the collection may be necessary in order to see the material. Be aware that an online search will only reveal documents that have been catalogued using the same keywords used in a search. Discussion with a knowledgeable curator may uncover other relevant material, which may not have been revealed by a simple keyword search.

43.8 The site walkover survey

If permitted and if possible, a site walkover survey should be undertaken. Ideally, the walkover should be done about two-thirds of the way through the period assigned for the preliminary study, by which time enough background documentary information will have been gathered to inform what is seen on site, and sufficient time will still be available to follow up any unexpected observations and to integrate the walkover results into the overall site interpretation.

The planning of a site walkover visit should include due consideration of safety, and an appropriate risk and mitigation assessment should be carried out beforehand. Potential hazards are site dependent, and may range from aggressive livestock in rural areas to unfriendly locals in urban areas. In most cases, at least two people should undertake a walkover visit together, to avoid the hazards of lone working.

What to look for during a walkover survey will depend on the context: walkover surveys at a hill farm and at a city street would be substantially different. An experienced practitioner will have a mental checklist of basic items to look for (such as utility service covers; repairs in roads and pavements; drains and ditches; marshy ground; uneven ground; cracks in structures; geomorphology; rock outcrops; slopes and slope instability; evidence of cut and fill; species of trees; types of buildings; presence of basements; possible sources of contaminants; access for ground investigation drill rigs; likely locations for boreholes; overhead constraints) plus a long list of project-specific and site-specific items.

On site, all relevant features should be noted and photographed. Photocopies of site maps or satellite images can be annotated with observations and comments. For large or rural sites, a hand-held GPS to record coordinates of features can be helpful. Photography is an essential tool. Plenty of photographs should be taken: not all need be used in the report, but a photograph can often help answer an unforeseen query when reviewing data back at the office. The preliminary study report should include a photographic key plan showing the location and direction of each photograph reproduced in the report.

The walkover survey should not be limited to the site itself, but should also look beyond the site boundaries. If access onto neighbouring land is not possible, then a visual assessment can usually be made across the boundary line.

It can be useful to talk to local people, as they usually know their area and its history. However, it is important to

verify anecdotal recollections independently, and to indicate clearly any anecdotal information that is included in the report. When undertaking a walkover survey, especially when talking to locals, a degree of circumspection may be appropriate. Before undertaking a walkover visit, one should check how much of the proposed project is public knowledge, and whether the project is considered sensitive either locally or commercially, particularly if planning consent has not yet been granted.

43.9 Writing the report

When writing a preliminary report, much inspiration can be gained by looking at good examples of previous reports for other sites, but these should never be relied upon wholly. Each site is unique. Factors that may not have been relevant at another site, and which were, therefore, disregarded in its report, may be critical at the site of interest.

As with all technical report writing, the preliminary study report should be written clearly and carefully. Useful guidance on writing geotechnical reports is presented in three guides produced by the Association of Geotechnical & Geoenvironmental Specialists (AGS) and available at www.ags.org.uk:

- *Guide to Good Practice in Writing Ground Reports*;
- *Management of Risk Associated with the Preparation of Ground Reports*;
- *Guidelines for the Preparation of the Ground Report*.

The AGS guides emphasise the importance of taking care when writing a technical report because, for example, a negligent and damaging error could potentially lead to claims against the organisation that issued the report.

The structure of the report will be partly dictated by the material that has been reviewed. The list of items in 'What goes into a preliminary study?' (section 43.3) can be used as a basis for the overall report structure, and as headings for sections and sub-sections.

As in any professional technical report, it is important to distinguish between factual and interpretative elements. The report should include a clear statement of what sources of information have been reviewed in preparing the report. Equally, if any key items have *not* been reviewed (perhaps a document that is known to exist but has not been received in time, or has been withheld by a third party), this should be stated in the report, as it affects the scope and completeness of the report.

The source of all material used in the report should be clearly stated, even if the material is not copyrighted (see Box 43.6). The aim should be to provide readers with enough information to enable them to find the same material again. As a minimum, this should include the source and date of the material. By doing this, a signal is sent to the reader about the provenance and, hence, the likely reliability of the information upon which the preliminary study has been based.

Box 43.6 Copyright

Care is needed when using material, especially images, taken from copyrighted sources. Preliminary studies are produced for clients, and are, thus, commercial reports. Under UK copyright law, the consent of the copyright owner should be sought if an extract of their work, or an adaptation of their work, is to be included within a commercial report. Also, the owner of the copyrighted material should be clearly stated in the report. For fuller details of copyright issues and UK copyright law, check the UK Intellectual Property Office's website (www.ipo.gov.uk).

If reproducing Ordnance Survey (OS) maps, these should be clearly labelled as being Crown copyright material. The OS operates a licensing system for businesses wanting to reproduce OS maps in professional reports. A copy of the OS's *Paper Map Copying Licence* can be downloaded from the OS website www.ordnancesurvey.co.uk.

43.10 Summary

The preliminary geotechnical report is the essential first link in the chain of geotechnical project risk management (see **Figure 43.1**). Site information, ground models, geohazards, uncertainties and ground-related risks identified in the preliminary study will form the basis for the next stages of the project, and may potentially influence the direction of the project and the engineering solutions adopted.

A well-planned, well-executed preliminary study that is followed through with a suitable ground investigation can help significantly in reducing the likelihood of unexpectedly encountering problematic ground conditions during construction, although it cannot reduce that likelihood to zero.

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43.11.1 Useful websites

Association of Geotechnical and Geoenvironmental Specialists (AGS); www.ags.org.uk
British Geological Society (BGS); www.bgs.ac.uk
British Library; www.bl.uk
British Standards Institution (BSI); http://shop.bsigroup.com/
Coal Authority; www.coal.gov.uk
Department of Communities and Local Government, UK; www.communities.gov.uk
English Heritage; www.english-heritage.org.uk
English Heritage online archive of grade-listed structures; www.imagesofengland.org.uk
Google (search engine); www.google.co.uk
Institution of Civil Engineers (ICE); www.ice.org.uk
National House Building Council (NHBC); www.nhbc.co.uk
Ordnance Survey (OS), including the Land-Form Profile Service; www.ordnancesurvey.co.uk
Royal Commission on the Ancient and Historical Monuments of Wales; www.rcahmw.gov.uk
Royal Commission on the Ancient and Historical Monuments of Scotland; www.rcahms.gov.uk
Subterranea Britannia; www.subbrit.org.uk
Summary of bombing densities; www.zetica.com
UK Intellectual Property Office; www.ipo.gov.uk

It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

Chapter 44

Planning, procurement and management

Tim Chapman Arup, London, UK
Alister Harwood Balfour Beatty Major Civil Engineering, Redhill,UK

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Coherent planning is required for every ground investigation in order to ensure that it is carried out simply and cost-effectively and produces all the requisite information. Poorly planned ground investigations cause significant problems in terms of damage to neighbourly relations and damage to buried utilities, the latter exposing the site operators to severe safety risks. A well-planned two-phase campaign can offer advantages for longer and more complex investigations, as information discovered in the first phase can be further investigated in the second. A clear form of contract and specifications reduces the risk of disputes. Regular review of the information recovered is the ultimate success of a grand investigation campaign.
A well-planned campaign is essential to anticipate future likely changes in the scheme. It saves construction cost and programme by revealing important site features at an early stage which can then be planned for.

44.1 Overview

The success of a ground investigation depends on the extent of prior planning, the clarity of the procurement processes used and the care with which it is managed. These factors become even more important for larger ground investigations. The trend towards frameworks and large infrastructure projects means that some ground investigations are now becoming very large indeed.

A ground investigation can be considered to have failed if:

- The process does not gather all the data intended, or if a party subsequently realises that more or other data should have been gathered at the same time, or if the ground investigation reveals aspects of the site which should have been considered before. Note that ground investigations can never discover everything about a site, and the possibility of surprises should never be discounted. The discovery of legitimately new information in the investigation should be regarded as a success rather than a failure.
- The process results in a significant misunderstanding or dispute involving the client, the contractor, a landowner, an authority or some other body. These aspects should have been considered beforehand and the contract documents should have set out clear rules to avoid such an occurrence.

The standard guidance documents for ground investigation are those of the *Site Investigation in Construction* series, produced by the Site Investigation Steering Group, introduced in Chapter 42: *Roles and responsibilities.*

44.1.1 Key messages in this chapter

- Ground investigation is done most efficiently if properly planned in advance.
- Planning a ground investigation is firmly based on logic, but balancing the many conflicting constraints and priorities sometimes makes it akin to an art-form. The planner must not lose sight of the main objective, which is to gather sufficient data for the design of the planned development and minimise ground risk. The various constraints imposed on the ground investigation must not compromise the final information to any unacceptable level.
- Safety is a key objective. There may in some circumstances be some conflict between the safety of the site investigation operatives and the need to gather information to minimise the risk to the workforce on the main development. In those circumstances, careful consideration will be needed. Any residual risk in the site investigation must be fully communicated to all those going on site and every effort must be made to mitigate the remaining risks to tolerable levels.
- Ground investigations often take longer than many people would wish. Completion of the site works is not the end of the story. Sufficient time should be allowed in the overall project programme for timely completion of the ground investigation before the information is needed for design.
- Choosing the right people to undertake the various geotechnical activities – design and investigatory – depends on the relative strengths of the various project participants. But it is important to facilitate as far as possible a coherent geotechnical design process – disjointed steps in that process are a major source of project risk.
- As for any construction activity, a clear specification and contract with equitable terms should reduce the risk of disputes.
- Locating unforeseen conditions in a ground investigation should be seen as a success and not a failure, even if the ground investigation is pushed over budget.

44.2 Planning the ground investigation

44.2.1 Developing a coherent campaign

44.2.1.1 Meeting the needs of clients, designers and contractors

Planning for a ground investigation can only take place after an understanding has been gained of the likely geology

(see Chapter 40 *The ground as a hazard*) and the desk study has identified the key uncertainties and hazards on the site (see Chapter 43 *Preliminary studies*). Rudolf Glossop said in his 1968 Rankine lecture: 'If you do not know what you should be looking for in a site investigation, you are not likely to find much of value' (Glossop, 1968).

Planning of a ground investigation should target the particular development scheme proposed and it is important that all aspects of the scheme and its implications are known, for instance:

- a six-storey building over a two-level basement; perimeter retaining wall needed on north side; foundations may be piled to 30 m depth, large surface car park to west.

Thus if a tall tower is added or if the building location changes, further supplementary ground investigation will be needed. The effects of the location of buried utilities, highways works and the necessary traffic management on the investigation also need to be considered early, as these issues often require lengthy negotiation and agreement (see Chapters 41 *Man-made hazards and obstructions* and 42 *Roles and responsibilities*). Leaving these aspects until too late, for instance delegating them to the contractor, who cannot start the negotiations until appointed, can lead to delay and extra costs.

The development of a coherent campaign of ground investigation requires considerable planning, particularly where third-party property owners believe they will be affected by the site works. Site occupants and immediate neighbours will be directly affected. Generally the ground investigation is the first physical sign of the new development that they experience, and so any opposition or concern to the development may be focused on the ground investigation fieldwork campaign. Thus special care at this stage is needed, as a carelessly executed ground investigation that causes much inconvenience can sour neighbourly relations for some time and complicate the main construction process,

Often, ground investigations are carried out while existing buildings on the site remain fully occupied and operational. This is because the site is only usually vacated in preparation for demolition; few clients will commit to demolishing while they can continue to receive rent and will delay evicting occupants until they have let the contract to build the new structure or building. The demolition cannot be let until the structural design has been completed, which relies on the completed ground investigation. The ground investigation therefore usually has to fit around occupants and operational business. This can be a significant cause of conflict unless the planning of the site investigation ensures that occupants are not unduly disturbed, operation of the facilities is not unduly impaired and safety of the normal site occupants is not compromised. These aspects are discussed in more detail in section 44.2.3.

Often formal permissions or licences may be needed in order to undertake the investigation, especially when access is required across neighbouring land, or where parts of the investigation have to be undertaken off the site, for instance on the public highway. The programme for the investigation needs to allow time to make all agreements, obtain all necessary licences and follow any restrictions legitimately placed on the operations by relevant parties.

Ground investigations next to live railways tend to be the most tightly regulated, because of the severe consequences if a rig overturns or a slope is destabilised next to the operational railway. The person planning the investigation needs to be aware of all specific rail-related regulations and procedures that may affect the work, including training of site workers and appointment of railway look-outs. Ground investigations next to water should receive similar attention in planning.

The Party Wall Act etc. 1996 often governs the interface with neighbours when any form of construction work is being undertaken. A very thorough explanatory leaflet is available from the Department for Communities and Local Government, published in March 2002.

Section 6 of this Act applies where a landowner wants to excavate, or excavate and construct foundations for a new building or structure, within:

- 3 m of a neighbouring owner's building or structure, where that work will go deeper than the neighbour's foundations; or
- 6 m of a neighbouring owner's building or structure, where that work will cut a line drawn downwards at 45° from the bottom of the neighbour's foundations (see **Figure 44.1**).

The limit for excavations is sometimes taken to mean excavations for ground investigations, although it is not thought that that was the intention of the Act. Unfortunately, excavations for ground investigation are not explicitly excluded under the Act, so the Act can sometimes be deemed to apply even to small boreholes or limited trial pits. Where the Party Wall Act may apply, it is therefore prudent to obtain the requisite permissions, especially if neighbourly relations are not good.

44.2.2 Considering all aspects of the site and all users for the data

Ideally the ground investigation is planned to address several needs at the same time, for instance:

- Carrying out geotechnical design of foundations and sub-structures for the new development, including all significant ancillary works such as retaining walls and car parks.
- Producing information for the design of temporary construction works, such as excavation slope stability or dewatering during construction, as otherwise tendering contractors will have no other source for the data and will make conservative assumptions that will push up the construction price.
- Gaining knowledge of current foundations for existing buildings in order that any necessary structural changes can be designed to allow the new development to proceed.
- Understanding the state of contamination of ground or groundwater or establishing levels of deleterious or harmful substances in the ground.

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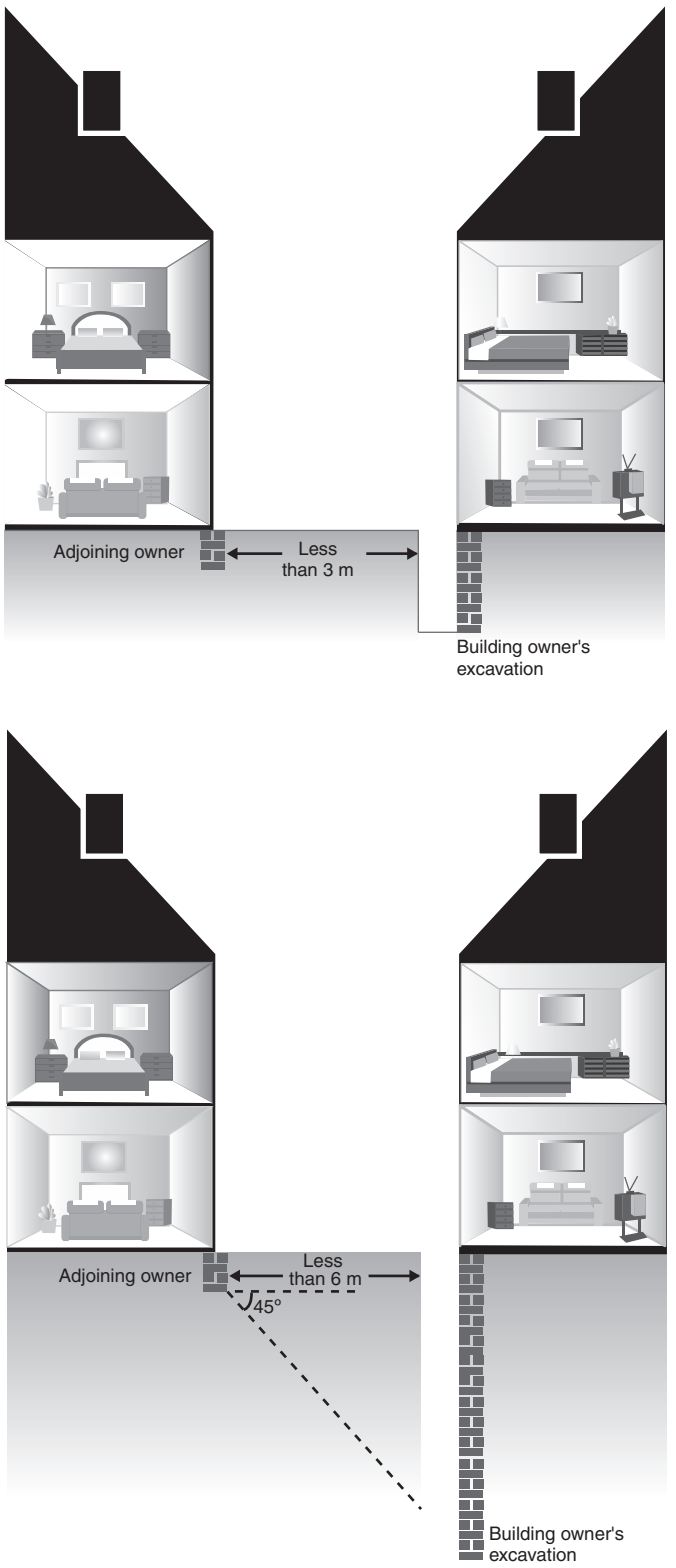


Figure 44.1 Application of Party Wall Act
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- Determining underground water flows under or around the site.
- Collecting data on existing buried services, sometimes just to ensure that they are avoided by the ground investigation but often to ensure their locations are known for main construction so that they can be avoided, diverted or protected.
- Examining the ground for any evidence of surviving archaeology and coordinating the campaign with any necessary archaeological excavations.

In such cases the site operations need to be organised and phased so that all relevant consultants have the opportunity to input. Their input is needed in planning to integrate all requirements, and during the site works to satisfy them that their specifications are being followed and the right information is being obtained in a holistic way, efficiently and minimising effects on people affected by the work. Sometimes these operations are carried out in a disjointed way with little coordination – such an approach usually leads to fewer data being obtained at greater cost and increased disruption. Sometimes aspects are neglected, such as those for temporary works – these omissions can prove exceptionally costly, either due to lack of appropriate information for tendering contractors or failure to identify key features until they are discovered during construction.

On complex sites where many of these requirements are not straightforward, one person should be responsible for ensuring that the campaign is executed coherently. That person may be an engineer's representative for the employer or a well-briefed agent for the contractor, depending on which approach is most appropriate.

A two-phase ground investigation campaign is recognised as being a valuable way to investigate the ground, particularly on sites with greater ground uncertainty, because it allows the second phase to build on the results of the first one, to refine what is found and to respond to new information. Generally the first phase would be a sparse but deep investigation, to detect the major geological formations and identify those that need greatest focus in the second phase. The second phase can then refine the results of the first phase, filling in details and providing appropriate design parameters for each major stratum. This approach is most valuable when there is significant uncertainty about the geology beneath the site, which can at least be partially clarified in the first phase. On major infrastructure projects, there may be several phases of investigation. Eurocode 7, Part 2 (BS EN1997-2:2007) (BSI, 2007) also supports a two-phase approach for larger and more complex investigations.

Major constraints to the execution of the ground investigation should be determined, e.g. existing tunnels beneath the site which need to be avoided, so that an optimal campaign can be planned.

Sometimes, the site constraints prevent a coherent campaign being planned to properly investigate the hazards. In that case, two options are possible:

- Defer the troublesome activities until a later time when they become possible, for instance by waiting for a window between the site being vacated and the start of demolition. This means that

important information will not be gathered until relatively late in the design process. This may not matter much provided the first phase of investigation gathers enough information for design, but perhaps omits information relevant to construction planning; however, it is not appropriate to delay the collection of information that may lead to a misunderstanding of the ground conditions in design.

- Accept the increased risks and make the development less dependent on them, e.g. adopt a more conservative design basis which can accommodate greater uncertainty. This may have a higher direct cost, but may be expedient in terms of a quicker programme and hence be more economical in terms of overall costs to the client.

An example of the first approach would be on a site where the variation in level of a particular stratum interface is in doubt. The first phase would confirm the general level and the design parameters. Proving the level of an interface sufficiently for construction, e.g. choice of temporary casing length for piling, need only be done immediately prior to letting the construction contract ,so that accurate prices can be obtained from the tendering contractors.

On some sites, and particularly those smaller sites where environmental concerns are greater, sometimes an initial geotechnical investigation is merged into a combined 'Phase 1' geotechnical and geo-environmental investigation as explained in Chapter 42 *Roles and responsibilities*. Such investigations sometimes fail to collect important geotechnical information such as adequate depth of investigation, proper soil descriptions, water level measurements and strength determinations. Failing to collect these data at an opportune time is a missed opportunity that increases project risk – every effort should be made to ensure that a Phase 1 environmental investigation also provides meaningful geotechnical information.

Providing easy access to all the data produced can be a challenge, particularly for large ground investigations. Hence consideration of how the data will be managed is important. For larger investigations, incorporation of the results into a GIS system can pay dividends in the subsequent design phases.

44.2.3 Timing, programme and space: balancing the constraints of site and project

Knowledge is more useful the earlier it can be obtained. However, a ground investigation usually has to be delayed until:

- The scheme it is to address has crystallised into what is likely to be built, at least in broad terms – doing it too early may mean a supplementary ground investigation is required later.
- The client is sufficiently committed to the scheme to invest significant resources in it – although ground investigations are a very small proportion of the total project cost, they are often the first construction-related activity on site and so represent the client's first large payment to a contractor. Hence this hurdle is sometimes a useful barometer for a client's commitment to a project.
- The start of ground investigations does not alarm current site occupants or neighbours, who in some circumstances may be opposed to the development objectives and plans.

A way to reduce these effects and to allow earlier collection of ground investigation data is to use a two-phase approach to the ground investigation as introduced in section 44.2.2:

- First phase to check key hazards, confirm any variation in the stratigraphy across the site and obtain outline design parameters.
- Second phase to provide more complete coverage and to confirm the applicability of design parameters.

The two-phase approach is especially valuable for sites where the ground conditions are not well known at the outset, as it permits some understanding to be gained by the ground investigation designer between collection of the data used to formulate the basic ground model, and filling in the detailed information needed for each layer.

An early ground investigation will normally take place while the site is still occupied, so sensitivity is needed to avoid causing excessive disruption to occupants, users and neighbours. The main forms of disruption are:

- Occupation of space – in addition to the space needed at each exploration point, the ground investigation contractor will need space for stores, spare drilling equipment (for instance, extra lengths of casing), waste skips, offices and parking.
- Restriction of access or impairment of site operations – some of the exploration positions may impede main access routes for vehicles and pedestrians or block loading bays.
- Noise and vibration – the exploration is often the first major construction activity for the new development, and so noise and vibration could be a cause of concern to neighbours and residents. On very restricted sites, boreholes or trial pits will inevitably encounter buried reinforced concrete or masonry, which will have to be broken out by pneumatic tools – limiting their use to particular times can do much to improve neighbourly relations but will increase ground investigation cost and programme. BS 5228: Part 2 (*Code of Practice for Noise and Vibration Control on Construction and Open Sites – Vibration*) (BSI, 2009) and CIRIA TN142, *Ground-Borne Vibrations arising from Piling* (Head and Jardine, 1992), although primarily intended for piling operations, provide useful guidance on controlling noise and vibration control and appropriate limits for ground investigation sites.
- Disruption to services - potentially, the investigation can lead to buried services being damaged or cut, which will affect the site occupants and may cause financial losses due to business interruption. Ideally, services these will have been identified and deactivated when planned works are in close proximity to them.
- Impairment of safety by extra vehicular movements or lack of care.
- Level of reinstatement – whether tarmac is fully reinstated; whether standpipes are made flush with the ground or raised etc. These aspects are best agreed in advance so that the contractor's cost can allow for the level that will be required, and to prevent higher costs being sought at a late stage.

A ground investigation can be modified to use less space, but cost and programme will consequently have to increase, and the technical output may be compromised. Examples include:

- A normal cable percussion ('shell and auger') boring rig needs a working area of at least 7 m × 3 m and direct vehicular access.

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A man-handleable portable rig can be used in confined areas, but is restricted in borehole diameter, depth and numbers of 'strings' of casing that can be inserted.

- Machine-dug trial pits may need a space of 3 m square; hand-dug trial pits can be done in a smaller area, but need to be shored and are significantly slower with increased concerns for safety. They may still need to break through buried reinforced concrete, so may not be quieter, and if slower may prolong the agony of neighbours.

A typical programme for the procurement and execution of a ground investigation is given in **Figure 44.2**. The programme is often significantly longer than clients expect, so it is worthwhile explaining the rationale for each of the programme elements in order that an instruction to proceed is given appropriately early and results can be available when needed.

44.2.4 Parameters and information needed

44.2.4.1 Information and requirements of different projects

The parameters collected in a ground investigation depend on the nature of the site and what one plans to build on it. A particularly hazardous site or a complex development will obviously need a more elaborate analysis to address the key risks in the most economical way. The more detailed the analysis, the more parameters usually need to be determined for the results of the analysis to be meaningful. There needs to be a degree of iteration around the *scheme–analysis–parameters–investigation* loop to ensure that the ground investigation is carefully targeted. A geotechnical adviser with experience across all these disciplines is most suitable to plan the ground investigation, draw together the disparate strands and not produce an overall investigation that responds disproportionately to the most vocal demands.

The need for ground investigation depends on what is intended to be built – and in turn, in extreme circumstances, what can be built depends on the results of the ground investigation. Thus the ground investigation needs to correctly anticipate the likely form of foundations. This cycle is illustrated graphically in **Figure 44.3**.

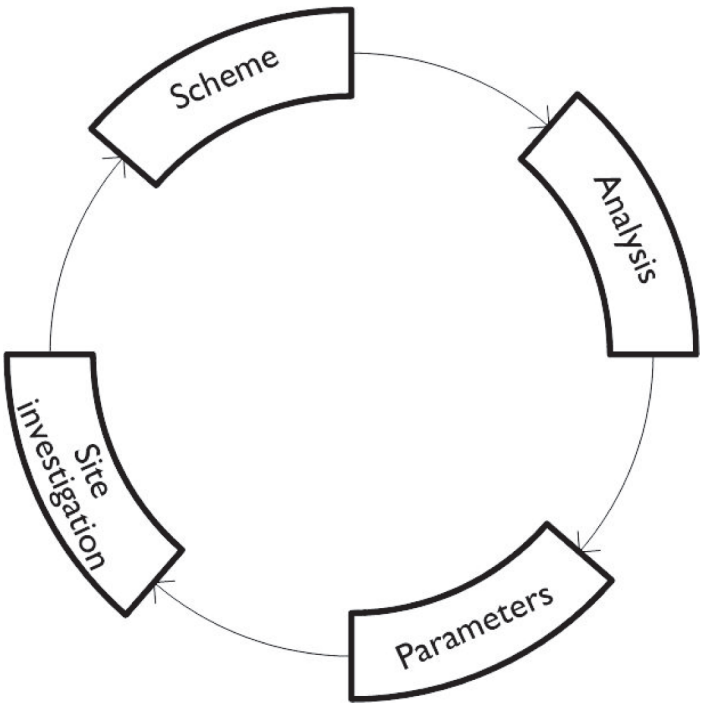


Figure 44.3 Loop connecting scheme with geotechnical analysis, soil parameters and site investigation

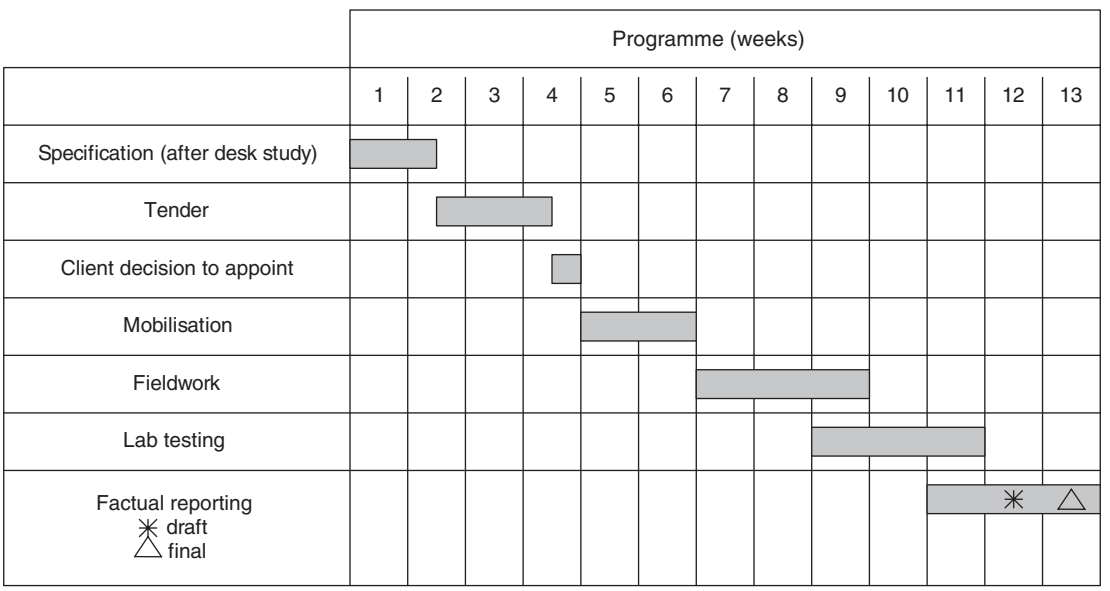


Figure 44.2 Typical overall ground investigation programme

As a minimum, the ground investigation should determine the following features:

- stratigraphy and its variation across the site;
- groundwater regime on and around the site; pressure variation with depth and time; directions of flow across the site;
- characteristics of principal strata in terms of their likely behaviour;
- strength, stiffness, *in situ* stresses and permeability for key strata.

A further aspect needing investigation is the near-surface ground. This is to detect:

- obstructions and buried services – see Chapter 41 *Man-made hazards and obstructions*;
- extent and levels of neighbouring foundations;
- weaker ground that could present a hazard for piling platforms (see Building Research Establishment, 2004) or other relevant temporary needs;
- tree roots in plastic clays, which could lead to desiccation shrinkage and movement of shallow foundations.

The ground investigation should appreciate the possibility of real geological variability and not just expect to validate the planner's preconceptions. The sorts of variability are explained in more detail in Chapter 40 *The ground as a hazard*, but could include, for instance, the presence of a significant sand channel in an estuarine deposit such as the Lambeth Group, or heavily reworked sand or clay such as the very deep post-glacial scour features that occur in London Clay sporadically but often associated with the confluence of old rivers.

It is important at the outset that the designer of the ground investigation is clear about the information that will be needed in the later analyses. It is seldom possible to repeat elements of the ground investigation without considerable disruption, expense and embarrassment.

The parameters that need to be gathered depend on the analyses required to complete the design. They must address all the key design issues in order to allow the most economical methods of construction and least project risk, as shown in **Table 44.1**.

44.2.5 Examples of scope issues

The following points need to be considered in determining the scope of a ground investigation:

- If piles are expected, boreholes will generally need to go deeper.
- If foundations on clay are next to trees, the extent of clay desiccation will need to be assessed.
- If the main construction works are likely to need extensive dewatering, the permeability for each of the strata will need to be estimated.

Clearly, on any site, the ground investigation for a low-rise housing development will be very different from that for a tall office block over a deep basement. Therefore ground investigation reports will always be scheme-specific rather than site-specific. While an old geotechnical report should provide very useful information about a site, its scheme-specific recommendations should be used with caution for subsequent and adjacent developments.

A ground investigation scope is always a balance between more comprehensive coverage and a limited budget. On any project, funds will be tight, particularly for up-front activities. On the other hand, it is not cost-effective to have to repeat site operations because parts of the ground investigation were omitted. It is therefore important that the geotechnical consultant should consult widely with the other team members, prior to completing the scope to:

- Include all requirements, not just those of the structural engineers needing foundations, as explained at the start of this chapter. Others may include the site-wide infrastructure engineer planning roads and car parks, the public health engineer who might be considering SUDS (sustainable urban drainage system), the services engineer who may wish for trial pits to explore existing service routes, and the architect who might be concerned about zones of ground with greater archaeological potential.
- Consider the effects of the whole development on the ground. Thus the car parking requirements may lead to a need for tiered retaining walls; the most cost-effective way to build a basement may be in open cut for which an increased knowledge is needed of the site's hydrogeology and strata permeabilities.

So when planning the ground investigation, a number of choices have to be considered such as the likelihood of:

- shallow or deep foundations;
- retaining walls or open cut;
- alternative access road routes;
- finding contamination or archaeology.

Element being designed	Analysis method	Parameters that need to be measured	Method of measurement
Pile design	Hand calculations	Stratigraphy, water pressures, ground strength	SPT/CPT tests in boreholes; lab tests on samples recovered from boreholes
Pile constructability		Borehole seepages, borehole collapses during boring	Index tests on borehole samples to identify changes in material
Concrete durability	BRE SD1	Sulfate content	Lab tests on borehole samples

Table 44.1 Relationship between pile design and need for information from the ground investigation

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44.2.6 Appropriate testing

44.2.6.1 Matching the testing schedule to the information requirements

The AGS's excellent *Guide to the Selection of Geotechnical Soil Laboratory Testing* (Association of Geotechnical and Geoenvironmental Specialists, 1998) addresses this subject.

Soil testing fulfils two essential aims:

- It confirms quantitatively the variation in nature of each stratum, spatially and with depth – this allows particular strata to be more simply considered together for engineering design, or indicates where a more complex sequence of strata needs to be considered (geologists tend towards more, detail whereas civil engineers tend towards simplification in terms of considering the engineering properties of different geological strata).
- It provides data for design – engineering design of foundations and sub-structures requires a quantitative basis in terms of strength and stiffness for the subsequent analyses.

When planning tests, it is often worthwhile considering these two aspects separately. A simple investigation, such as following the NHBC rules for house foundation design, will be principally concerned with the general characterisation of the soils beneath the site. A major building with deep basement will need to more accurately quantify strength and stiffness for each layer, and may also need other parameters such as permeability. Before planning the test strategy, it is useful to have clarity on the division into likely key strata.

The relative reliance on field and laboratory tests varies between different countries according to local practice. A balance has to be struck between:

- Making measurements crudely on the actual ground at the correct density and *in situ* stresses and in relatively undisturbed state.
- Taking discrete samples back to a laboratory where tests can be made under more controlled conditions, albeit on samples that have been disturbed during fieldwork and subsequent transportation and examination.

Attempts have been made to improve the level of technology applied in field tests – e.g. pressuremeters – and to make laboratory tests more representative – e.g. stress path triaxial tests. In fact, for most typical design cases, the usefulness of all tests depends on their ability to simply apply previously observed behaviour of real foundations in the field to the new situation.

A simple classification of all common test types used on soils in the UK is given in **Table 44.2**.

The range of possible contamination tests is very wide and is addressed in Chapter 48 *Geo-environmental testing*.

For larger structures where more complex analyses are planned, the data gathered in the ground investigation need to be geared towards those analyses – inputting less accurate data into complex analyses can result in meaningless output. Therefore some redundancy is needed to allow a more refined choice of parameters, so sometimes parameters should be deduced from several test types, both field and laboratory.

An advantage of field tests over laboratory tests is that their results are available relatively quickly. A drawback of advanced laboratory tests – particularly tests on clay either in the triaxial cell or shear box at a drained rate of shearing – is that the results can take a considerable length of time to become available. The results may arrive after important design decisions for the whole project have already been taken. The geotechnical designer should not wait for these parameters as the basis for conservative analyses in order to retain the confidence of the rest of the design team and to be able to influence the design correctly.

Laboratory and field techniques for investigating the ground are described in more detail in Chapters 47 *Field geotechnical testing* to 49 *Sampling and laboratory testing*.

44.2.7 Frequency and depths of fieldwork

44.2.7.1 Choosing the right techniques and equipment

The amount of investigation that is appropriate depends on the consequences of missing possible variations. Basing a design on just one exploration point suggests confidence that all stratum boundaries are reliably flat over the whole site and soil properties do not vary. In previously unexplored conditions, three boreholes are needed to define the angle of dip of key stratum boundary planes, assuming no folding or faults in the planes.

The key aim of any investigation is to sensibly investigate the strata and interface levels that are important for design. Thus a presumption towards a building founded on piles might indicate a much more complex investigation than one where

	Field tests	Laboratory tests
Characterisation	SPT cone	Grading (sieve and pipette)
		Atterberg limits mineralogy
Strength	Hand vane pocket penetrometer cone	Triaxial shear box
	Pressuremeter indirectly from SPT	
Stiffness	SPT pressuremeter	Triaxial with local strain measurement
Permeability	Pumping test with pumping well and observation wells (standpipes)	Oedometer test
	Rising/falling head tests	Permeameter

Table 44.2 Range of common soil tests

only shallow foundations need be considered. The implications of these ground investigation planning decisions should be explained to the rest of the design team, so that factors which might invalidate them can be explored.

If parameter uncertainty is the only variable, a poor quality or less extensive investigation can be compensated for by a more conservative design, albeit at a likely higher overall direct cost to the project. For instance, a conservative assessment of skin friction in pile design can obviate the need for more boreholes and more testing. However, reducing the extent of the site investigation may cause one to miss a significant change in ground conditions that would invalidate the choice of pile type. The project will then suffer serious cost and programme penalties while the team designs a more appropriate pile type and the contractor waits with expensive equipment standing idle.

BS 5930:1999 (BSI, 1999), Clause 12.6, recommends the spacing between exploration points:

- a relatively close spacing between points of exploration, e.g. 10–30 m, is often appropriate for structures;
- for structures small in plan area, exploration should be made at a minimum of three points, unless other reliable information is available in the immediate vicinity;
- where a structure consists of a number of adjacent units, one exploration point per unit may suffice;
- certain engineering works, such as dams, tunnels and major excavations, are particularly sensitive to geological conditions, and the spacing and location of exploration points should be related more closely to the detailed geology of the area than is usual for other works.

Eurocode 7, Part 2 (BS EN1997-2:2007; BSI, 2007), Clause 2.4.1.3, advises:

> The locations of investigation points and the depths of the investigations shall be selected on the basis of the preliminary investigations as a function of the geological conditions, the dimensions of the structure and the engineering problems involved.

It goes on to require:

> The depth of investigations shall be extended to all strata that will affect the project or are affected by the construction. For dams, weirs and excavations below groundwater level, and where dewatering work is involved, the depth of investigation shall also be selected as a function of the hydrogeological conditions. Slopes and steps in the terrain shall be explored to depths below any potential slip surface.

Appendix B3 to BS EN1997-2 (Eurocode 7, Part 2) provides useful guidance on investigation point spacings for a range of situations, including a spacing of 15–40 m for 'high-rise and industrial structures', less than 60 m for 'large area structures' and 20–200 m for long linear structures such as roads, railways, retaining walls, pipelines or tunnels. It should be noted that these spacings do not necessarily indicate frequent deep and expensive boreholes – just enough exploration points to detect the more critical changes in ground conditions. For pile design in London Clay, the critical variable with the likely widest variation is the surface level of the London Clay: it can sometimes vary by several metres due to the routes of old rivers, or even by several tens of metres due to scour hollows. This means that more boreholes need to reach and prove the surface of the London Clay than boreholes that go right through the whole stratum and prove its base. Similarly, Made Ground is much more variable and may change over very short distances. Its surface thus needs to be proven more frequently, and its inherent variability needs to be recognised and allowed for in design. Adopting this philosophy should result in the most cost-effective extent of fieldwork-economical but still effective in reducing the key risks.

Where the extent of the development is uncertain, a balance has to be struck between the right level of fieldwork for the most extensive development and the savings that can be made by planning for a more modest development, with the possibility of a return visit later. The choice may need to be discussed with the client to agree what is most appropriate – the client may be prepared to live with the risk that more work will be required later to realise a potential saving. Particularly where the difference in cost is relatively small and the project programme is tight, the client may not be able to live with the risk of missing critical information. Sometimes a compromise is possible where sufficient data are gathered to enable scheme design for all options, but there are only enough data for complete design of the less extensive option. It also depends on the procurement route chosen for the main contract and the mechanism for transferring risk.

As stated earlier, the ground investigation should provide the data for the likely temporary works (so far as they can be predicted at the time). There is unlikely to be time later in the temporary works design for fresh information to be obtained, so the project's overall risks (and probably costs) will be higher if this is not done at an early stage.

44.2.8 Clear campaign aims

Once a campaign of ground investigation has been planned, it is important there is a clear strategy for its execution. For larger and more complex investigations, it is good practice to set out the strategy on a site plan and on an idealised geological section. As each hole is completed, its contribution to the overall campaign can be tracked. This is especially important when there are a number of aspects of the investigation which need to be resolved spatially and with depth. The strategy, if properly executed, should ensure that sufficient results are obtained without breaching the budget. Features that often need a formal strategy at the start of the campaign include:

- stratigraphy – proving the surface of each important layer to get a good spread spatially;
- water pressures – variation with depth, spatially and in each stratum;

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- geotechnical test results – variation in each stratum with depth; detecting spatial variations;
- contamination test results – variation of different chemical determinands in each area of suspected contamination.

An example is shown in **Figure 44.4**.

Without clearly defined aims at the outset, a ground investigation can quickly collapse into chaos, as a huge flow of unrelated data can overwhelm the recipients.

44.3 Procuring the site investigation

44.3.1 Typical costs

The site investigation operations of desk study and ground investigation are some of the best-value operations possible in terms of reducing risks on construction projects. The old adage says, 'you pay for a ground investigation whether you have one or not'. A poorly conceived or planned ground investigation often costs more in the end than a good one, because the cost of project change typically increases as the programme progresses.

Typical costs for ground investigations are shown in **Table 44.3**. Although these data are old, the values for buildings correspond to current experience.

Current costs of ground investigation for buildings are typically in the range 0.1–0.2% of the building cost. For small domestic works, investigation costs can often be higher – perhaps 0.5–1.0% of the works cost.

Thus, for a £30 million structure, the cost of the ground investigation would be typically £30 000 to £60 000. Higher rates apply either where the ground hazards are greater or where the proposed foundation methods may make higher demands on the ground than normal. The lower proportion normally applies when the site investigation is being carried out for a 'park' of several adjacent buildings, and efficiencies are possible, or where the structural form chosen is particularly expensive, but the foundations use simpler techniques.

For major infrastructure projects, current costs of the ground investigation phase prior to construction are in the range 0.1–0.15% of the project value. This does not include the cost of any initial phases of investigation carried out at preliminary design phases.

The ground investigation must target the specific site hazards, so arbitrary limits on its cost based on a fixed sum or typical proportions are likely to lead to a greater exposure to risk for the project. As explained in Chapters 7 *Geotechnical risks and their context for the whole project* and 40 *The ground as a hazard*, ground risks are some of the most extreme faced by most projects and economising at the expense of investigating hazards tends to result in increased costs later, so a very false economy.

A ground investigation is essentially an exploratory process, so its out-turn costs may vary enormously depending on what is found. It must be recognised by all parties at the outset that a ground investigation rig being delayed by obstructions is much cheaper and less programme-disruptive than, for example, the piling rig being delayed by the same obstructions.

Laboratory testing is a significant element of the ground investigation budget, so the extent of testing should not be underestimated when compiling the budget.

A rough rule of thumb is to allow £100–200 per linear metre of borehole drilled, including most types of field and laboratory testing. Thus, an investigation for a £300 million highways project involving 120 boreholes with an average depth of 20 m would be expected to cost approximately £360 000.

A ground investigation cost-overrun as a result of unexpected ground conditions being encountered may be regarded

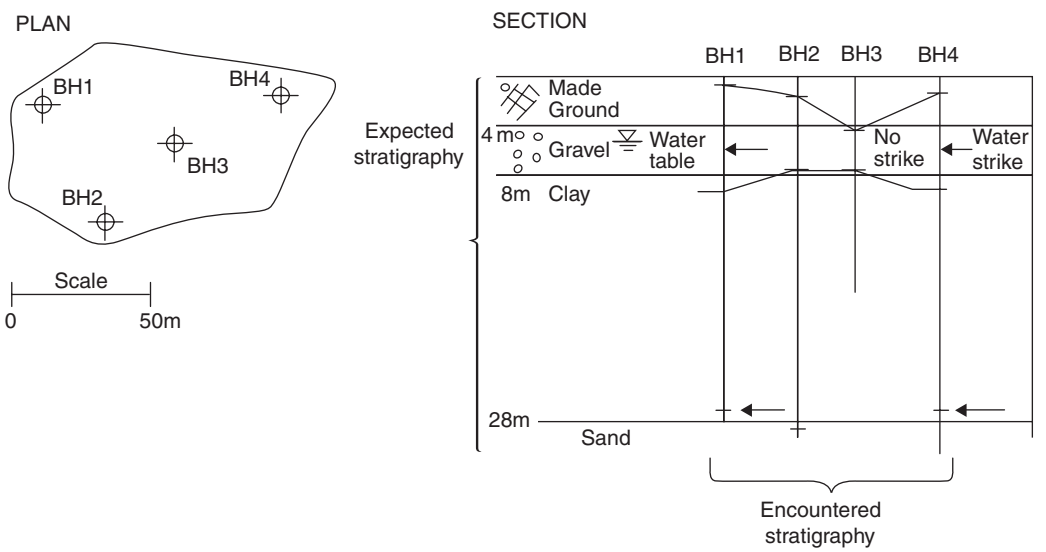


Figure 44.4 Example showing comparison of expected and encountered stratigraphy to assist with managing a ground investigation

as a success, because potential problems have been discovered early and measures can be taken to guard against them during the main construction works. Close, proactive control of the investigation by the geotechnical engineer and a responsive approach from the ground investigation contractor should allow the correct parameters to be determined within the overall investigation budget.

It is prudent to allow a sensible contingency which can be expended during the ground investigation in response to the evolving understanding of the ground. Normally, the contingency is derived partially from a generous estimate for rates which are more likely to be exceeded, plus an extra 10% to allow for unexpected conditions. A full schedule of rates covering all foreseeable tests should be obtained with the ground investigation tender.

If very unusual conditions are revealed, there may be a need for the project team to re-evaluate and agree a new ground investigation budget. The contractual form should allow for the investigation to be varied according to the findings.

44.3.2 Selecting consultants and contractors

Buying a ground investigation is like any other purchase – the buyer wants to make sure they get what and when they want and expects that there will be no nasty surprises on price or programme. The analogy of buying any commodity can be helpful: the buyer wants to be protected against things they do not expect. The more careful buyer will read all the small print and might negotiate to get what they want. But once the contract is signed, they are committed to their particular purchase.

A first step is to make sure that a proper brief is prepared. As few clients are expert in buying ground investigations, professional assistance is needed. The success of the purchase will depend on the clarity of thought that has gone into defining what will be required, and the clarity of specification in explaining that to the supplier. The next two sections explain how to choose the necessary professional assistance. The following sections explain how to describe the work needed in a contractual framework that provides protection to the client.

Type of work	% of capital cost of works	% of earthworks and foundation cost
Earth dams	0.89–3.30	1.14–5.20
Embankments	0.12–0.19	0.16–0.20
Docks	0.23–0.50	0.42–1.67
Bridges	0.12–0.50	0.26–1.30
Buildings	0.05–0.22	0.50–2.00
Roads	0.20–1.55	(1.60)?–5.67
Railways	0.60–2.00	3.5
Overall mean	0.7	1.5

Table 44.3 Typical site investigation costs
Data taken from Rowe (1972)

For a large building project, the **architect** may take on the role of *principal technical adviser* recommended in Part 2 of the *Site Investigation in Construction* series (Site Investigation Steering Group, 1993): *Planning, Procurement and Quality Management* (see Chapter 42 *Roles and responsibilities*). Normally, though, for a building project, it is usually the **structural engineer** who recognises the need for geotechnical support and who advises when it should be commissioned. Most development teams rely on their structural engineer to recommend the approach for geotechnical investigation and design. Their advice is likely to rely on the following factors:

- Geotechnical abilities of the structural engineering firm – especially whether it employs sufficiently skilled geotechnical professionals with experience in assessing ground variability and hazards.
- Attitude to geotechnical risk of the structural engineer – some tend to delegate as much responsibility as possible to other organisations in the belief that their own exposure to the geotechnical risk will correspondingly reduce. This is obviously not true if the choice leads to a greater overall risk to the project from disjointed design. If overall project risk increases, then all will be affected by the consequences.
- Special skills demanded by the particular development, for example whether special investigation techniques or advanced geotechnical analysis will be required.

A structural consultant who also operates as a ground investigation contractor may offer a complete ‘one-stop shop’ service, providing integrated structural and geotechnical design and geotechnical investigation capabilities.

To some extent, it is up to the project promoter to decide whether or not to seek the advice of the structural engineer also for geotechnical matters. This will depend on the confidence he or she has developed in the structural engineer to control what are potentially the highest project risks. A key factor for assessing the competence of the structural engineer for allocating ground risk is to understand the geotechnical ability of the company. If they directly employ or have ready access to experienced geotechnical advice, then they are more likely to engage in the reduction of ground risks.

The project roles outlined at the start of this chapter offer a useful means for assessing the geotechnical competence of the organisations proposed. The *geotechnical adviser* has a key role; previous successful experience and track record on similar projects are good indicators of the suitability of the individual. Before appointing a geotechnical adviser, it is worth challenging their:

- experience with projects of similar structural complexity and demands on the ground;
- familiarity with the expected ground conditions and special hazards faced;
- understanding of the full investigation, design and construction process – beware people who offer a scope of investigation

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without understanding the likely extent of basement and types of foundations;

- views of previous clients for previous projects: Did they perform their duties well? Were they proactive in spotting problems and reducing risks?
- ability to add value to the project and call on extra resources if needed.

An enthusiastic geotechnical adviser should offer a strategy for completing all the geotechnical work for the project, with project-specific ideas on how to reduce risks and improve buildability at each stage. The new UK Register of Ground Engineering Professionals (RoGEP) provides an appropriate qualification route.

The *Site Investigation in Construction* series documents (Site Investigation Steering Group, 1993) make sound recommendations for the employment of a geotechnical adviser to oversee a coherent geotechnical design process. They suggest that the need for this geotechnical adviser should be suggested to the client by the principal technical adviser, who for many projects will be the architect.

Figure 44.5 shows the typical decision-making process for ground investigations proposed by the Site Investigation Steering Group (SISG).

Some geotechnical advisers work for companies that are pure consultancies – that is, they need ground investigation contractors to be separately employed-while some work for companies that also operate equipment and laboratories. There are pros and cons to both alternatives, but generally the combined approach offers advantages for smaller and simpler commissions and ground investigations, while the separated approach is better for larger and more complex investigations, particularly where the client is suspicious of an integrated contractor proposing extra ground investigation purely to increase its own revenue and profit.

44.3.3 Defining the scope of work

The scope of work can be defined either by the consulting engineers providing geotechnical services for the project or by the ground investigation contractor. The former route permits a competitive tender for the work to be carried out fairly – it is unfair (and bad practice) to invite tendering contractors to compete on the extent of work, as that rewards those who are prepared to suggest the least work. This apparent saving will mean that hazards are investigated less thoroughly; contractors tendering for the main construction work will face greater uncertainty and risk, for which they will price, or which will not become clear until unexpected conditions develop during construction.

The scope of ground investigation depends on the scope of the planned construction.

When planning the scope of work needed in a ground investigation, beware of artificial restrictions on its extent – normally imposed by an overly cost-conscious client who does not understand the increased risks the restrictions might lead to. If limitations are placed on the scope, those limitations should be spelled out in the report: either budgetary or on extent caused by lack of access. This allows:

- others to understand risks identified but not followed up, so that the design can address that additional uncertainty;
- responsibility for lack of coverage to be properly apportioned.

44.3.4 Ground investigation contract documents

A clear set of contract documents is needed for any ground investigation to describe the task and to regulate how it is carried out, and these are introduced in Chapter 42 *Roles and responsibilities*. In broad terms, it is helpful to separate these concepts into *specification* and *contract*. The specification should provide the technical requirements and scope, while the contract documents should regulate how the work is carried out in order to avoid disputes. The two parts are distinct and input should clearly be for one purpose or the other, and not mixed. The bill of quantities will reflect both.

44.3.4.1 Specification

The purpose of a technical specification is to clearly define the work that is to be carried out, in terms of both extent and quality. A clear specification offers the following advantages:

- prior to tender it allows the design team to discuss and reach agreement on what work needs to be done;
- during tender, it unambiguously informs each tendering contractor what should be allowed for in the price;
- during the works, it provides a basis for discussion between the client team and contractor should any disagreement occur;
- after the works are complete, it allows fair payment to be made.

Phrases like ‘to the satisfaction of the engineer’ are unhelpful in specifications, because the contractor can never know how easy the engineer will be to satisfy. It rewards more lax contractors who might assume a lower standard over more responsible or experienced contractors, who might make a more realistic or generous allowance. It is seldom necessary to use such phrases, as the engineer can normally replace it with the criteria that he or she would wish to invoke.

A clear technical specification describes:

- the background to the work – its purpose, i.e. what will be built; the expected ground conditions; the current usage of the site, etc., including access and necessary constraints;
- the work that is needed;
- the standards to which the work should be carried out, referring as much as possible to national standards.

Part 3 of the *Site Investigation in Construction* series (Site Investigation Steering Group, 1993): *Specification for Ground Investigation* (the ‘yellow book’), provides a very well-formatted

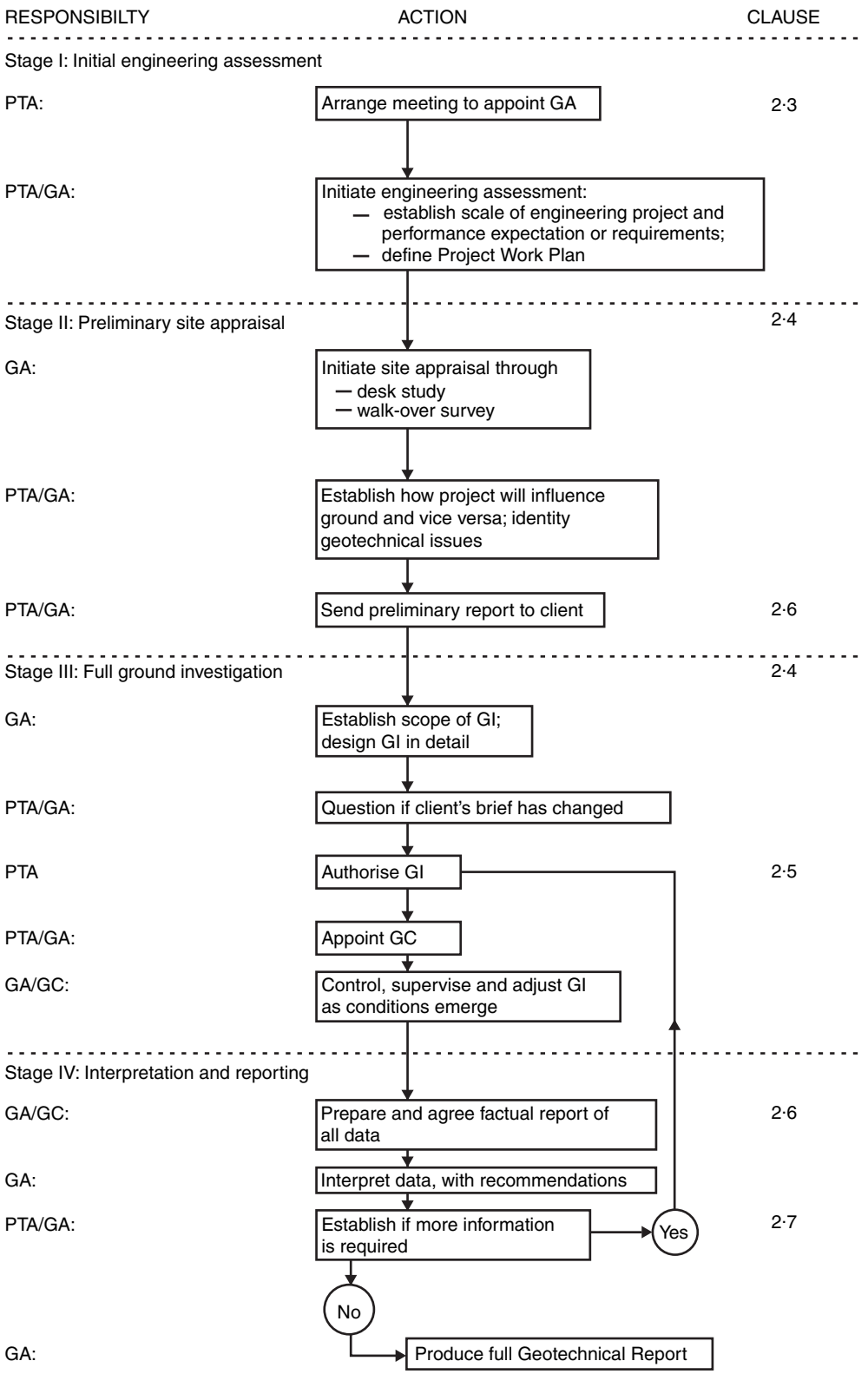


Figure 44.5 Decision-making process of site investigation
Reproduced from Site Investigation Steering Group (1993) Part 2: *Planning, Procurement and Quality Management*

national specification for the UK. National specifications are better than client or engineer specifications because:

- contractors should automatically carry out work to the required standard;
- contractors should be aware of the normal requirements, which facilitates easy tendering with less risk of omissions;
- all work in the UK should comply with the minimum standards, so when a completed ground investigation report is given to a new consultant, the consultant should find it easier to accept its findings.

The yellow book includes a general specification which addresses most of the issues needed for most ground investigations. It also includes draft bills of quantities. It allows the specifier freedom to fully describe the scheme-specific details in a series of schedules:

- Schedule 1 gives the background to the project and describes in words what is needed, together with key constraints, such as access.
- Schedule 2 is a table that lists for each exploration hole what is needed. It can give options – such as borehole depth, which can either be given in absolute terms, or can require the ground investigation contractor to continue to a certain penetration into a stratum.
- Schedule 3 is a list of facilities needed by the site supervisor – normally the engineer's representative under ICE forms of contract.
- Schedule 4 is a list of any amendments that the specifier might wish to make to the standard specification.
- Schedule 5 is a list of additions to the specification, either where extra requirements are being imposed, or where new areas of work are required which are not covered by the standard specification.

44.3.4.2 Forms of contract

For all forms of construction, a pre-agreed contract is necessary for the share of risk between client and contractor to be clear. This is also true for ground investigation contracts to govern what happens if things go wrong. Some common disputes in ground investigations are payment (like all contracts!) and who is responsible if buried services are damaged during the fieldwork.

The conditions of contract must address:

- what the contractor is required to do;
- delegated decision-making powers;
- commencement, programme and completion date;
- alterations and change process;
- delay procedures;
- measurement protocols;
- certification requirements;
- insurances, ownership of the works and safety;
- method of payment (lump sum or re-measurement to bills of quantities), including retentions (for instance to protect the client, who derives little benefit from the fieldwork until the contractor's factual report is delivered; or in case of inadequate reinstatement) and bonds;
- repair of defective work;
- termination clause;
- resolution of disputes.

Many different forms of contract are available. What the investigation will find is uncertain at the outset, so the contract should allow the supervisor to change the order and scope of the works, without too cumbersome a control and approval process.

ICE forms of contract have fallen out of favour recently, because many clients do not like the amount of control delegated to the engineer to make changes without client agreement. However, this feature makes it a very suitable contract for a site investigation, which must be varied if it is to respond nimbly to unexpected conditions. Normally, the amounts being spent are relatively small in the context of the overall project, so the degree of control is less of an issue. The 7th edition of the ICE *Standard Conditions of Contract* Institution of Civil Engineers, 2003b; (current at the time of writing) need modifying to make them useful for use in site investigations. Fortunately, ICE has reissued the second edition of its *Conditions of Contract for Ground Investigations* (CCGI; Institution of Civil Engineers, 2003a), and this is recommended for use where possible. If other forms of contract are used, it is important that they are made so as to allow a swift reaction to unexpected findings. Other forms of contract, such as JCT forms, tend not to work well as they are intended to limit flexibility on site, which can be counterproductive. The New Engineering Contract (NEC) has not so far penetrated the ground investigation market to any great extent. Ground investigation contractors may add a risk allowance to cover unfamiliar contract conditions.

Ideally, for a ground investigation, a resident engineer from the designer will be on site to witness key elements of work and to understand what has been uncovered. If only part-time coverage is provided, the contractor must have clear instructions of how to proceed if unexpected conditions evolve, and a fair way of paying the contractor so that the individual takes the initiative to get the best results for the whole project. The key is to involve the designer, geotechnical adviser and ground investigation contractor in a collaborative approach to solving problems and obtaining the appropriate parameters for the project.

The complete contract documents should encompass:

- conditions of contract, including the form of agreement;
- technical specification;
- priced bills of quantities;
- accompanying drawings, possibly including site location plan, borehole location plans, service routes and historical or geological figures from the desk study.

44.3.4.3 Bill of quantities

The bill of quantities (or sometimes schedule of rates with provisional quantities, to emphasise the potential for variation) allows a tender price to be agreed. The method of measurement

should also be defined, as otherwise differences in measurement understanding may lead to disputes. Beware using provisional sums or 'rate only' items, as in the former case no cost is agreed, and in the latter case, the contractor's tender total is not penalised by the use of high rates.

44.3.5 Risk sharing

In most contractual relationships, the client tries to pass risk to the contractor. In ground investigations, because of the fundamental uncertainty about what might be found, there is greater risk sharing and the client must accept the possibility that more work may be required. A ground investigation that identifies serious features meriting greater attention, resulting in cost and programme overruns, should be seen as a great success and not a failure. The consequences of those features remaining undiscovered until construction would be far worse than identifying them at an early stage.

Where a main contractor is on site and suitable plant is available, the main contractor may be in a better position to negotiate access for the ground investigation with landowners and provide temporary access tracks for investigation plant. Responsibility for, and thus risk on, access could then be removed from the ground investigation contractor's contract, leading to a more competitive price.

44.3.6 Tender evaluation and award

It is important that all tenders are fairly evaluated, on the same basis. Therefore careful consideration is needed to ensure that each restriction or exclusion is evaluated. While the tenders should be evaluated for the understanding demonstrated of the site and tasks, it is too late to regret seeking tenders from a less competent contractor who happens to submit the lowest price. If there were doubts about the contractor's competence, they should not have been invited to submit a tender.

44.4 Managing the site investigation

44.4.1 Safety management

All construction sites are potentially dangerous places. This section highlights some essential features of safety on ground investigations, but should not be taken as a definitive guide. Statutory guidance changes with time, and users must ensure that they comply with the latest regulations – in particular the Construction (Design and Maintenance) Regulations (2007), also known as the 'CDM Regs' – see Chapter 42 *Roles and responsibilities*.

Typical hazards in site investigations include:

- Trial pit collapses, often due to inadequate shoring. This topic is addressed by the leaflet *Safety in Excavations* (Health and Safety Executive, 1997). Regulation 12 of the Construction (Health, Safety and Welfare) Regulations 1996 (SI 1592) requires that 'All practicable steps shall be taken, where necessary to prevent danger to any person, to ensure that any new or existing excavation or any part of such excavation which may be in a temporary state of weakness or instability due to the carrying out of construction work (including other excavation work) does not collapse accidentally.' This used to be set at a 1.2 m safe depth before shoring was required, but the current regulations are more general.
- Injuries in cable percussion boring caused by use of heavy weights – particularly to fingers.
- Hitting buried services – this is a potential cause for major injury to site workers, dispute and disruption to neighbours and businesses. The hazard is so great that time should always be made to obtain plans from all owners of buried services and carry out site scanning before work starts.
- Investigation of potentially contaminated sites.

The location of buried services is critical prior to digging holes on a site. The various steps that should be taken are:

- Contact all the likely owners of buried services to receive contemporary plans – these are often inaccurate and seldom show domestic connections. Sometimes unusual services can be found, like strategic oil pipelines. Ask around. Contact Groundwise.com for the normal range of buried utilities or Linesearch.org for pipelines, in the UK. For sites where the owner has a maintenance department, talk to the people who have worked on the site for a long time.
- Lift manhole and drain covers on the site to locate likely service routes. Lines of replaced concrete or tarmac are often associated with service laying.
- Use a scanner to try to detect pipes and cables – note they seldom find plastic pipes. This can be done in conjunction with representatives from the companies owning the services on site for best results.
- Depending on the likelihood of buried services remaining undiscovered, hand-dig 'starter pits' at each location to reduce the chances of damage being caused. Vacuum excavation techniques are also available to ensure the safety of site workers.

Sometimes major services are a significant constraint on ground investigation operations – both buried utilities and overhead cables. See Chapter 41 *Man-made hazards and obstructions*.

The Control of Substances Hazardous to Health Regulations 2002 (COSHH) require employers to control exposure to hazardous substances which could put people's health at risk.

In buildings dating even to the mid 1980s, the presence of asbestos in lagging to pipes, walls and ceilings should always be suspected. If the presence of asbestos is suspected, no intrusive work should be carried out until its absence has been verified by sight of the building's asbestos register.

The person planning the ground investigation must not locate boreholes and trial pits in locations which could be more hazardous if other safer locations are feasible. For example:

- boreholes under overhead power lines pose a significant hazard for those erecting and using the boring rig;
- trial pits next to a cliff pose a significant threat to the operator of the excavator;

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- machine-excavated trial pits over live buried services pose a threat to all in the vicinity.

However, it may be that after consideration of the aims of the ground investigation, no other locations offer such benefits, in which case the designer must try to mitigate the risks as far as possible, for instance by following the guidance for digging near services, and by only using air blowing or vacuum excavation or hand-digging techniques until all the services have been found.

Particular care is needed when planning investigations inside buildings. Where it is necessary to bore from ground level through a basement, a number of checks are needed:

- make sure that no cables, pipes or ducts will be damaged at any level by the breaking out or boring works;
- make sure that the suspended slab is capable of supporting the weight of the boring rig, especially when a hole has been broken in the floor.

44.4.2 Supervision

A strong case can be made for the ground investigation contractor to provide full-time technical supervision on site. This person will be involved in maintaining technical quality, ordering the works, reacting to anomalies, logging pits, borehole samples and drill runs, and selecting suitable samples for later laboratory testing. Doing these activities rapidly is often essential for samples for subsequent chemical analysis. However, it must be noted that there is a cost associated with provision of such a person, and standard fieldwork rates are seldom adequate to enable the ground investigation contractor to provide that person without some payment. Hence it is helpful if the specification and bill of quantities define the expected basis.

Additional audits should be carried out, normally by the designer or geotechnical adviser, to ensure that supervision is being carried out properly and that the specification is being reliably followed. Such visits should apply both to siteworks and to the testing in laboratories. Often laboratory checks are neglected as it is difficult to arrange the timing so that the particular samples from the site can be witnessed.

It is also of great value if the geotechnical designer can see the actual ground conditions on site, and so additional visits for this purpose should be made. Visits coinciding with the opening of trial pits prove most valuable as the stability of excavations and the inflow of water can be assessed.

44.4.3 Quality management and quality control

Quality management should be undertaken by a registered quality system to ISO 9001 by both consultants and ground investigation contractors. Quality control is vital to ensure that the tasks are carried out diligently and to a reliable set of procedures. Such quality procedures are very important for the simple repetitive testing carried out in soil testing laboratories where small changes in operator procedure can have a profound effect on the results.

44.4.4 Data management

A ground investigation produces considerable quantities of data, and these are usually best managed in electronic form. The ground investigation contract should specify that all data are to be produced in Association of Geotechnical and Geoenvironmental Specialists (AGS) format. This simple, standardised format allows the data to be imported into all spreadsheet packages and specialist borehole management software. From here, the data can be imported into geographical information systems (GIS) and 3D virtual construction models to aid integration with the design model.

Current products under development include robust site datalogger tablets. These will enable the ground investigation operatives to input data direct from the borehole or trial pit and transmit them in real time to the geotechnical adviser. Other useful innovations include GPS tagging of site photographs and barcoding of samples.

44.4.5 Fulfilling vital campaign aims

The ground investigation should have had clear aims at the outset. Those are sometimes changed as the work takes place, but they should remain visible and coherent. Hence, at key stages, it is worth reviewing whether those aims have been met, before it becomes too late. Such reviews can be tricky as at such key stages there are usually many more data and clarity can be difficult. It is worth formally reviewing the aims a week before the ground investigation contractor leaves site, after which time any further fieldwork will become extremely expensive and disruptive, and at the time of scheduling the last tranche of laboratory tests, after which time there is a severe risk of delaying the main deliverable from the ground investigation, the contractor's factual report.

44.4.6 Effects on site occupants and neighbours

The works for the ground investigation are usually the first manifestation of the new project on site. For the reasons outlined in section 44.2.3, occupants and neighbours may not welcome the start of the ground investigation. Therefore, it needs to be started sensitively, and with good supervision to intervene between anxious residents and the site operatives. It is often a good idea to agree a simple communications plan to handle queries, complaints and correspondence in a consistent and professional way.

Every reasonable effort should be made to reduce the level of disruption. If not, the local authority environmental health officer (EHO) in UK locations may become involved and impose severe restrictions on working hours. Where periods of noisy work are inevitable, it is best to inform stakeholders – including the EHO – beforehand, and then keep any promises made.

The level of reinstatement needs to be carefully considered. On the one hand, it can be seen as wasted expenditure, as the main development will obliterate it. On the other hand, a degraded site environment will make it less attractive to users,

may render parts of the site unusable and could make it unsafe. Issues to consider include:

- reinstatement of slabs, hard standings, roadways and grassed areas;
- future protection of standpipe and piezometer stopcock covers.

44.4.7 Reinstatement

It is important that the site after completion of the fieldwork is left in a condition acceptable to the site occupiers/owners. This means that:

- Paved areas should be reinstated as needed.
- Grassed areas should be rehabilitated; this includes wheel marks for access.
- Structures should be repaired, and at least made safe. It is not unusual in investigations in buildings for boring to be made from ground floor level through a basement: in such cases the holes should be securely fixed and may need to be made weatherproof to prevent water leaking into the basement, creating a further safety hazard.
- All the old spoil should be cleared away; some contractors have even been known to abandon equipment at the site.

In addition, the investigation should not leave the subsoil in a condition that could cause problems in the future:

- Boreholes should be grouted up on completion. Ungrouted boreholes can leave a connection between different aquifers, which can spread pollution and can also be a source of hazard for piling and tunnelling work.
- Trial pits should be well compacted. Loosely compacted trial pits later become zones of unexpectedly weak ground. This can be a hazard for shallow foundations and for temporary piling platforms. For this reason, for housing developments it is preferable to position pits away from the footprint of the houses where known.

44.4.8 Contract administration and completion

As in all contracts, standard resident engineer (RE) records should be kept so that payment items can be fairly evaluated. The RE may be required to resolve misunderstandings with neighbours, particularly when the contractor has no professional staff on site. Payments need to be processed according to the contract, with certificates provided as necessary.

The RE records should be kept, as they may be instructive in understanding any anomalies detected on the site subsequently – for instance a length of rotary cored drill-hole over which no core was recovered may have been thought to have been lost because of poor drilling but it may later transpire to be due to cavities in the ground.

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It is recommended this chapter is read in conjunction with

- Chapter 8 *Health and safety in geotechnical engineering*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 45

Geophysical exploration and remote sensing

John M. Reynolds Reynolds International Ltd, Mold, UK

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This chapter provides a brief overview of the range of geophysical methods (surface and down-hole) and remote sensing available for use in engineering investigations. The techniques described are well proven, and when used in a properly designed site investigation can add significantly to the robustness and cost effectiveness of such a survey, especially when used in combination with intrusive methods.

The chapter explains the role of geophysics and describes how best to procure geophysical surveys. Current best practice is to engage the services of an independent engineering geophysical adviser to design the works, supervise the fieldwork and undertake the detailed analysis, modelling and integrated interpretative reporting. Guidance is provided as to which geophysical techniques are best suited to which applications.

The most commonly used geophysical techniques are described with examples of outputs available. These include micro-gravity and magnetometry; electrical methods (resistivity tomography, induced polarisation, self polarisation); electro magnetic methods (including ground penetrating radar); seismic methods (refraction, reflection, surface waves and ground stiffness); over-water surveys (hydrographic and sub-bottom profiling); and borehole logging techniques.

A brief overview of remote sensing image analysis is also provided for optical (e.g. Landsat, SPOT, ASTER), synthetic aperture radar (SAR), LIDAR and thermal infra-red (TIR) techniques.

45.1 Introduction

Investigating the ground remotely can be achieved with a range of techniques, from satellites orbiting the Earth using optical imaging, through to radio waves being generated from a hand-held sensor on a concrete slab inside a building to locate steel reinforcement bars. Sensors can be deployed from different platforms to produce information for regional investigations over thousands of square kilometres to ultra-high-resolution testing on a decimetre scale.

With advances in technology over the last few years there is also a blurring of the boundaries between the modes of deployment of techniques. For example, LIDAR (LIght Detection And Ranging) using laser measurements was originally made from fixed-wing aircraft to produce high-resolution digital elevation models of the ground surface. Now the same technique is also available from ground stations looking laterally at rock cliffs to monitor slope stability and from boats to image marine infrastructure, for example. Individual geophysical sensors have become smaller and more sensitive, and capture data faster with modern electronic and computer technology. Multiple sensors can be mounted on a wide range of platforms (hand-held, towed devices, booms and stingers on aircraft, 'birds' suspended from helicopters; single sensors, horizontal and vertical gradiometers, and multi-channel systems with many sensors mounted in a range of configurations).

Many geophysical techniques have been transformed over the last few years by faster and more efficient data acquisition, so that much more information can be captured in much less time, making it possible to increase the spatial resolution by at least an order of magnitude. Sensors have become more sensitive, so permitting the detection of ever smaller targets. There has also been massive improvement in geophysical data processing capability, and in the availability of increasingly sophisticated software. Recently, it has become possible and practical to combine both geophysical and borehole data analytically into 3D ground models.

This chapter provides a very brief overview of the range of geophysical and remote sensing methods that are available. Every modern engineer should be aware of these techniques, which have all become well established and are thoroughly proven. The chapter is sub-divided into an overview of geophysical methods (including over-land and water, airborne and down boreholes) and a brief description of key remote sensing techniques.

45.2 The role of geophysics

The primary role of geophysics is to reduce risk. By undertaking a professionally designed and executed survey it should be possible to:

- *reduce* health and safety risks – avoiding buried utilities, locating voids, etc.;
- *reduce* professional indemnity risks – not missing key features on a site;

- *reduce* over-engineering of structures to cover uncertainty in ground investigations;
- *reduce* project delays and associated costs through 'unforeseen ground conditions'.

And it should be possible to:

- *increase* the technical reliability of site investigation information;
- *increase* the cost-effectiveness of the site investigation through targeting intrusive tests on anomalous areas;
- *increase* the technical robustness of ground models through integrating multiple data sets and types.

The primary benefits of geophysical surveying (Reynolds, 1996) include:

- rapid areal coverage (hectares per day);
- fine spatial resolution (to $\ll 1$ m);
- volumetric sampling rather than spot measurements;
- non-invasive and environmentally benign nature;
- time-lapse measurements;
- quantitative rather than qualitative data.

One of the primary benefits of geophysical investigations is that they are environmentally benign by being largely non-intrusive. Most methods involve a technician moving a sensor either across the ground or just above its surface, or placing metal electrodes or spiked geophones into only the top few centimetres of the ground surface. The measurements are made without disturbing the subsurface, which makes some methods ideal for time-lapse surveys, where profiles are re-surveyed after a period of time to determine any changes in the subsurface environment. In contrast, a borehole, for example, might provide a pathway for fluid flow, thereby affecting the local hydrology; removal of the borehole casing does not restore the situation to a pre-borehole status.

Geophysical methods are able to sample at a fine spatial interval, often of the order of every 10 cm, or in the case of ground penetrating radar, at less than every centimetre. By covering whole swathes of a site these techniques provide almost complete coverage of the available ground. At the most basic level, such surveying provides 2D reconnaissance by which anomalous areas can be identified. This on its own is only of qualitative use. It might indicate relative anomalies that might show high and low values of apparent conductivity, but not what might be causing such values to occur. This is where the intrusive testing is so valuable. As will be demonstrated later, intrusive investigations, where deep enough and appropriately detailed, can complement geophysical techniques and permit the translation of variations in a geophysical parameter to specific ground conditions (Reynolds, 2004). By correlating the material types with the apparent conductivity anomalies, the spatial extent of these materials might be indicated.

Where geophysical techniques are used to provide information about both horizontal and vertical variability along a profile, and where the analysis provides the means of determining depths to subsurface targets, detailed borehole logs can be invaluable in providing ground truth data. Borehole logs can define layer thicknesses and material types that can be used to constrain geophysical modelling, so reducing a degree of freedom in the interpretation process. It is important to emphasise that geophysical methods do not necessarily reduce the number of intrusive tests required but permit the positioning of such tests in places where they are needed. This makes the combination of methods far more effective than intrusive testing alone and can reduce overall costs significantly.

It is crucial, however, that geophysical surveys are designed to meet a particular technical objective, such as to map the boundary of a former closed landfill. This dictates the spatial sampling required, the likely techniques to be used, what kind of analysis is needed to yield the necessary information, and how the survey should be undertaken. A common mistake in procuring geophysical investigations is to choose a method based on its familiarity to the specifier or on the general availability of equipment, rather than selecting the most appropriate method(s) to meet the survey objectives. Data acquisition is a means to an end, not an end in itself. Many unsuccessful geophysical surveys undertaken have been a waste of money because they have been incorrectly specified, poorly designed or undertaken with inadequate field practices (sometimes using inappropriate techniques), with inadequate position fixing and/or even misusing the equipment. Furthermore, there are many examples of cases where perfectly adequate field data have been wrongly interpreted, as little thought has been given at the design and specification stage to the analytical methods required once the data had been collected. It is fundamental to the integrity of each geophysical survey that thought is given first to the design and execution of the analysis, data processing and interpretation of the geophysical results and only secondly to the specification of the data acquisition. For many modern-day surveys, a specialist independent engineering geophysical adviser (EGA) should be appointed to the project team along with a geotechnical adviser, as recommended by the guidelines on engineering geophysics (McDowell *et al*., 2002). Best practice is for the EGA to design the overall geophysical investigation (from data acquisition through to integration of results and interpretation) and also to supervise the site works and undertake the detailed interpretation of the final results. Data acquisition can be undertaken by suitably experienced geophysical contractors. Further advice on the use of geophysics in engineering has been provided by Schoer (1999), Darracott and McCann (1986), Anon. (1988), Reynolds (1996), McCann *et al.* (1997) and, for archaeological investigations, David *et al*. (2008).

Given the range of geophysical methods generally available, it is possible to categorise the applications to which they are most suited or not appropriate at all. An extremely general

guide to the possible usefulness of the various methods is given in **Table 45.1**. However, in many cases it is advisable to use more than one method in order to reduce possible ambiguity in the interpretation and to respond to different physico-chemical characteristics within the same environment. For example, in the investigation of a former landfill, electrical resistivity tomography is an ideal method to sense the conductive waste and possible leachate plumes migrating away through the base and/or sides. The resistivity results would respond predominantly to the electrical properties of the fluids present rather than to the lithological boundaries, for which the seismic refraction method might be better suited. Consequently, these two techniques are both applicable and are complementary. Similarly, in environmental investigations over brownfield sites, a common pairing of techniques is electro magnetic mapping with magnetometry. Furthermore, coincidence of anomalies can help to increase confidence in the interpretation. The information given in **Table 45.1** should not be used prescriptively for survey design or specification purposes. To increase the benefit of any geophysical survey, the design and specification should be prepared by an experienced and suitably qualified EGA. A guide to the physical properties of materials and the corresponding geophysical techniques sensitive to those properties is given in **Table 45.2**.

45.3 Surface geophysics

There are four groups of geophysical techniques in common usage and each of these is described briefly in turn. While the emphasis is on the use of these methods over the ground (or water) surface, some techniques can also be readily deployed in airborne and/or down-hole modes. Most commonly, 1D soundings and 2D profiles are acquired. Data from closely

Geophysical methods	Depth to bedrock	Stratigraphy	Lithology	Fractured zones	Fault displacement	Buried channels	Natural cavity detection	Mine workings/ adits/shafts	Groundwater exploration	Landfill investigations	Leachate plumes & migration	Brownfield site mapping	UXO detection	Buried artefacts/ USTs
Potential field methods														
Gravity	1	0	0	0	2	2	4	4	1	3	0	1	0	1
Magnetics	0	0	0	0	2	1	0	2	0	3	0	4	4	3
Electrical methods														
Resistivity – sounding	4	3	3	2	2	3	2	2	4	4	4	0	0	1
Resistivity – tomography	3	2	2	4	3	3	3	3	4	4	4	2	0	3
Induced polarisation	2	2	3	1	1	2	0	0	3	3	2	2	0	0
Self potential	0	0	0	2	2	1	1	1	4	4	3	2	0	0
Electro magnetic methods														
FDEM	2	2	2	4	2	3	4	4	4	4	4	4	4	3
TDEM	2	2	2	3	2	3	1	1	3	3	3	1	4	1
VLF	0	0	0	1	1	1	2	2	3	3	0	1	0	0
Ground penetrating radar	2	3	1	2	3	2	3	3	2	2	0	2	3	4
Seismic methods														
Refraction	4	3	2	3	4	4	1	1	2	4	0	1	0	1
Surface wave profiling	3	4	3	4	3	3	2	2	2	4	0	2	0	1
Reflection – land	2	2	2	1	2	1	2	2	2	2	0	0	0	1
Reflection – over water	4	4	2	2	4	4	0	0	0	0	0	0	0	2

Key: 0 = Not considered applicable or untried; 1 = limited use; 2 = used, or could be used, but not best approach, or has limitations; 3 = excellent potential but not fully developed; 4 = generally considered as excellent approach, techniques well developed.
Note: Applicability of techniques is dependent upon a range of parameters including physical/chemical contrast with host material, target dimensions, scale of survey and intended depth penetration, site suitability, site dimensions and access, surface topography and material type, local noise, etc. It is always best to select the method(s) on a site-specific basis and with respect to the nature of the targets being sought. It is emphasised that this table is a very general guide only and actual suitability of a technique to a site may vary from the values indicated above. Always seek advice from a professional geophysicist.

Table 45.1 Suitability of geophysical methods for a range of shallow engineering applications
Data taken from McCann *et al*. (1997)

Geophysical methods	Dependent physical property	Hydrocarbon exploration	Regional geological studies	Mineral exploration/ development	Engineering site investigations	Environmental investigations	Hydrogeological investigations	Detection of sub-surface cavities	Mapping of leachate & contamination plumes	Location of buried metal objects	Archaeogeophysics	Forensic geophysics
Potential field methods												
Gravity	Density	P	P	s	s	s	s	s	!	!	s	!
Magnetics	Susceptibility	P	P	P	s	P	!	m	!	P	P	!
Electrical methods												
Resistivity	Resistivity	m	m	P	P	P	P	P	P	s	P	m
Induced polarisation	Resistivity; capacitance	m	m	P	m	s	s	m	m	m	m	m
Self potential	Potential difference	!	!	P	m	s	P	m	m	m	!	!
Electro magnetic methods												
FDEM	Conductivity; inductance	s	P	P	P	P	P	P	P	P	P	m
TDEM	Conductivity; inductance	P	P	P	m	s	s	!	m	s	!	!
VLF	Conductivity; inductance	m	m	P	m	m	s	s	s	m	m	!
Ground penetrating radar	Dielectric permittivity; conductivity	!	!	m	P	P	P	P	s	P	P	P
Seismic methods												
Refraction	Elastic modulus; density	P	P	m	P	m	s	s	!	!	!	!
MASW/ground stiffness	Elastic modulus; density	!	!	!	P	!	s	m	!	!	!	!
Reflection	Elastic modulus; density	P	P	m	s	!	s	m	!	!	!	!

Key: P = Primary method; s = secondary method; m = may be used but not necessarily the best approach or has not been developed for this application; (!) = unsuitable.

Table 45.2 Common geophysical methods, their dependent physical properties and their generalised main applications
Data taken from Reynolds (2011)

spaced 2D profiles can be incorporated into a 3D volume. For specialist applications it is possible to acquire true 3D data sets by deploying a grid of detectors all of which sense signals from a common source (e.g. 3D resistivity and seismic reflection surveys). Re-surveying profiles/grids after a significant period of time permits the acquisition of time-lapse surveys, by which it is possible to determine changes as a function of time. It is beyond the scope of this chapter to explain the principles of operation in any detail; readers are directed to more comprehensive sources of information (e.g. Milsom, 2003; Reynolds, 2011).

45.4 Potential field methods

45.4.1 Gravity

The gravity method is sensitive to variations in the density of the ground, and gravimeters are designed to detect changes in the Earth's gravity field. This is measured by determining the changes in length of a spring resulting from the force exerted on a mass suspended on the spring. Modern micro-gravity meters are capable of making measurements to an accuracy of better than 5 μGal with a daily productivity rate of between 50 and 80 stations per day, with typical anomaly amplitudes of several tens of μGal in engineering investigations (such as in void detection) (1 μGal = 1 cm/s²). It is essential to be able to survey in each measurement point to an accuracy of better than ±0.1 m horizontally, and ±0.02 m vertically to achieve the best results. Given their heightened sensitivity, modern gravimeters pick up noise and vibrations, including distant earthquakes, which can degrade the data quality and slow the survey rate. The instrument provides values of gravity measured over discrete time periods, typically 60 s at each measurement 'station'. The values are averaged either to give a mean figure over that time period or until a designated standard deviation has been achieved. It is essential also to establish a base station at which values of gravity are measured at intervals typically no greater than every 2 hrs, depending upon the type and characteristics of the instrument being used. These base station values are used to derive the diurnal drift of the instrument, which has to be compensated during the data correction process.

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Gravity values (g_{obs}) are influenced by earth tides, the position on the Earth's surface with respect to its shape (oblate spheroid) and surface elevation, terrain, isostacy (in the case of larger scale surveys), physical movement if the gravity meter is being used from a moving platform (ship or aircraft), and the physical presence of the ground itself. These corrections are required to reduce the values of g_{obs} to what is known as the Bouguer anomaly (Δg_B). This may need to have the regional gravity gradient removed and also the values corrected for the presence of buildings and walls, so that the data to be interpreted are corrected for all above-ground influences. The resultant anomaly is known as the residual Bouguer anomaly, which should indicate only the variations in the density of the materials beneath the gravimeter at a given station.

The residual Bouguer anomaly data can be plotted as 2D contour maps showing the variation in values across a site. Low values represent areas of density deficiency that might be attributable to the presence of voids or poorly consolidated backfill; high values may represent dense infill or higher-density bedrock. Commonly, data along extracted profiles can be modelled to produce 2D sections indicating the dimensions of a target, its depth of burial, and its density contrast with the surrounding material (indicating for instance if a void is air-filled, partially filled with loose material, etc.) (**Figure 45.1**).

45.4.2 Magnetometry

Modern caesium vapour magnetometers measure the total intensity of the Earth's magnetic field, which varies from around 30 000 nT (nanotesla) at the equator to 60 000 nT at the poles (Finlay *et al.*, 2010). If magnetisable objects are present, they distort the Earth's field in characteristic ways. Magnetometers are capable of measuring to 0.01 nT sensitivity with a sampling rate of up to 10 measurements per second, with common anomaly amplitudes ranging from a few tens of nT to many hundreds of nT. The ground coverage achievable is dependent upon the speed at which the instrument is transported along a profile (transect). At walking speed this equates to about one measurement every 10–20 cm. Position fixing with a differential global positioning system (dGPS) can be achieved and recorded simultaneously with the magnetic field intensity values. It is possible to use magnetometers either singly or with multiple sensors to measure horizontal and/or vertical magnetic gradients.

Recorded data can be displayed as 2D colour contoured maps of the magnetic field intensity and horizontal or vertical magnetic gradient (nT/m), and as discrete profiles. Interpretation is significantly more complicated than for micro-gravity, as magnetic measurements are a function of both the induced magnetisation of the target given a magnetic susceptibility, and any inherent remanent (permanent) magnetisation. Where induced magnetisation is the predominant cause, the anomaly shape and size are also dependent upon the orientation of the target, its dimensions, and its depth of burial. Magnetic profiles can be modelled and field values compared with synthetic values derived for a user-selected model, which is changed until the difference between the observed and synthetic data

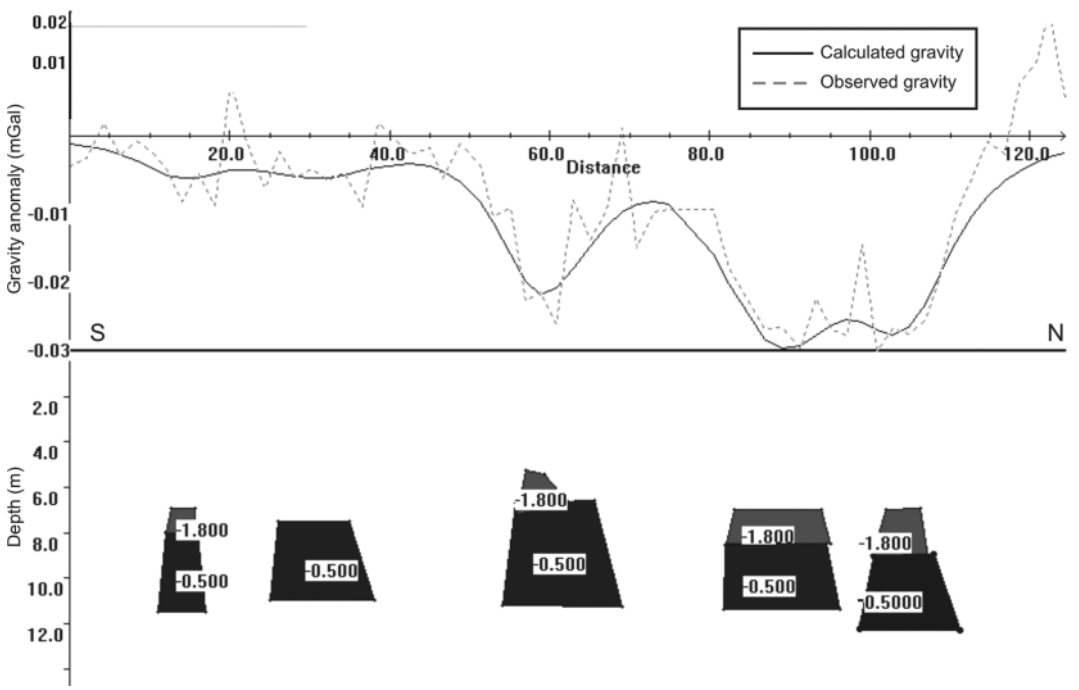


Figure 45.1 Bouguer anomaly micro-gravity profile showing observed and modelled values over mine galleries in chalk. The negative values refer to the density contrast with the host rock, with −1.8 Mg/m³ indicating an air-filled void and −0.5 Mg/m³ a partially back-filled void. These results were confirmed by subsequent probe drilling.

falls within a predetermined standard deviation or RMS error (**Figure 45.2**).

Common applications of magnetometry in engineering investigations are to locate ferrous metal objects (underground storage tanks (USTs), iron pipes, steel drums), but it can also be used to map magnetically susceptible material such as ash, clinker and bricks, and to delineate zones with different materials within landfills. A growing use of magnetic methods is in the detection of unexploded ordnance (UXO), such as ferrous metal bomb casings, mortar shells, etc. However, the technique will not find UXO devices specifically designed to avoid detection, such as modern land-mines.

45.5 Electrical methods

45.5.1 Electrical resistivity

Electrical resistivity techniques detect changes in the electrical potential between a pair of electrodes generated by a separate pair of electrodes through which a low-frequency alternating current is injected. By increasing the separation between electrodes, greater depth penetration can be achieved. The technique effectively measures the resistance for a given electrode geometry to produce an *apparent* resistivity (ρ_a). This value in itself is a form of volumetric average and has no direct physical significance. There are two principal modes of use: (a) vertical electrical sounding (VES), and (b) electrical resistivity tomography (ERT).

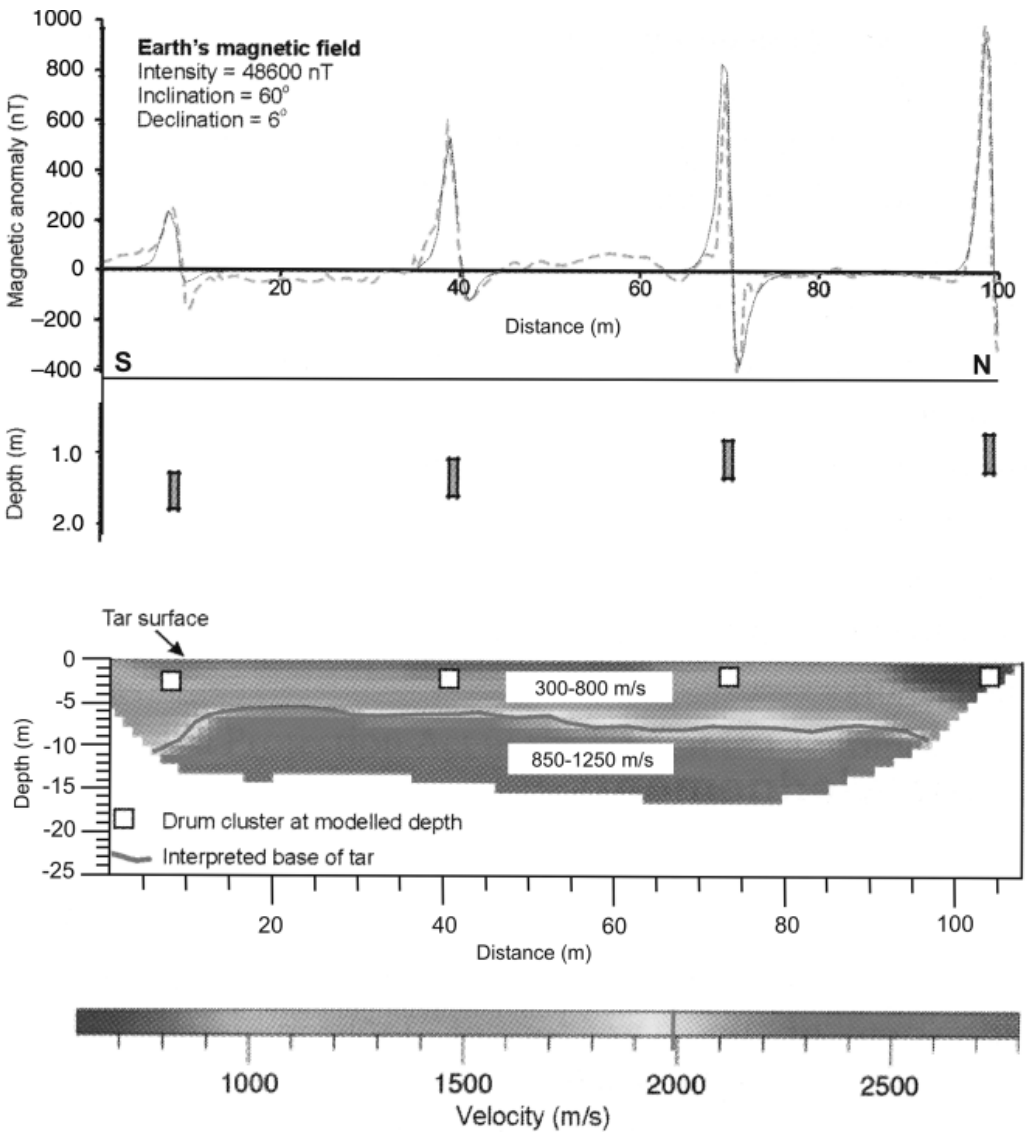


Figure 45.2 (Top) Profile of magnetic field intensity showing observed results (solid line) and those generated by modelling (dashed line) the depth of burial of steel drum targets in an acid tar lagoon. (Bottom) Small squares indicate the position of the steel drums as derived from the magnetic modelling, superimposed on the results of seismic refraction velocity modelling, indicating their positions within viscous tar above a solid base
Reproduced from Reynolds (2002), European Association of Geoscientists and Engineers

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VES is achieved by measuring ρ_a using an expanding array of electrodes to produce a one-dimensional vertical profile (sounding) of ρ_a as a function of electrode separation. VES curves can be analysed (using a process called 'inversion') with commercially available software (e.g. IX1D, Interpex Ltd.) to produce an interpretation comprising a series of horizontal layers whose thickness and *true* resistivity are derived by the inversion process. The *true* resistivity is characteristic of the type of material present.

The ERT method uses a series of electrodes, commonly 72 or more, connected by a multi-core cable to a resistivity meter by which a 2D panel of apparent resistivity values is constructed. The resistivity data can be inverted using specialist software (e.g. RES2DINV, Geotomo Software) to produce 2D sections showing modelled values of true resistivity along the profile as a function of depth (**Figure 45.3**). The software also permits topographic corrections to be applied.

ERT is used more frequently than VES nowadays, although the latter is extremely useful in providing better vertical resolution of a layered structure; soundings are, however, sensitive to 3D structures, making the 1D inversions unreliable under these circumstances. Software is available to produce 2D and 3D models of resistivity so as to be able to test the detectability of particular structures and targets.

Resistivity techniques are used particularly where fluids in the pore spaces within materials have contrasting resistivity values, such as in mapping leachate plumes and imaging landfills. They are also useful for investigating natural and man-made voids and mine workings, for hydrogeological investigations, for imaging through landslides and in geohazard studies.

Two other electrical techniques are also used in geoengineering and environmental applications, namely *induced polarisation* (IP) and *self (spontaneous) polarisation* (SP).

45.5.2 Induced polarisation (IP)

Induced polarisation profiles are acquired in a similar manner to ERT sections but in addition to measuring the apparent resistivity, values of chargeability, which are a measure of the amount of polarisation observed, are also acquired. IP is used in base metal, geothermal and groundwater exploration, but also increasingly in environmental investigations.

45.5.3 Self polarisation (SP)

The SP method uses two non-polarisable electrodes to measure the electrical potential between them generated by the flow of ion-rich water. The method is used in landfill studies and especially to detect leaks through embankments and natural dams, and also in hydrogeological investigations.

45.6 Electro magnetic (EM) methods

45.6.1 EM methods – non-GPR

There are a wide range of electro magnetic (EM) techniques available, of which the commonest and most widely used (and abused) is ground penetrating radar (GPR). In this section, non-GPR techniques are described. There are two main types of EM equipment: (a) frequency-domain EM (FEM) and (b) time-domain EM (TEM). FEM uses a pair of coils at a fixed separation (the dipole length); one coil is used to generate an EM field that propagates into the ground. A subsurface conductor responds to the primary EM field by setting up eddy currents within itself that in turn generate a secondary field that travels back to the surface and is measured by the secondary receiver coil. The ratio of the primary and secondary fields is a measure of the *apparent* conductivity of the volume of material being sampled. Conductivity is the inverse of resistivity; apparent conductivity is a function of the conductance of the ground and the geometry of the dipole arrangement and in

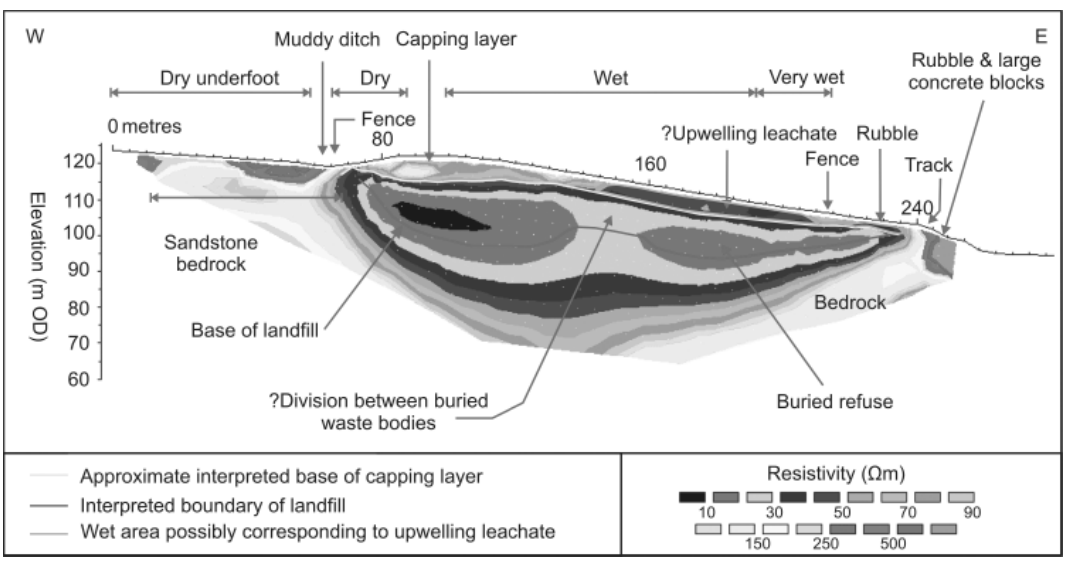


Figure 45.3 Electrical resistivity tomography section across a closed landfill



Figure 45.4 Geonics electro magnetic ground conductivity meters: (a) an EM31 with GPS antenna, and (b) an EM34 (white coils)

itself is not physically diagnostic of material properties. The most popular EM equipment is made by Geonics Ltd. They produce a family of EM ground conductivity meters, ranging from the smallest, the EM38 (nominal depth of penetration to 1.5 m) that is used predominantly for archaeological and agricultural surveys, through the EM31 (nominal depth of penetration to 6 m) with two coils in a rigid boom (**Figure 45.4(a)**) to the EM34, that has three sets of dipole separations (10 m, 20 m and 40 m) with separate coils (**Figure 45.4(b)**), giving nominal depths of penetration to 15 m, 30 m and 60 m respectively. Other manufacturers have developed instruments (e.g. GEM2) that comprise a fixed dipole separation with two coils in a rigid boom but achieve a graduated depth of penetration by using different frequencies, with the highest frequencies being used for the shallowest investigations and increasing depth of penetration with lower frequencies. Typically, these instruments have five to eight different frequencies available. All of these instruments measure two components of the EM field, the in-phase and quadrature components. The latter, which is 90° out of phase, is a measure of the apparent conductivity, while the in-phase component gives an indication as to the presence of metalliferous material (ferrous and non-ferrous). Data can be recorded simultaneously with position-fixing data from connected dGPS systems, permitting very rapid ground coverage and display of apparent conductivity and in-phase component at each measurement point. All of these systems are capable of recording up to 10 values a second. These displays show the relative variation in apparent conductivity across a site and are very useful at a reconnaissance stage of an investigation.

FEM surveys are extremely effective where the targets are conductive, such as in landfill investigations, leachate plumes, brownfield investigations, searches for metal culverts, detection of live electric cables, etc.

TEM systems work on a different principle from FEM systems. A coil of wire is energised by passing a current through it. This generates an EM field that propagates into the ground. The current is then switched off and a receiver coil is used to measure the residual response of the decaying secondary field. The rate of decay is affected by the conductivity of materials present. Measurements of the secondary field are made as a function of time following shutting off the current. Coil sizes are very large where great depth of penetration is required (many hundreds of metres). Special shallow-sensing devices known as very early time-domain EM (VETEM) instruments are used where higher resolution is required in the near surface, typically the top few tens of metres. TEM systems are used to produce a 1D vertical sounding at each location. Profiles are produced by undertaking a series of soundings along a transect and then gridding the results to form a 2D section. TEM soundings can be inverted like 1D VES profiles to produce an interpretation with layer thicknesses and *true* conductivities that are physically diagnostic of the material present. The results from adjacent 1D soundings can be correlated and gridded to form a 2D true conductivity-depth model.

Geonics Ltd. also make geophysical metal detectors based on the TEM principle (EM61-MK2 and EM63). Two small flat-lying coils are mounted vertically above each other and the time decay measured. These instruments have a shallow depth of penetration (< 5.5 m) and high lateral resolution, and are used effectively and extensively for the detection of UXOs and USTs.

45.6.2 EM methods – ground penetrating radar (GPR)

GPR works on the basis of transmitting radio waves with a known centre frequency from an antenna and measuring the time it takes for signals to be reflected back to a receiving antenna. By measuring the two-way travel time (TWTT) and by knowing (or assuming) the speed at which the radio waves travel through the material present, it is possible to measure the depth to a target. In many respects the technique is conceptually similar to the seismic method where sound waves are used.

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GPR systems have been developed for specific uses. Broadly, these fall into very low frequency systems (frequencies < 10 MHz) for deep investigations (≫ 100 m), particularly of polar ice sheets and glaciers; those for geological applications (25–500 MHz); those for engineering investigations (500 MHz–1.5 GHz); and those for non-destructive testing (NDT) investigations, (frequencies above 900 MHz to 2.5 GHz) , where the required depth of penetration is of the order of decimetres but the resolution needed is down to around 1 cm or less. Special GPR systems have been developed for utility mapping and to be used by non-specialist technicians. GPR antennae can be deployed by dragging an antenna system across the ground, towing it behind a vehicle or mounting it on a boat or aircraft. As a general rule: high frequency, fine vertical resolution; low frequency, coarse vertical resolution. GPR techniques are not effective in highly conductive environments due to the high attenuation of the radar signal, i.e. they should not be used in saline or conductive clay-rich environments.

GPR surveying works on the principle that reflections will occur where there is a contrast across an interface between the relative dielectric permittivities of the materials present. Consequently, the reflection characteristics can be as important in interpretation as the TWTT values. The biggest uncertainty in interpreting GPR data is the variability in radio-wave velocities when translating time-domain radargrams (the raw data images) to depth images. GPR will not achieve any significant depth of penetration in areas with high conductivity, such as in saline environments or over seawater. It is, however, capable of imaging through fresh water. In the UK, typical depths of penetration are of the order of a few metres; in resistive environments, imaging can be achieved to more than 10 m using a low-frequency system.

The GPR method is used in a myriad of applications such as: geological mapping; environmental, engineering and archaeological investigations; surveys of highways and airfields; bridge decks; ballast surveys for railways; and in mapping animal burrows. It can be used to measure the water table and its characteristics, and groundwater and contaminant movement (DNAPLs and LNAPLs, dense and light non-aqueous phase liquids, respectively). Hydrocarbons floating on the water table can be imaged in the right conditions.

Raw radar data can be processed much in the same way as seismic data, with a wide range of processing techniques available to filter, amplify, enhance and migrate the data to aid interpretation (**Figure 45.5**). As with seismic data processing it is full of potential pitfalls and opportunities to make mistakes for the unwary, and GPR data processing and interpretation should be undertaken only by skilled and experienced radar processors and interpreters.

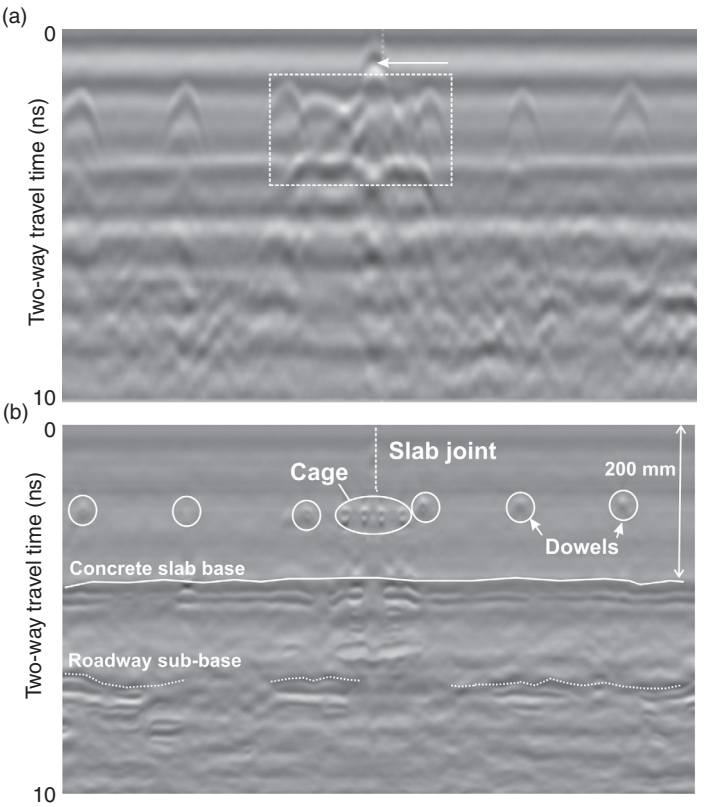


Figure 45.5 Example of (a) raw and (b) migrated GPR data acquired across the junction of two adjacent reinforced concrete slabs within a road pavement

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45.7 Seismic methods

Seismic methods use the generation and propagation of elastic body waves through the ground and the measurement of their respective travel times. A wide variety of sources exist by which the signals are generated, such as a sledge-hammer strike onto a base plate, blank shotgun shells fired vertically into the ground (e.g. 'Buffalo' gun), accelerated weight drops, and explosives, to name but four. The signals are detected by geophones (on land) or hydrophones (in water). The type of source is chosen based on anticipated ground conditions and target depths. As with the GPR technique, there is a trade-off between vertical resolution and depth of exploration.

There are three broad styles of seismic surveys, ranging from seismic refraction and reflection through to the use of surface (Rayleigh) waves. Each of these will be described briefly in turn.

45.7 1 Seismic refraction

A seismic refraction survey comprises an array of geophones and an energy source, such as a hammer strike onto a ground plate ('shot'). Each shot generates seismic waves that travel to a boundary across which it is assumed that the seismic velocity increases; the energy is refracted along the interface and re-radiated back towards the surface at the critical angle (i_c) of refraction, where $\sin i_c = V_1/V_2$ where V_1 and V_2 are the seismic velocities above and within the refractor, respectively, and $V_1 < V_2$. By measuring the time taken from the shot to when

the signals arrive at each of the geophones, a time–distance graph can be constructed, from which it is possible to compute the seismic velocities of the surface, and deeper, layers causing the refraction. After careful processing, seismic velocity images can be produced showing 2D sections along a profile that indicate the lateral and vertical variations in sound-wave velocity (**Figure 45.6**). Depths to refractor interfaces can also be derived.

Refraction surveys are used to determine the depth to bedrock, to locate disturbed zones indicating faulting, to measure the depth of landfills and backfilled quarries, for foundation surveys for large dam construction, to locate major solution features, and for ground investigations for other major engineering structures (tunnels, roads, embankments, etc.). For shallow engineering investigations, depths of investigation are typically less than 20 m.

45.7.2 Reflection seismology

Seismic reflection measures the two-way travel time taken for a seismic wave to travel from a source vertically down into the ground and back to the surface. It provides information about the geometry of subsurface structures, and on the physical properties of the materials present. These techniques are highly developed in the hydrocarbon industry and there is an increasing transfer of such technology to the high-resolution shallow engineering and environmental sector.

The reflection method uses the same types of sources as refraction surveys, and depths of investigation range from at least 20 m (at the shallowest) down to several hundred metres. There are many ways to process the recorded data, using a wide range of commercially available software. Typically, the raw data are edited, filtered, amplified, processed to remove the effects of the geophone array geometry and subjected to the analysis of seismic velocities, to produce a 2D section, which is typically what is then interpreted. Further data processing is possible to enhance the data to aid further interpretation and to translate time sections to depth sections. Details of the type of data processing available have been described extensively by Yilmaz (2001). Seismic reflection surveying is commonly used in engineering studies for deep bridge foundations, and road and tunnel route surveys to map geological structures.

45.7.3 Surface waves

There are a number of different techniques that use surface shear waves or Rayleigh waves to determine physical properties of subsurface materials. For the purposes of this chapter, two common methods are discussed, namely, (a) ground stiffness sounding, and (b) multi-channel analysis of surface waves (MASW).

45.7.3.1 Ground stiffness sounding

The majority of strain levels within the ground at working loads are typically less than 0.1%. It is therefore important that stiffness values to be used in geotechnical calculations should also be measured at these small strain levels. One way to measure the very small strain stiffness, G_{max}, is to use ground stiffness 'sounding' or continuous surface wave seismics (CSW), which is a non-invasive analogue of the cone penetration test (CPT) or standard penetration test (SPT) (Butcher and Powell, 1996). A vertical 1D sounding comprises the determination of the variation of G_{max} as a function of depth to provide the required information on the geomechanical properties of the subsurface layers (i.e. hard and brittle, soft and plastic, etc.). The technique works on the basis of using a multi-frequency vibrating source, typically weighing 70 kg (**Figure 45.7(a)**), which can be tuned to a given frequency (f) in the range from 1 Hz to several hundred Hz. At the chosen frequency, the seismic vibrator generates Rayleigh waves through the surface of the ground that are detected by a short array of between two and six geophones. Nominal depths of penetration are equivalent to one third of the wavelength, which is controlled by the frequency of the source wave. By measuring the phase difference and wavelength across the geophone array for a given frequency it is possible to derive the ground stiffness at a range of depths. A 1D ground stiffness sounding is achieved by taking measurements at discrete frequencies from high to low frequencies, giving the mechanical strength of the materials in terms of the resistance to shearing forces. Rayleigh wave velocity (V_r) is typically 5% lower than that of shear waves (V_s) so the derived velocities are adjusted accordingly (Butcher and Powell, 1996), as G_{max} is proportional to V_s^2. Ground stiffness soundings can reach to depths of 20 m or more (**Figure 45.7(b)**), depending upon the materials present, and can reveal detailed variations in value up to several hundred MPa.

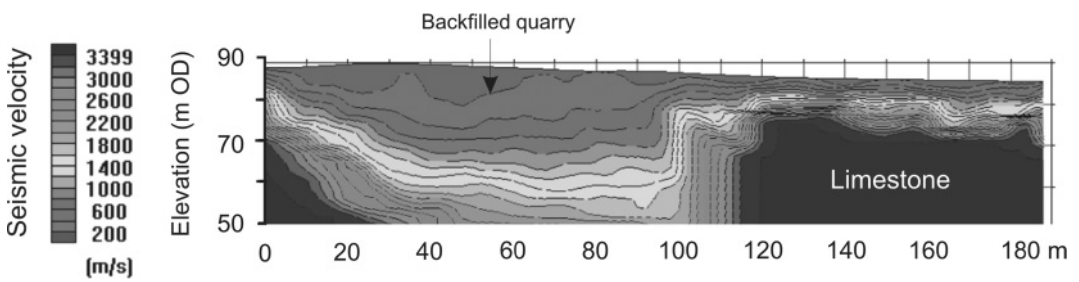


Figure 45.6 Seismic refraction velocity section across a closed landfill in a former quarry in limestone

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The MASW method uses the same layout as for seismic refraction. A shot record is produced and processed, and the shear wave velocity is determined as a function of depth below the given geophone array. The results of each sounding are juxtaposed to form a 2D section displaying the variation in shear wave velocity as a function of depth and position along the array (**Figure 45.8**). The method is used to map zones of weak ground, fractured rock and voiding.

45.7.3.2 Over-water hydrographic and seismic surveys

For engineering investigations over water, such as for bridge foundations, mapping palaeochannels, offshore wind farms, etc., hydrographic and seismic surveys are commonly used. The principle of these surveys is similar to those used for hydrocarbon exploration, but they are suitably scaled down for the intended applications. Over-water hydrographic and seismic surveys can be undertaken in lakes and artificial lagoons, rivers, estuaries, docks and canals, in water depths as shallow as 0.5 m.

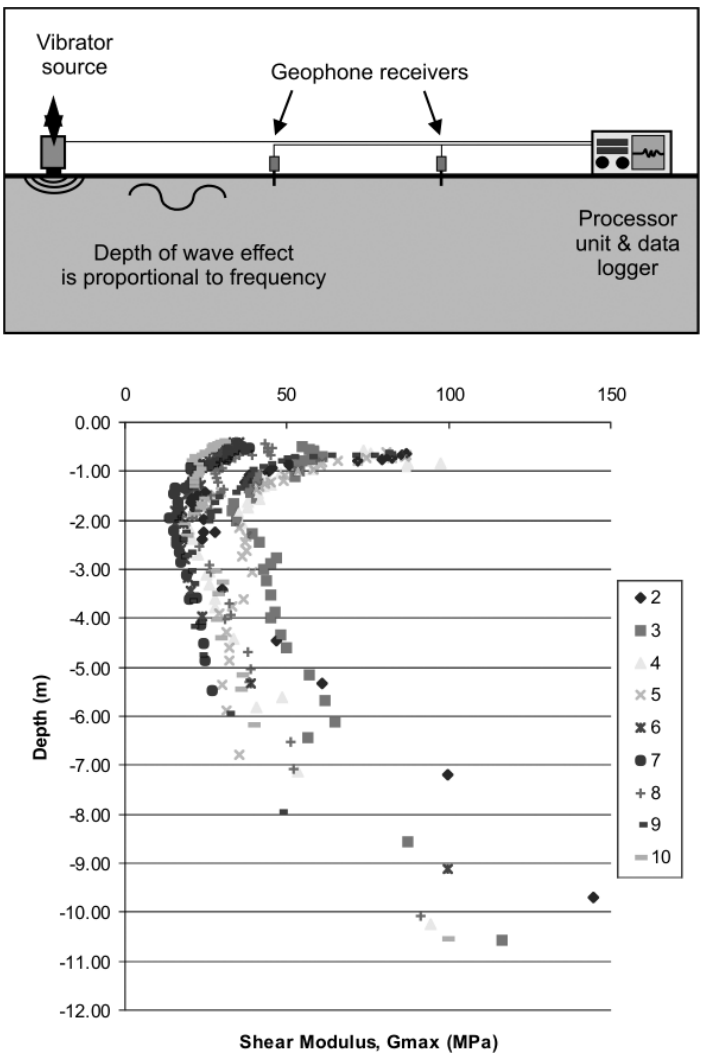


Figure 45.7 (a) Schematic showing the general layout of equipment to derive a ground stiffness vertical profile using a Rayleigh wave generator. (b) Example 1D ground stiffness profiles through landfill material

Side-scan sonar – This is a water-borne technique where high-frequency (100–600 kHz) pulses of sound are beamed from a towed transducer through the water column vertically below the survey vessel and to either side as the vessel progresses. Reflections are produced from objects within the water column below the sensor and from the seabed surface, and their locations and acoustic characteristics are recorded and displayed on a sonograph, which is effectively an image of insonified features. Side-scan sonar images are processed to form scaled mosaic pictures of the seafloor (analogous to aerial photographs of the ground surface), from which seabed features (e.g. sand waves and ripples, gravel and clay patches, iceberg and trawl scours, boulders, wrecks, anchor chains, etc.) can be identified. Individual features, such as a boulder as small as 0.3 m high, can be imaged.

Swathe bathymetry – Traditional echo sounding provides an estimate of water depth vertically below the acoustic transducer. Bathymetric maps are produced by contouring echo sounding values across an area. However, far greater spatial accuracy is achieved by using multi-beam echo sounding (MBES; swath bathymetry), which uses an oblique-facing transducer to isonify a swathe of ground at right angles to the direction in which the transducer is being moved. This permits the determination of values of water depth for pixels of 0.5 m size, with vertical resolution of 0.1 m across a swathe width typically four times the water depth. Systems can also produce a back scatter image analogous to **side-scan sonar** that produces an acoustic image of the seafloor or lake bed. By geo-referencing each pixel, 2D side-scan sonar images can be draped over the swathe bathymetry results to produce 3D acoustic images of the seafloor or lake bed.

To investigate below the seafloor or lake bed, it is necessary to deploy a high resolution **sub-bottom profiling** system. Seismic sources typically consist of a single transducer system with very high resolution (Pinger), or a Boomer plate source, which is used with a string of hydrophones in a single oil-filled buoyant flexible tube (streamer). Vertical discrimination is of the order of 30 cm. Where greater depth penetration is required (at the expense of vertical resolution) higher energy sources such as a Sparker (which generates a sound pulse by an electrical discharge in salt water), air gun or water gun can be used. The acoustic source generates sound waves that travel in all directions. The waves that penetrate into the subsurface are reflected back to a string of hydrophones towed close to the seismic source to produce a seismic image. At a basic level of interpretation, such as for dredge studies, several reflections are picked to produce simple line interpretations showing the elevations of selected interfaces along each survey profile. However, if the data are converted to an internationally recognised data format standard called SEG-Y it is possible to undertake much more sophisticated data processing

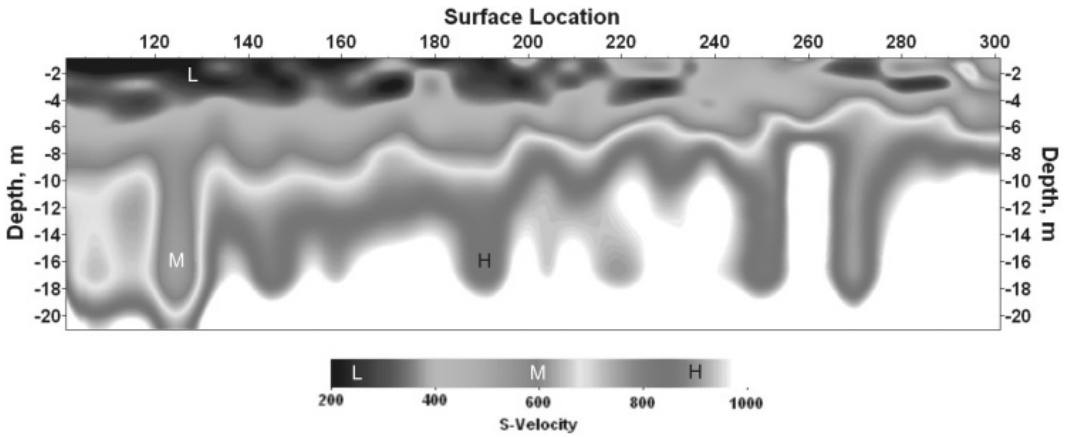


Figure 45.8 An MASW shear-wave velocity section across an area with areas of weak ground

and analysis, such as minimising the effects of multiple reflections, and filtering to enhance the primary reflections. It is also possible to analyse the data in oil-industry-standard 2D and 3D interpretation software, in which it is also possible to display borehole data. Lithological boundaries can be identified and projected onto the seismic data to facilitate the selection of the corresponding reflection events, which can be gridded to produce 3D seismic surfaces. While the levels of post-processing and analysis are much higher than for the most basic single-channel interpretation, the resulting interpretations have a greater level of confidence and technical robustness. Such high levels of analysis are justified for major engineering investigations such as for large-bore tunnels (e.g. Crossrail and Thames Tideway projects) in London and for offshore wind farm investigations prior to finite-element engineering design for pile foundations.

45.7.3.3 Integration with other data sets

Geophysical data are commonly interpreted without reference to other information, i.e. an unconstrained interpretation (satisfies statistical error limits, appears physically plausible and is internally consistent). Many geophysical models can have an almost infinite number of equally valid solutions. Where complementary geophysical techniques have been used over the same ground, separate interpretations can be produced for each method and compared with each other to achieve consistency. For example, it might be possible to use seismic refraction results to yield estimates of depth to bedrock. A corresponding resistivity section might then show that, for example, conductive leachate is migrating into and through this otherwise resistive bedrock, as the resistivity sections would respond to the pore fluid component rather than the solid matrix on which the seismic method depends. Although it is possible to jointly process complementary geophysical data sets to yield a single interpretation, this is non-standard currently.

A more comprehensive approach would be to integrate other forms of information, such as historical data from a desk study, aerial photographs, remote sensing images, airborne or downhole geophysical results, etc., so that a more holistic interpretation can be produced. This can be achieved where each data set is geo-referenced to a common coordinate system; where appropriate the information can be managed through geographical information system (GIS) software. Some dedicated geophysical software also permits this approach, such as Oasis Montaj (Geosoft) and Surfer (Golden Software). It is possible to integrate geophysical software with GIS, which also permits the incorporation of borehole data in digital (AGS) format (**Figure 45.9**).

45.7.3.4 3D visualisation

If 2D geophysical data can be incorporated into a 3D interpretation package, it is possible to produce 3D interpolations and surfaces that can then be viewed from a 3D perspective. Surfaces can be shaded and illuminated from different directions to aid the visualisation of the 3D model. Fly-throughs of the model can also be generated.

45.7.3.5 UXO investigations

A sad legacy of war and other forms of military conflict is the presence of unexploded ordnance, much of which in the UK, for instance, was delivered through aerial bombardment during World War Two. However, UXOs are also found on military bombing and firing ranges. Consequently, there is an ongoing requirement to develop techniques that will successfully and unambiguously detect remnant ordnance so that it can be rendered harmless and cleared. In 2008 CIRIA published the first comprehensive guidance for the construction industry on unexploded ordnance (Stone *et al.*, 2009). Despite the fact that geophysical techniques form one of the most significant methods of remote detection of UXOs, this publication barely mentions the techniques, or more worryingly, the limitations of such methods.

Primarily, magnetic, electro magnetic (FEM and TEM) and GPR techniques are used for UXO detection. Commonly,

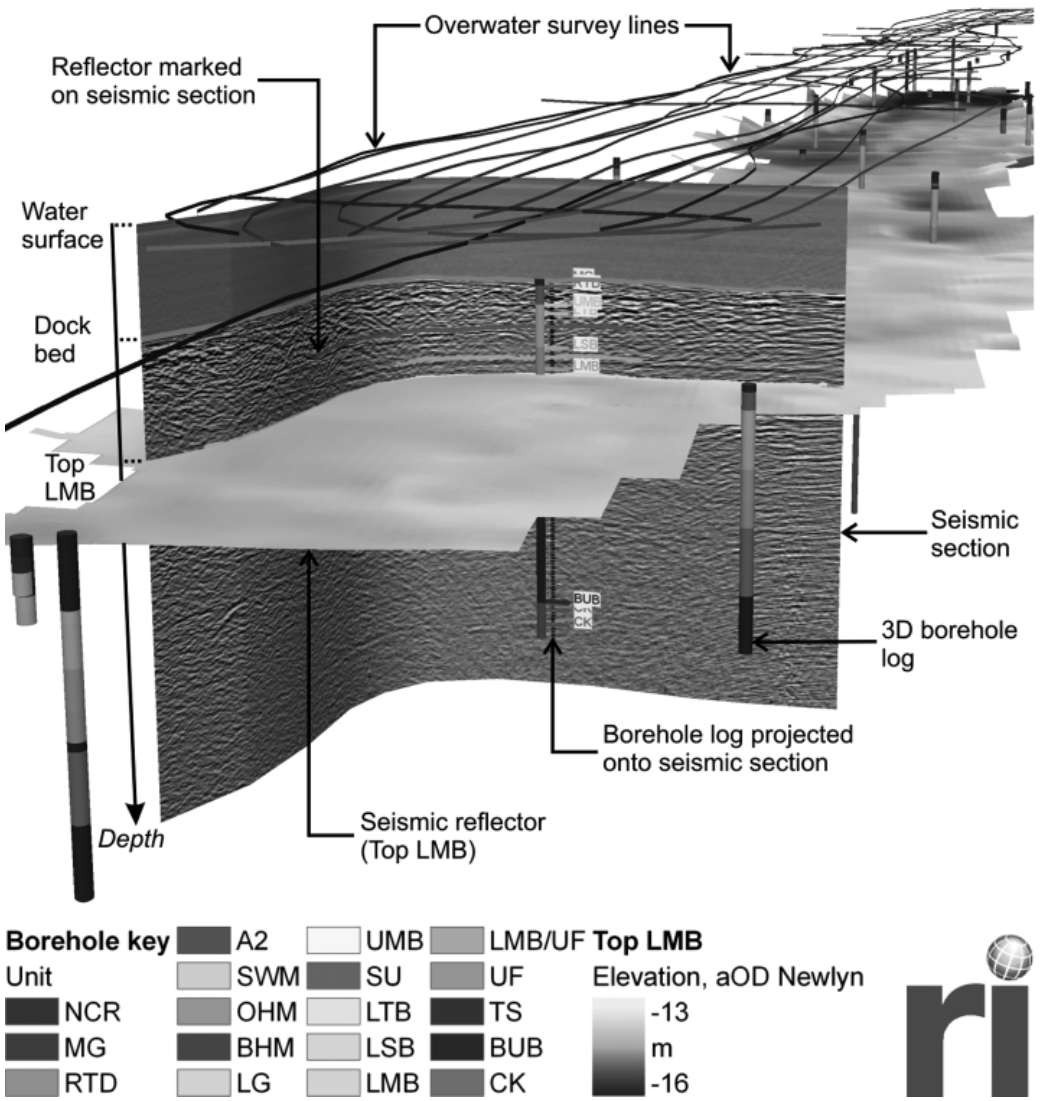


Figure 45.9 Boomer seismic reflection data, Boomer survey track plots and borehole data integrated within a 3D interpretation package

instruments are deployed using a single operator, who may carry a number of magnetic sensors, for example walking across a site; or instruments can be deployed as multiple sensor arrays from a variety of platforms to investigate a swathe of ground in a single pass. For coverage of large areas, such as bombing ranges, helicopter-mounted multiple sensor arrays of magnetometers or TEM coils can be used, where the helicopter flies with a ground clearance as low as 1 m (**Figure 45.10**). Where the presence of UXOs is suspected in sediments beneath water, suitable submersible sensors can be deployed, such as marine magnetometers or gradiometers, or TEM systems (such as the Geonics EM61-SS).

Magnetometers can also be used with a cone penetrometer test rig, where the sensor is pushed into the ground or inserted into a pre-drilled borehole (with non-magnetic casing), the hole being constructed in 1–1.5 m stages and the sensor lowered into each hole extension incrementally. Both the borehole and CPT approach require the sensor to be able to detect the presence of ferrous UXO within 1–1.5 m ahead of the hole.

Some software packages are available that provide specific tools to help pick potential UXO targets. However, such programs do not guarantee that all possible UXOs will be identified, and a manual interpretation by a geophysicist experienced in UXO detection should also be undertaken to help reduce uncertainties in the interpretation. Neither is there an increased guarantee of higher quality of work if a military search team is involved. In all cases where UXO risk is to be assessed, clients must spend time determining the qualifications and experience of the teams to be contracted to investigate a site. The 2008 CIRIA guidance is useful in providing information on how to do this.



Figure 45.10 Helicopter-borne systems for UXO detection: (a) FEM and (b) magnetic multi-sensor gradiometer systems
Courtesy of Batelle Inc., USA

45.8 Borehole geophysics

An important adjunct to surface ground investigations is the deployment of geophysical sensors down boreholes. These can be as single sensors, or down-hole tools in which a number of different sensors are installed. There are two main forms of deployment: (a) single *down-hole techniques* (borehole logging), where the output is a vertical log of a measured parameter as a function of depth down the hole, or (b) *cross-hole tomography*, where a pair of normally vertical boreholes is used with some form of geophysical source being placed at various depths in one hole and the series of detectors being located in the other hole. Signals are transmitted between the holes to produce a 2D panel of data between the boreholes. Borehole tomography is commonly undertaken using seismic, electrical or radar methods. The separation between boreholes should not normally be more than 10 times the minimum dimension of the target being sought, i.e. boreholes should be no more than 10 m apart to image a target with a 1 m diameter. Cross-hole tomography uses techniques that are analogous to their surface equivalents and will not be discussed further here.

There is a wide range of borehole logging tools available (**Table 45.3**) for a large number of different applications. As with surface geophysical techniques, tools should be selected depending upon the most diagnostic physical properties of the targets being sought. Borehole logging can have a very high vertical resolution but may have a very limited penetration beyond the wall of the borehole. However, inter-borehole correlation can also be used to define geological structure (**Figure 45.11**). Geophysical CPT sensors (soil moisture probe, seismic cone) and traditional down-hole tools should also be considered in survey design.

Specialist borehole logging contractors are available through whom advice can be sought about the most appropriate logging tools to be used for a given purpose. Before constructing access boreholes, the size of the down-hole tools, and whether they can work in cased or uncased boreholes or in air- or fluid-filled boreholes, should be determined.

45.9 Remote sensing

Remote sensing represents a wide spectrum of techniques relating to optical, infra-red and radar imaging, predominantly from orbiting satellites. However, there is an increasing range of additional platforms from which observations can be made, such as aircraft (fixed-wing and helicopters for aerial photography, LIDAR, thermal infra-red (TIR), hyperspectral imaging), blimps (unpowered, unmanned inflatable platforms), drones and related autonomous unmanned vehicles (AUVs – airborne and submersible), and elevated aerial platforms and masts from surface vehicles, such as for near-surface TIR imaging. Hyperspectral imaging uses between 100 and 200 spectral bands of relatively narrow bandwidths (5–10 nm), in contrast to multi-spectral data sets, which usually comprise about 5 to 10 bands of relatively large bandwidths (70–400 nm). For the purposes of this chapter, discussions are restricted to satellite imaging, LIDAR and TIR methods.

45.9.1 Optical remote sensing

Optical remote sensing has revolutionised the geosciences in being able to image large areas of the Earth's surface with increasing levels of accuracy and ground resolution. Not only are 'pictures' produced but also multi-spectral data sets that can be analysed in ever more sophisticated ways to determine a number of properties of the Earth's surface. The choice of image type is a function of the ground area to be covered, the required ground resolution, and the purpose of obtaining the imagery (with the largest image sizes having the coarsest ground resolution). For example, different applications might require panchromatic (black and white only), four-colour (red, green, blue and near infra-red (NIR)) or more spectral bands. The level of processing of the image data is also dictated by the application, i.e. whether to use the image for illustrative purposes (lowest level of processing) through to full ortho-rectification, which is the technique by which distortions are corrected in image geometry resulting from the combined effect of variations in terrain elevation and non-vertical angles from

Borehole geophysical methods	Depth to bedrock	Stratigraphy	Lithology	Fractured zones	Fault displacement	Buried channels	Natural cavity detection	Mine workings/ adits/shafts	Groundwater exploration	Landfill investigations	Leachate plumes & migration	Brownfield site mapping	UXO detection	Buried artefacts/ USTs	
Borehole logging techniques generally	4	4	4	3	4	3	2	2	4	1	2	2	2	0	
Cross-hole seismic	0	2	3	4	4	3	3	3	0	0	0	0	0	0	
Cross-hole GPR	0	3	3	4	4	3	4	3	0	0	0	0	0	0	
Borehole magnetometer	0	0	0	0	0	0	0	0	0	0	0	0	3	1	

	Bed boundaries	Bed thickness	Bed type	Porosity	Density	Permeable zones	Borehole fluid quality	Formation fluid quality	Fluid movement	Direction of dip	Shale/sand indication	Fractures/joints	Casing	Diameter	Type of hole
Self potential	•	•	•					•			•				O,W
Long and short normal, & lateral resistivity	•	•	•	•				•				•	•	•	O,W
Natural gamma	•	•	•								•		•		A
Gamma-gamma	•	•	•	•	•	•						•			A
Spectral gamma	•	•	•												A
Neutron	•	•	•	•		•									A
Fluid conductivity						•	•		•			•			L,O,W
Fluid temperature						•			•			•			L,O,W
Flow-meter						•			•			•			L,O,W
Dip-meter	•									•		•	•		O,W
Sonic (velocity)	•	•	•	•								•			L,O,W
Caliper	•	•				•					•	•	•	•	A
Borehole televiewer	•	•				•						•			O,W

Key: 0 = Not considered applicable or untried; 1 = limited use; 2 = used, or could be used, but not best approach, or has limitations; 3 = excellent potential but not fully developed; 4 = generally considered as excellent approach, techniques well developed.
Types of borehole construction: L = Lined; O = Open hole; W = Water/mud filled; A = All types including air-filled.
Note: See the notes to **Table 45.1**.

Table 45.3 Suitability of borehole geophysical methods for a range of shallow engineering applications

the satellite to each point in the image at the moment of acquisition. Ortho-rectification allows the image to be incorporated into a geographic information system to maintain full spatial referencing in a designated coordinate system.

Widespread coverage of the Earth's surface was originally provided by Landsat satellites, the first of which was launched on 23 July 1972. Subsequent missions have provided improved sensor capabilities, with Landsat 5 being launched on 1 March 1984 with both multi-spectral (resolution 57 m by 79 m) and thematic mapper (resolution 30 m) sensors on board; the thermal image scene has a resolution of 120 m. Landsat 7, launched on 15 April 1999, has eight spectral bands (three visible, two NIR, one TIR and one panchromatic) all with 30 m ground resolution, plus a thermal band with 120 m ground resolution. Landsat 5 and 7 both provide scenes with an area of 170 km by 185 km. Since October 2009, all Landsat imagery has been freely available. The Landsat Data Continuity Mission is due to be launched in 2012 to provide imagery through to 2020.

The ASTER system comprises three separate sensors – visible and near-infra-red (VNIR; 15 m ground resolution), short-wave infra-red (SWIR; 30 m), and TIR (90 m), with a scene size of 60 km by 60 km. The system images to latitudes

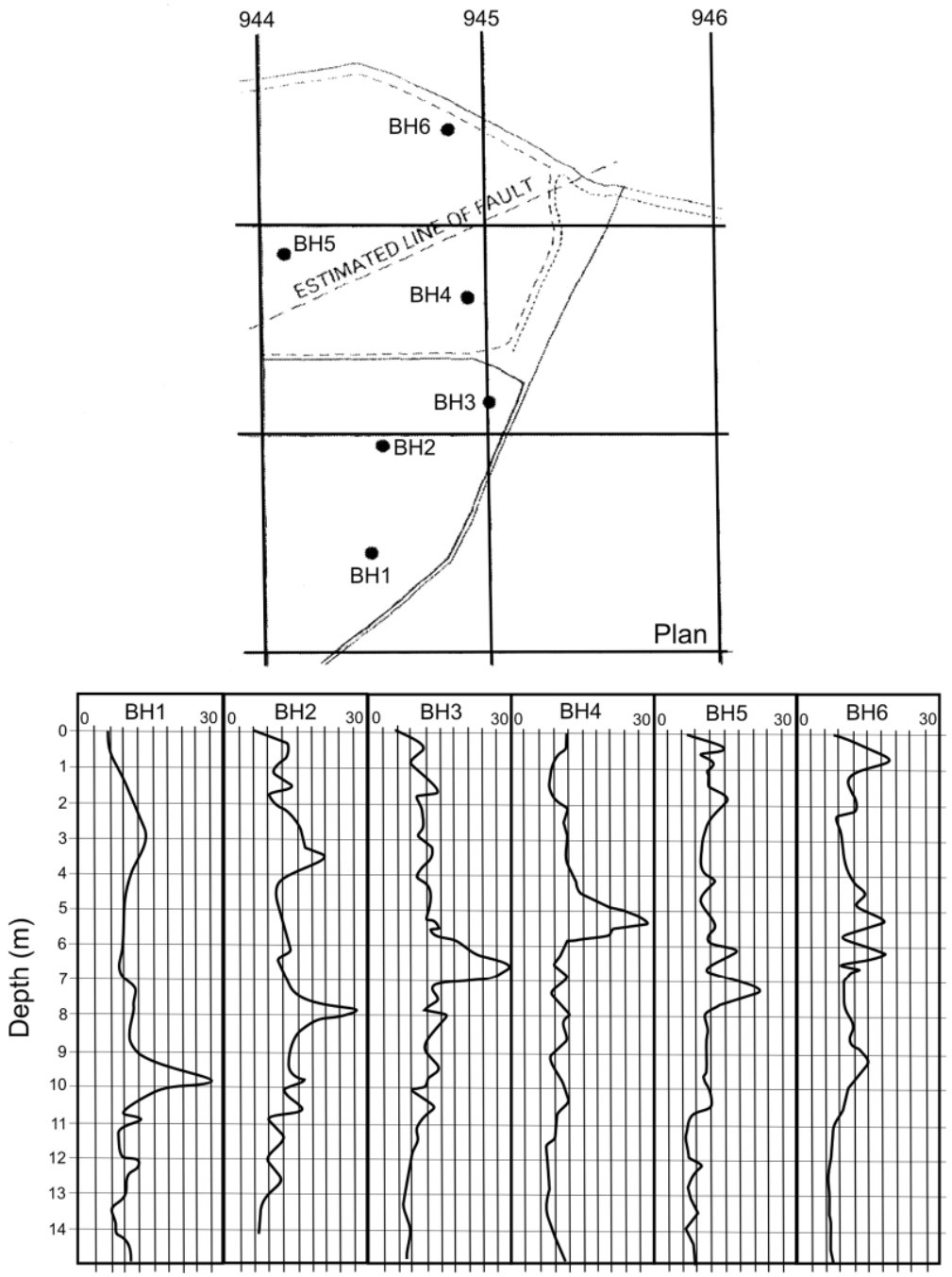


Figure 45.11 Correlation of natural gamma logs between closely spaced boreholes in an engineering site investigation
Reproduced from Reynolds (2011), John Wiley & Sons, Inc. (modified from Cripps and McCann, 2000, Elsevier)

of ±83° of the equator. In 2009, an ASTER Global Digital Elevation Model was made available at 30 m resolution. Geo-rectified images can be draped over this DEM to provide 3D perspectives (**Figure 45.12**).

Higher-resolution multi-spectral and panchromatic images are available from a number of satellite platforms, such as, for example, SPOT, IKONOS, FORMOSAT-2, KOMPSAT-2 and Quickbird, with increasing levels of ground resolution. The latest SPOT imagery (SPOT5) is available in scenes 60 km by 60 km with ground resolutions of 2.5 m, 5 m, 10 m and 20 m in panchromatic and colour, with an additional option of colour at 20 m ground resolution. Stereo imaging is also possible; from

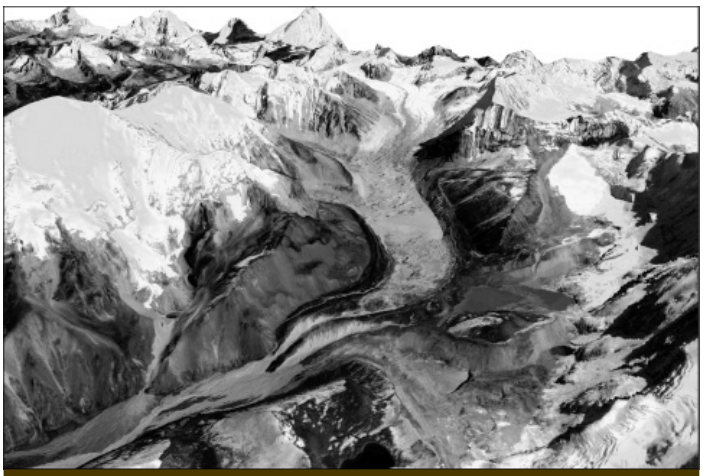


Figure 45.12 A colour ASTER image pan-sharpened and draped over a digital terrain model for a scene in Tibet (shown as grey scale)

this, digital elevation models (DEM) can be derived, although for best results, users should provide accurate ground control points within the area of coverage of the scenes to calibrate the DEM. IKONOS has been available since September 1999 with panchromatic and four-colour images at 1 m and 4 m ground resolution, respectively, and a revisit period of three days. FORMOSAT-2 has 2 m and 8 m ground resolution in panchromatic and four-colour mode, respectively, with a daily revisit capability, making it ideally suited for rapid imaging after natural disasters. KOMPSAT-2 has 1 m and 4 m ground resolution in panchromatic and colour modes with a 28-day revisit period and scene size of 15 km by 15 km. The satellite with the finest ground resolution is Quickbird, which was launched in October 2001. It achieves resolutions of 0.61 m and 2.44 m in panchromatic and four-colour modes, respectively, over a scene size of 16.5 km by 16.5 km. The EROS satellite produces panchromatic-only images over 13.5 km by 13.5 km with a 1.8 m resolution, although hyper-sampling can refine this to 1 m over an area of 9.5 km by 9.5 km.

Interpretation of images can range from photo-interpretation techniques through to software-controlled terrain analysis. Time-separated images can be used to measure rates of change by tracking the lateral movement of surface objects or through comparison of changes in surface emissivity.

45.9.2 Synthetic aperture radar (SAR)

While optical satellite imagery can be restricted by daylight and visibility limited by cloud cover, radar imaging can be used at any time and through cloud cover. A number of radar satellite imaging systems are available, such as INSAR and TerraSAR-X. The latter, for example, provides three ranges of ground resolution: *Spotlight*, ≤ 1 m over an image 10 km wide and 5 km long; *Stripmap*, ≤ 3 m, 30 km wide by 50 km long; and, *Scansar*, ≤ 18 m resolution, 100 km long by 150 km wide.

Radar sensing can be used not only to provide images of the ground covered by extensive cloud cover, but also can be used to derive changes in surface elevation arising from earthquakes or hydrocarbon exploitation, for instance, using interferometric methods. Rates of lateral movement of the ground surface can be measured through rapid repeat imagery, where flow rates as small as 2 cm per day can be resolved over Himalayan debris-covered glaciers.

45.9.3 LIDAR

Light detection and ranging (LIDAR) has now become an established, very high-resolution remote mapping tool that is predominantly deployed from aircraft but is increasingly becoming available for ground-based lateral scanning of near-vertical faces. As the survey aircraft flies over a designated survey area, a wide-aperture laser beam scans across the line of flight measuring the transit time from up to four returns from the reflective surfaces beneath, each return generating a data point within a point cloud. The transit time is converted to distance from the sensor, and, knowing the exact height and aspect of aircraft, the elevation and position of the reflective surface can be determined; thus a digital surface model (DSM) can be created. The first return may be associated with the tree canopy and vegetation, while the last return is normally associated with the ground; from this, a digital terrain model (DTM) can be derived. Intermediate returns may provide information from within the tree canopy (giving information about the species of trees, for example). Using specialist software it is possible to remove the effects of trees and buildings from a DSM to generate the DTM. This can provide detailed topographic information otherwise obscured by vegetation. LIDAR has been used highly effectively to map landslides and archaeological remains concealed beneath dense tree canopies. Ground resolution is of the order of 0.25 m and vertical topography determined to within 0.1 m, subject to survey control.

Similar technology is now available for terrestrial lateral LIDAR scanning by which detailed point clouds can be obtained within a range of ~1.2 km of steep surfaces, such as rock cliffs or man-made superstructures. Point clouds can be used to generate 3D images and also to map changes in surface elevations over time, such as for pseudo-real-time monitoring of slope stability. It is now also possible to deploy LIDAR scanners from boats; images can be combined seamlessly to produce a continuous image from the seafloor and the submerged part of marine infrastructure using high-resolution multi-beam swathe bathymetry to the exposed superstructure using boat-based LIDAR.

LIDAR images can be used to generate topographic maps, DTMs and a wide range of topographic products, as well as providing the base onto which aerial photographic or satellite images can be draped to produced 3D isometric images and animated ‘fly-throughs’. As the data produced are geo-rectified, they can be integrated into GIS databases and merged with other spatially rectified data sets in common coordinate sets. For example, LIDAR data can be integrated with airborne geophysical data sets, and airborne geomagnetic data can be

presented in colour contoured format overlain on panchromatic satellite imagery draped over a LIDAR-derived DTM.

45.9.4 Thermal infra-red (TIR)

TIR imaging can be achieved using sensors deployed on satellites and airborne or surface-mounted platforms, such as telescoping masts. The principle is based on being able to convert radiated or reflected heat into real-time pictures or images and depends on there being a detectable surface thermal contrast present, such as from a hot lava flow, exothermic methanogenesis within a landfill, an underground fire within a tyre dump, or man-made infrastructure (pipes, buildings, electric cables, etc.). For engineering applications, ground-based or aerial surveys are undertaken. Subsurface heat sources may be indicated at the ground surface through thermal conduction. Timing of surveys should be planned to enhance the thermal contrasts, such as just after dawn. While thermal imaging can be undertaken at night, it is more often undertaken in daylight in conjunction with optical imaging (photography/video) to aid interpretation.

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45.10.1 Useful web addresses

ASTER; www.asterweb.jpl.nasa.gov
Landsat; www.landsat.usgs.gov
Satellite images – SPOT; www.spotimage.com
Satellite images – other; www.infoterra.co.uk

It is recommended this chapter is read in conjunction with

- Chapter 13 *The ground profile and its genesis*
- Chapter 24 *Dynamic and seismic loading of soils*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 46
Ground exploration

John Davis Geotechnical Consulting Group, London, UK

This chapter describes the main intrusive techniques available for ground exploration. These techniques are some of the tools by which the ground profile and soil behaviour elements of the geotechnical triangle are established.

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46.1 Introduction

Ground exploration is defined as the collection of data from intrusive ground investigation techniques. Non-intrusive techniques such as geophysics or surface loading are not discussed here.

Ground exploration includes techniques to expose the ground or collect samples for visual inspection or testing, intrusive testing of the ground without collection of samples, and any subsequent installation of monitoring instrumentation in the ground. Describing and testing of recovered samples and any interpretation of recovered *in situ* or monitoring data is covered in other chapters (Chapters 49 *Sampling and laboratory testing* and 50 *Geotechnical reporting*).

The major ground exploration activities are divided into two types: those that involve intrusive means to characterise and/or sample the ground, and those involving subsequent monitoring of the ground.

When planning a ground investigation it is important to choose a suite of ground exploration techniques that are appropriate to the likely ground conditions and that also provide the data the project requires. Guidance on this is given in Chapters 43 *Preliminary studies*, 47 *Field geotechnical testing*, 48 *Geo-environmental testing*, and 49 *Sampling and laboratory testing*.

It is important to understand that each technique provides both opportunities and constraints for data collection. Data requirements range from soil or rock description, through quantifying parameters (via lab and *in situ* testing), to provision of monitoring opportunities. Other chapters provide further guidance on these issues. An alternative source of detailed advice on the applicability of the various ground investigation techniques is a specialist ground investigation contractor.

46.2 Techniques

This chapter describes the nature of the main techniques and outlines the pros and cons for:

- different types of soils and rocks;
- sampling, *in situ* testing;
- monitoring.

Techniques are divided into three classes: excavation techniques, probing techniques and drilling techniques. The list of techniques discussed is not exhaustive.

46.3 Excavation techniques

46.3.1 Trial pits

Trial pits can be hand-dug or machine-dug. Hand-dug trial pits are typically used as starter pits to identify and avoid services or to uncover details of existing shallow foundations. They are normally limited to relatively easy-to-excavate soils and to very shallow depths. The need to provide shoring and the inevitable slow rate of progress mean deeper hand-dug pits are not often specified.

Suitable ground conditions and depths for machine-dug trial pits are determined by the capacity of the machine. All natural soils are suitable, as are some weaker weathered and fractured rocks. Care must be taken in very soft soils and sands and gravels below the water table, as excavations of these materials are particularly liable to collapse. In these cases it is not usually possible to recover the required information from below the water table.

Trial pits are excellent for displaying the fabric of the ground and for obtaining large disturbed samples such as those required for some compaction tests. Generally trial pits are only used to obtain disturbed samples. Shallow (<1 m) trial pits are also often used to carry out plate bearing tests or *in situ* California bearing ratio (CBR) tests. Care should be taken to site trial pits in areas where the backfilled pit will not cause problems for any subsequent construction. It is possible to install standpipes for monitoring gas and groundwater in trial pits as they are backfilled.

46.3.2 Trial trenches

These are similar to trial pits but are long compared to their width, and useful for finding features at shallow depths that might have a limited extent in plan, such as old foundations, or systematic changes in rockhead level, such as the edges of buried cliffs.

46.3.3 Trial excavations

Trial excavations are used to investigate the feasibility of much larger excavations. A trial excavation is defined here as an excavation which contains the excavation plant.

46.4 Probing techniques

Probing techniques are those that recover information about the ground but do not recover a sample. The principal types are:

- dynamic probing;
- static cone penetration testing.

In dynamic probing, a series of rods with a standard cone-shaped shoe are driven rapidly into the ground by repeated blows of a standard-weight hammer falling through a repeated fixed distance. A penetration resistance in the form of blow count per unit length is recorded and logged, typically blows per 100 mm. Different standards exist for hammer weight and hammer drop length. More information on the various types can be found in British Standards or Eurocodes. The technique is broadly analogous to continuous standard penetration testing (SPT) (see below). However, it is important to realise the results cannot normally be used to safely generate equivalent SPT 'N' values. The technique is useful for rapid and inexpensive determination of variations in penetration resistance, such as where soft spots are thought to exist and their extent needs to be determined. The equipment used is typically light and portable and can be used in spaces with restricted access such as basements. The technique is unlikely to achieve significant penetration in stiff clays or dense granular deposits.

In static cone penetration testing, a series of rods are driven hydraulically and continuously into the ground, typically from a purpose-built heavy vehicle which provides a reaction to the hydraulic thrust. The lowest rod (the 'cone') can contain various sensors used to measure properties of the ground it passes through. The most commonly used sensors measure the resistance the ground provides to the end of the cone as it is pushed into the ground, and the frictional resistance the ground provides to the sides of the cone. A continuous and instantaneous read-out of these values is provided. These parameters can be interpreted to give an indication of the soil type and to provide various strength and stiffness parameters. These values of cone resistance and cone friction can also be used directly in some design calculations. One of the great advantages of cone penetration testing is its ability to identify very thin clay or granular layers in other strata. These can have significant impacts on geotechnical processes and are easily missed by many other ground exploration techniques.

Other types of cone can be used to give measurements of pore pressure and seismic velocity. Other varieties of cone also exist. This is discussed in greater detail in section 46.5.3.3. Cone penetration trucks can also be used to recover very small diameter continuous disturbed samples, though not at the same time as or exactly in the same location as any other cone penetration testing as described above.

The technique is applicable to most soils; achievable penetration depths depend on the strength of the soil and the reaction provided by the equipment the cone is mounted on. Typically, significant penetration is limited to clays that have a consistency less than stiff, and to granular soils whose relatively density is less than medium dense. Penetration will be greater in finer soils compared to coarse soils. Penetration in very coarse soils (gravels and coarser) will be limited regardless of relative density. Penetration is not limited by the presence of groundwater. In soft or loose soils great depths (around 40 m) can be achieved very rapidly and in these conditions a single truck-mounted cone can achieve several hundred metres of testing in a day. A typical cone 'truck' is a highway-legal road vehicle weighing approximately 18 tonnes. Lighter vehicles with large low-pressure tyres are available for very soft sites. Cone penetration equipment can also easily be mounted on pontoons or jack-up rigs for offshore work and can be skid mounted for work in restricted access sites, though in this case consideration needs to be given to provision of a reaction by the use of props, anchors or kentledge.

Neither dynamic probing nor static cone penetration testing provides for the installation of instrumentation or post-fieldwork monitoring.

46.5 Drilling techniques

46.5.1 Window sampling

A window sampler is a tube with an open slot or window down one side which is driven rapidly into the ground using high-speed percussion. Window samplers are typically 1–3 m long and less than 100 mm in diameter and may or may not contain a rigid plastic liner. The logging of the soil can be done through the window. 'Windowless' samplers are also available. In suitable self-supporting ground, they can achieve depths of 7 m or more by repeated extraction and reinsertion of progressively smaller sampler tubes using rods to extend the depth. They are generally only suitable for obtaining disturbed samples. It is sometimes possible to install standpipes for monitoring gas and groundwater using window sampling equipment. The principal advantages of window-sampling are speed and cost-effectiveness. The equipment is small and portable and can easily be used in areas with restricted access. It is often used alongside other techniques as an inexpensive way of providing a widespread and large number of sampling points for contaminated-land testing.

Window sampling of groundwater is difficult. Recovery in soils where the largest clasts, such as gravel, are similar to or larger than the internal diameter of the tube is not easy. Recovery of samples in granular soils below the water table is very difficult. High soil strengths and relative densities also restrict the depths achievable with this technique. Another drawback is that the equipment is noisy for its size.

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46.5.2 Auger drilling

In geotechnical investigation (as opposed to mineral prospecting) auger drilling normally uses hollow stem continuous flight augers. The hollow stem typically has a removable plug at the lower end and the auger is 'corkscrewed' into the ground. The material retained on the auger flights on extraction gives an approximate profile of the strata encountered, and the hollow stem and the plug can be removed from the surface to allow the use of driven samplers in the hollow stem. The technique is fast and can cope with most soil types, though sands and coarser soils generally inhibit the use of the hollow stem. Anything other than relatively shallow depths requires a powerful rig. The technique is not commonly used in geotechnical investigations and tends to be restricted to sampling for contamination testing.

46.5.3 Cable percussion boring and rotary drilling

46.5.3.1 Casing

Casing is used in both cable percussion and rotary boreholes to support the borehole walls and to seal off inflows of groundwater. It is typically formed of lengths of robust steel tubing with threaded ends allowing simple connection. A cutting shoe or bit is always present at the bottom of the lowest section of casing. Casing is not always required. Typically the base of the casing during drilling is always slightly higher than the base of the borehole. Casing is typically installed using the principal method of the rig involved – percussion for cable percussion rigs and rotary drilling techniques on rotary rigs.

46.5.3.2 Cable percussion boring

Cable percussion boring is also commonly known as shell and auger boring and is the mainstay of the UK geotechnical investigation industry. It is used routinely in most materials from soft organic soils up to very weak rocks. The term 'shell and auger' is misleading as augers are almost never used in this context in the UK.

A cable percussion rig consists of a winch with a friction clutch or brake that winds a wire rope to and fro over a pulley at the apex of two connected A-frames. The winch is used to repeatedly lift and drop a variety of steel boring tools into the ground to create the borehole and to remove the displaced soils.

A 'claycutter' is, unsurprisingly, used to advance the borehole in clay soils. It has a similar form to a window sampler in that it is a steel cylinder, open at the bottom end where it is fitted with a cutting shoe and with two open slots in the side. When this relatively heavy tool is dropped on to the base of the borehole, clay is forced up inside it. The cutter is then raised to the surface and the clay is knocked out through the shoe by inserting a metal bar through the open slots.

A shell is used to recover sands and gravels. This is a steel cylinder, open at the bottom end, where it is fitted internally with a hinged circular plate which forms a non-return valve or 'clack'. The sands and gravels are often recovered by surging the shell up and down just above the base of the hole. This draws water into the hole and loosens the soils allowing them to pass above the clack and be retained by it. Water is often added to the borehole to facilitate this process. One of the disadvantages of this technique is that the samples it produces tend to under-represent the finer-grained fractions of the sands and silt that may actually be present in the ground.

A variety of chisels are also available for breaking through hard layers.

The recovered materials are removed frequently (typically at least every 0.5 m) by dropping or knocking them from the shell or claycutter onto a board adjacent to the top of the hole. Some or all of this material is collected as both small ('jar') and bulk ('bag') samples for logging by the engineer or geologist either at the rig site or later elsewhere. These samples can also be used for laboratory index testing.

Two other important tools are routinely used: the SPT hammer, in many materials, and the U100 sampler, in clay soils.

46.5.3.3 The standard penetration test

The standard penetration test (SPT) is a commonly used *in situ* index test. It can be carried out successfully in almost all soils and in some weak rocks. The SPT equipment consists of a trip hammer which drops a standard-sized weight (65.5 kg) through a standard distance (760 mm). The hammer drives either:

- a solid rod of a standard length (610 mm) and diameter (2 ft or 50.8 mm) with a standard-shaped cone on the lower end;
- an open-ended split sample tube of a standard length and diameter with a standard-shaped cutting shoe or 'spoon' on the lower end.

The solid rod is used in rocks and soils with significant gravel or larger content, and the split spoon sampler in finer grained soils. The samples obtained in this way are taken as small disturbed samples.

The test is carried out by repeatedly dropping the hammer and recording the penetration for each of six successive penetrations of 75 mm. The lowest four sets of 150 mm blow counts are totalled to give the SPT 'N' value. Various corrections can be made to this N value to account for the effects of variations in grain size, overburden pressure and hammer calibration. N values have been correlated with many soil and rock parameters. For further information on this see section 46.6 below. Where resistance to penetration is very high, equivalent N values are often extrapolated from tests with less than the normally required penetration.

The test is not always very successful in very weak soils, where the tool can sink under its own weight or penetrate several hundred millimetres with a single blow. The test is also unreliable in soils with many larger clasts (gravel and larger), where the strength of individual clasts dominates the results. The test is very useful in some very weak rocks (such as high porosity sandstones), where it is sometimes the only source of quantitative information. The test is now considered to be an

unreliable indicator of geotechnical properties in chalk. The test can also be carried out in rotary bored holes.

46.5.3.4 The U100 sampler

The U100 sample is a low-quality undisturbed sample that is obtained by driving an open metal cylindrical tube into the soil. The tube is 450 mm long and has an internal diameter of 100 mm. The lower edge of the U100 sample tube sits within a steel cutting shoe containing a sprung core catcher, and its upper edge is connected to a driving head and rod. A sliding hammer connected to the rig winch moves over this rod and is repeatedly lifted and dropped, driving the sample tube into the ground. The material retained in the cutting shoe is usually sampled separately as a small disturbed sample. The open ends of the U100 tube are normally sealed with wax and covered with plastic end caps. The samples obtained in the U100 sample tube are commonly referred to as 'undisturbed'. This is not strictly true and if high quality samples are required for advanced triaxial testing then thin-walled push samples or samples taken from a rotary core should be used. U100 sampling is used in all types of clay soils. It is often not very successful in very soft or strong clay soils or clay soils with a high stone content. The number of blows used to fill the sample tube cannot be used in any way to correlate with SPT N values.

Eurocode 7 classifies a U100 sampler as a thick-walled sampler (see British Standard Institution, 2006, Table 3). This effectively disallows its use as a means of obtaining samples for quantitative design (a requirement for most EC7-compliant design). Modified thinner-walled U100 sampling tubes are beginning to become available and the use of these partially overcomes this problem. These sampling tubes are inevitably less robust than their thicker-walled predecessors and this can cause problems in some soils traditionally sampled with U100s such as tills and weaker clay-based rocks. In most cases EC7 compliant sampling in these soils can be achieved by using rotary coring methods.

46.5.3.5 Thin-walled and piston samplers

This type of sampler is typically used to obtain high-quality undisturbed samples in soft clays where other forms of sampling would be likely to cause excessive sample disturbance. They vary in diameter but are typically 100 mm or larger. Lengths also vary. Thin-walled samplers feature a non-return valve at the top of the sampler to allow escape of water or air during sampling. Thin-walled samplers are typically pushed continuously from the surface using a hydraulic jack or using the rig winch in conjunction with pulleys. Piston samplers are a type of thin-walled sampler. The sample tube is pushed by the drill rods into the soil over a close-fitting piston head which can be held in a fixed position by surface-operated clamps. Both types can be used with rotary rigs and cable percussion rigs. Piston samplers for use in stiff clay soils have also been developed

46.5.3.6 Rotary drilling

There are many different types of rotary drilling technique used in ground exploration. One of the main distinctions is between cored and non-cored techniques. Within the cored category there are further distinctions between conventional, wireline and sonic coring. Choice of a flushing medium is also an important consideration. The sections below give an outline of these techniques. This, however, is a complex subject and it is recommended that these choices be discussed with a prospective ground investigation contractor at an early stage of the investigation procurement process.

46.5.3.7 Flush

Most rotary drilling systems involve the use of some sort of flush (air or liquids; the liquids include muds, foams and mists). The flush is circulated by pumps down through the drill bit and back up to the surface through the annulus between the boundary of the hole and the drill 'string' (the sequence of bit core barrel and drill rod or cable in the borehole). One of the main purposes of the flush is to remove cuttings away from the bit and transport them up the hole to the surface. Another significant benefit of flush is in resisting any sub-artesian groundwater pressures encountered during drilling.

46.5.3.8 'Open hole' rotary drilling

Rotary drilling that does not involve the collection of core is often called 'open hole' drilling. In open hole drilling the bit occupies the full cross-sectional area of the borehole as it is advanced, and all the material below the bit is fragmented and passed up the borehole as cuttings by the flush. This is a very rapid drilling technique. Typical applications are those that require high metreages to be drilled but do not require the recovery of core. Examples are searching for voids, such as those found in abandoned mines, and the rapid creation of boreholes for the installation of instrumentation where the ground conditions are well known or established. Open hole techniques are also used within short lengths of borehole to pass through strata where coring techniques might struggle to progress.

The only way to log rotary open hole boreholes is by examination of 'cuttings' in the flush returns. In soils, these are likely to be minimal. In rocks, examination of the small rock chips returned in the flush gives a broad view of the nature of the strata being drilled. This examination must be done *at the rig at the time of drilling*. Voids such as coal mine workings are often identified by a loss of flush. Unworked coal seams are usually identified by the distinctive arrival of black coal chips in the cuttings.

Where boreholes are being entirely drilled using open hole techniques a system commonly known as ODEX is used. ODEX is a trade name that has come to be associated with equipment that allows the borehole casing to be advanced at the same time as the bit. Such systems often utilise 'down the hole hammers' (DTH) to increase the rate of penetration.

46.5.3.9 Rotary coring

There are three main types of rotary cored drilling: conventional, wireline and sonic. Conventional and wireline coring share many characteristics and will be discussed first. Coring of soils and rocks involves the recovery of a cylindrical 'stick' of the ground using a rotating 'core bit' and 'core barrel'. These combine to create a cylindrical tube with a cutting surface around the lower edge. Typically in geotechnical investigations most barrels are 'double' tubed. In these systems the inner barrel is decoupled from the outer rotating barrel that carries the cutting edge to minimise disturbance to the core. This system usually also incorporates a rigid 'core liner' within the inner barrel. This liner is a plastic tube in which the core is collected and retained as it is placed in the core box at the end of the core run.

Each run has its own fresh length of liner. Spring systems are used to retain the core within the barrel as it is returned to the surface at the end of the core run. Core barrels come in a variety of lengths and diameters. Typical geotechnical investigation core diameters are between 70 mm and 100 mm. The wider diameters tend to maximise the likelihood of good core recovery. Core barrels are typically less than 3 m long. Core 'run' lengths (the length of borehole drilled in a single attempt) are typically restricted to less than the barrel length.

The difference between conventional and wireline systems is speed of operation in deeper (say greater than 30 m) boreholes. In conventional rotary coring the barrel is delivered to the bottom of the hole using coupled rods; these provide a rigid means of delivering the rotary motion and the flush to the bit. If the ground conditions require the hole to be cased then a typical sequence of operations would be:

- Attach the barrel to the rotary head on the rig and lower it into the existing hole until the head is just above ground level.
- Clamp the top of the barrel, remove the head and pick up a rod, attach the rod to the top of the core barrel, unclamp it and lower the rod and barrel until the head is just above ground level.
- Repeat until the bit is at the base of the hole.
- Turn on the flush pumps, core a length of the ground (the 'run'), then repeat the sequence in reverse to remove and empty the barrel.
- Add a length of casing to the top of the casing string and drill this down to just above the base of the new hole created by the barrel.
- Repeat to the specified base of the hole.

In a wireline system a length of casing is added at the top of the borehole and the barrel is lowered into the casing using a cable. At the base of the hole the barrel locks into the lowest section of casing and the casing is used to delivering the rotary motion and the flush to the bit. When the run is complete the barrel is unlocked from the casing and withdrawn using the cable; this is much faster than the constant connection and disconnection of rods in the conventional method.

Sampling of rock core for testing is straightforward for most competent rocks: the samples can be taken directly from the core liner or core box. Samples of rocks that are likely to dry out prior to strength testing (most rocks) should be protected from moisture content change by immediately wiping away excess drilling fluids, wrapping in clingfilm, and waxing. It is even more important that similar treatment be given to any core samples of overconsolidated clays required for anything other than simple index testing as soon as possible after the core is removed from the barrel.

SPT testing is possible in rotary cored or open holes, though the process can be time-consuming compared to normal rates of drilling. Incorporation of SPT testing can be one of the few reliable sources of information in some weak rocks, particularly weak porous sandstones, which tend to be destroyed by the coring process and are often recovered as sand. Thin-walled push samplers, used for obtaining high-quality samples of clays, can also be used in suitable soils by most rotary rigs.

46.5.3.10 Sonic coring

Sonic coring is an entirely different arrangement. It is a rapid coring technique capable of achieving great depths and continuous complete recovery through almost any material. The technique utilises a high frequency vibration either with or without rotation to advance what is typically a long single-tube core barrel (rigid liners can be used in some circumstances). No flush is used. As with the techniques discussed above, the drilling technique can be adopted to provide opportunities for conventional coring, SPT testing and piston or U100 sampling. The technique is not often used in geotechnical investigations but is increasingly used for installation of geotechnical instrumentation during construction. A number of constraints limit the use of the technique in geotechnical investigations:

- The high-frequency vibrations produce high sample temperatures and densify some soils; this means that any solid core samples recovered by this technique in soils or weak rocks should be considered disturbed from a laboratory testing point of view. The vibrations and temperatures can make even simple laboratory index test results unreliable in many soils.
- The vibrations can produce rock cores with a low solid core content compared to conventional rotary coring in the same materials.
- The use of single barrels and the lack of flush (to balance groundwater pressures) mean that significant quantities of information can be lost when coring strata that contain groundwater under artesian and sub-artesian pressures.
- The cores are typically removed from the barrel by the use of vibration, increasing the potential for disturbance.

46.6 *In situ* testing in boreholes

This topic is discussed in detail elsewhere. The notes below relate only to some of the practical issues involved in utilising these techniques.

Dynamic probing, static cone penetration testing and standard penetration tests are all forms of *in situ* testing and are all discussed above.

46.6.1 Permeability testing

Open boreholes, partially and fully cased boreholes, standpipes and standpipe piezometers are all potentially suitable for carrying out variable head permeability tests. Inflow tests using single or double packers (lengths of hollow rod surrounded by a rubber membrane that can be inflated to seal off a section of open borehole) can be carried out in stable open rotary cored boreholes.

46.6.2 Vane testing

The borehole vane test is directly analogous to the laboratory shear vane test. A large cruciform vane is lowered into the ground below the base of the borehole (typically into soft clays) and rotated, with the torque required to rotate the vane being recorded; the data can be converted into a value of undrained shear strength. The procedure is typically carried out using a cable percussion rig.

46.6.3 Plate bearing tests

It is possible to carry out plate bearing tests in boreholes. There are, however, many practical difficulties involved with obtaining a clean borehole base and continuous contact between the base of the hole and the inserted plate. Such tests are rarely carried out.

46.6.4 Pressuremeters and dilatometers

Pressuremeters and dilatometers are devices which use expanding membranes to measure strength stiffness and *in situ* stresses in the ground. Most of these instruments are utilised by lowering them into a test section of borehole that has been rotary cored and that is of a diameter only slightly larger than the instrument. Tight tolerances between the instrument and hole are needed to minimise the amount of expansion the instrument undergoes before contacting the borehole wall. As a result materials that are prone to collapse or swell if the borehole is left unsupported can be difficult to test in this way. These problems can be overcome by the use of a 'self-boring' pressuremeter. Strata that tend to produce rough irregular borehole walls can puncture the instrument's membranes. The pressure capacity of the instrument needs to be broadly matched to the strengths and stiffnesses that are to be measured.

46.7 Monitoring installations

46.7.1 Groundwater

Measurement of groundwater levels and pressures is a key objective of almost all geotechnical investigations. Successful design of groundwater instrumentation requires some knowledge of basic hydrogeology, in particular the distinctions between unconfined and confined aquifers and the difference between groundwater 'levels' and groundwater pressures (see Chapters 15 *Groundwater profiles and effective stresses*, 16 *Groundwater flow*, and 80 *Groundwater control*).

There are two main types of instrumentation for measuring groundwater: standpipes and piezometers. There are also several different types of piezometer, which are described below. A common feature of all types is an area within the instrument which allows groundwater to enter the instrument or which responds to groundwater pressure: this area is placed in the borehole where groundwater information is required (the response zone).

All piezometers are installed in boreholes in a similar way. The borehole is backfilled, usually with bentonite cement grout, to the base of the response zone. A permeable material, the nature of which varies with the type of instrument, is then backfilled around the instrument throughout the length of the response zone. Another bentonite cement grout seal is added above the response zone. The location of this upper seal in relation to the groundwater level varies according to the type of instrument.

46.7.1.1 Standpipes

Standpipes measure groundwater levels in unconfined aquifers (i.e. in relatively permeable strata). They typically consist of a tube with a diameter between 19 and 50 mm with a slotted section that is placed within the response zone. In a standpipe the response zone and the slotted section extend above the unconfined groundwater level. The groundwater level measured in the tubing directly replicates the groundwater level in the ground. Groundwater level is measured using a dipmeter. This is a tape measure containing wires connected to a battery and a buzzer at the surface. At the other end of the tape is a sensor. When the sensor breaks the surface of the water in the tube an electrical circuit is made and the buzzer sounds. The depth to groundwater can be read off the tape.

Measurements of ground gas composition are also a routine part of ground investigations. Standpipes of 50 mm diameter are typically used for this purpose. They are installed with continuous slotted sections above the water table and an upper bentonite seal close to the ground surface. The top of the tube is fitted with a bung containing a gas tap to which a gas analyser and flow meter can be attached

46.7.1.2 Piezometers

Piezometers measure groundwater levels or pore pressures in specific zones within the borehole. Zones are targeted for measurement on the basis of the aims of the investigation and the observations made during drilling. There are several types of piezometer.

46.7.1.3 Standpipe piezometers

Standpipe piezometers are the commonest and simplest form of piezometer. They typically consist of 19 or 24 mm tubing that is fitted with a porous 'element' of known permeability

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(typically 10^{-4} m/s) at its base; this element varies in length but is typically < 300 mm long in UK ground investigation practice. The porous element is installed in a response zone filled with sand. This sand filter is isolated in the borehole by bentonite seals above and below. The porous element allows water in the ground to enter the piezometer tubing. The height that the water reaches in the tubing reflects the pore pressure present in the response zone. If correctly located the bentonite seals either side of the response zone isolate the pore pressure in the response zone from any other different pressures that may be present in the ground.

Practical considerations involved in achieving seals above and below response zones mean that it is unusual to install more than two standpipe piezometers in a borehole.

Standpipe piezometers can be used in all types of ground. However, in low permeability materials the height of water in the tubing can take a very long period of time to come to equilibrium with the pore pressures in the ground. Standpipe piezometers can be used to carry out variable head permeability tests. Water level in the tubing is measured using a dipmeter as described above.

46.7.1.4 Other types of piezometers

In a pneumatic piezometer twin pneumatic tubes run from the piezometer tip to the surface. The top of the piezometer contains a diaphragm which separates water pressure in the ground and gas pressure fed down one of the tubes from the surface. Gas pressure is provided by bottled nitrogen. Water pressure readings are obtained by adjusting the surface gas pressure with the water pressure. When they are equalised, gas is forced into the second tube and this is detected at the surface and the pressure recorded. These systems typically form part of long-term datalogged monitoring systems where a single datalogger and gas control system can be set up to log several boreholes. A pneumatic piezometer can be used to measure short-term negative pore pressures.

In a vibrating wire piezometer a tensioned wire in the piezometer tip is made to vibrate using an electromagnetic field. The tension in the wire varies according to external water pressure on a diaphragm in the tip. This varies the frequency at which the wire vibrates. The variation in this frequency can be measured and calibrated in units of water pressure. This system is suitable for use with automatic dataloggers or with hand-held readout units. The prinicipal advantage of vibrating wire piezometers is that they come to equilibrium very rapidly in low permeability soils. This allows them to be installed directly in cement bentonite grouts rather than in conventional sand filters. Vibrating wire piezometers cannot reliably measure negative pore pressures and can become unreliable for measuring positive pressure if they are exposed to negative pressures. Great care is needed in their installation.

Pumping wells in ground investigations are installed with an associated array of monitoring piezometers when there is a need to carry out a pumping test. This test provides information on permeability, transmissivity and storage within an aquifer and is required when a project involves dewatering. The key differences between a pumping well and a piezometer are usually greater diameter (to permit submersible pump access) and the presence of a well screen. A well screen is normally used to line the borehole in the region where the water is to be pumped from, while the rest of the hole is normally lined with impervious casing. The screen's purpose is to permit passage of water from the ground to the hole whilst at the same time supporting the sides of the hole and controlling the quantity of material drawn into the hole by the flowing water. Successful screen design can be complex and consideration should be given to seeking expert advice before carrying out installation and testing.

46.7.2 Movement and load monitoring

46.7.2.1 Monitoring horizontal movement

Inclinometers are used to measure and monitor horizontal movement within the ground. This need typically arises where there are slope instability concerns or where excavations and retaining structures are being monitored. An inclinometer installation in a borehole takes the form of a circular tube installed to the full depth of the borehole, where the bottom of the borehole is well below the expected zones of movement. The annulus between the borehole and tubing is filled with grout and the lower end of the tubing is closed so the inside of the tubing remains full of water and/or air. The inside of the tubing contains four grooves or 'keyways' at 90° intervals. The lengths of tubing are connected using external sleeves in such a way as to ensure that the four keyways are continuous throughout the borehole and that each keyway is in the same orientation.

As the ground moves, the inclinometer tubing bends with it. The degree and location of the distortion of the tube is measured using an inclinometer. This is a metal 'torpedo' containing two pairs of wheels on sprung arms at either end of the torpedo. Both pairs of wheels are in the same plane. The wheels sit in a pair of the keyways in the tubing and the sprung arms hold the torpedo centrally in the tube. The torpedo contains a series of accelerometers oriented in the plane of the wheels and also at 90° to this. These accelerometers measure information on the inclination of the torpedo and these inclinations can be used to calculate horizontal displacement of the inclinometer tubing. In use the torpedo is lowered to the base of the tubing using its data cable. It is then pulled up 0.5 m at a time with a reading taken every 0.5 m and this information is usually recorded on a datalogger. The whole exercise is repeated with the torpedo turned through 90° and inserted in the remaining pair of keyways. The resulting data are processed using specialised software to give a profile of horizontal movement against depth. As this is repeated through a programme of regular monitoring a picture of displacement versus depth versus time is built

up. Inclinometers that are used horizontally are also available. These are occasionally used to check settlement profiles in earthworks embankments.

46.7.2.2 Monitoring vertical movement

Extensometers are used to measure vertical movement within the ground. The vertical movement can be heave or settlement. Typically a borehole installation involves the placing of a central access tube similar to inclinometer tubing to below the zone of likely movement. A series of magnetic 'spiders' are slid over this tubing at intervals during the grouting of the borehole /tubing annulus and grouted into place. As the ground moves up or down the 'spider' legs are held by the grout and ground and move up or down relative to the tube. The location of the spiders relative to the tubing is measured using a probe lowered down the central access tube.

Deep datum installations are also used to monitor the movement of the ground surface relative to a fixed point in the ground below any zones of movement. This is typically required where settlement or heave might be generated over a wide area and a reliable survey datum might otherwise have to be far from the site. These installations involve anchoring the base of a stainless steel or fibreglass bar into the ground at depth and isolating the remainder of the bar from the ground and borehole grout by some means, typically using an inner casing or sleeve. The top of the bar at ground level is then used as a datum for conventional topographic surveying techniques.

Monitoring of loads caused by soil or rock fill is often a part of earthworks operations rather than ground investigations. Plate-like pressure cells, typically vibrating wire pressure cells, are installed at the points of interest and wired to remote dataloggers for long-term monitoring. Such cells can be installed vertically behind retaining structures to verify active and passive pressure design assumptions.

46.8 Other considerations

Many of the issues identified in the desk study and walkover survey can have a considerable impact on the conduct of a ground investigation. Some of these are listed below:

- access for rigs;
- working space for rigs;
- location of underground services;
- water and power supplies;
- underground and overhead obstructions;
- soft ground in relation to stability of investigation plant;
- many drilling rigs find operating on sloping ground difficult;
- railway work has many unique bureaucratic requirements and access issues;
- overwater and inter-tidal work often requires special plant to enable access.

46.9 Standards

The UK standard that applies to execution of ground investigations is Eurocode 7, Part 2 (British Standards Institution, 2007). The previous standard, BS 5930:1999 (British Standards Institution, 2010), has undergone amendment and will later undergo revision. It is being retained as a normative reference.

There are a large number of 'attachments' to Eurocode 7, Part 2. These are listed below. At the time of writing many of these had not been published and some parts of older British standards still applied. This situation is complex and regular checks should be made with the British Standards Institution as to the current status of these documents.

46.9.1 Ground investigation and testing: sampling methods and groundwater measurement

BS EN ISO 22475-1	Part 1: Technical Principles for Execution
BS EN ISO 22475-2	Part 2: Qualification Criteria for Enterprises and Personnel
BS EN ISO 22475-3	Part 3: Conformity Assessments of Enterprises and Personnel by Third Parties

46.9.2 Ground investigation and testing: field testing

BS EN ISO 22476-1	Part 1: Electrical Cone and Piezocone Penetration Tests
BS EN ISO 22476-2	Part 2: Dynamic Probing
BS EN ISO 22476-3	Part 3: Standard Penetration Test
BS EN ISO 22476-4	Part 4: Menard Pressuremeter Test
BS EN ISO 22476-5	Part 5: Flexible Dilatometer Test
BS EN ISO 22476-6	Part 6: Self Boring Pressuremeter Test
BS EN ISO 22476-7	Part 7: Borehole Jacking Test
BS EN ISO 22476-8	Part 8: Full Displacement Pressuremeter Test
BS EN ISO 22476-9	Part 9: Field Vane Test
BS EN ISO 22476-10	Part 10: Weight Sounding Test
BS EN ISO 22476-11	Part 11: Flat Dilatometer Test
BS EN ISO 22476-12	Part 12: Mechanical Cone Penetration Test
BS EN ISO 22476-13	Part 13: Plate Loading Test

46.9.3 Ground investigation and testing: geohydraulic tests

BS EN ISO 22282-1	Part 1: General Rules
BS EN ISO 22282-2	Part 2: Water Permeability Test in Borehole without Packer
BS EN ISO 22282-3	Part 3: Water Pressure Test in Rock
BS EN ISO 22282-4	Part 4: Pumping Tests
BS EN ISO 22282-5	Part 5: Infiltrometer Tests
BS EN ISO 22282-6	Part 6: Closed Packer Systems

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46.9.4 Ground investigation and testing: laboratory testing

BS 1377:1990 Parts 1–8 remain in force until further notice (this situation should be checked with the BSI).

46.9.5 Ground investigation and testing: identification and classification of soil

BS EN ISO 14688-1:2002 Part 1: Identification and Description

BS EN ISO 14688-2:2004 Part 2: Principles for a Classification

46.9.6 Ground investigation and testing: identification and classification of rock

BS EN ISO 14689-1:2003 Part 1: Identification and Description of Rock

46.9.7 Other standards

The British Drilling Association (BDA) runs accreditation schemes for drillers in the UK. The use of accredited drillers has been a common contractual requirement in UK ground investigation practice for a number of years. The BDA also publishes a number of best practice guides related to the execution of ground investigations. Details of these documents can be found on their website.

The Institution of Civil Engineers (ICE) publishes a widely used standard specification (Site Investigation Steering Group, 1993) and standard form of contract for ground investigations (ICE & CECA, 2003).

The Association of Geotechnical Specialists (AGS) has produced a standard format for production of digital geotechnical investigation data (so-called 'AGS data'). This has been widely adopted in the UK, and a number of commercial software packages exist for the manipulation of this data format. Use of such a package can greatly speed the interpretation of geotechnical investigation results. Further details can be found on the AGS website.

46.10 References

British Standards Institution (1990). *Methods of Test for Soils for Civil Engineering Purposes (Parts 1–9)*. London: BSI, BS 1377.

British Standards Institution (2002). *Geotechnical Investigation and Testing: Identification and Classification of Soil – Identification and Description*. London: BSI, BS EN ISO 14688-1.

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British Standards Institution (2005b). *Geotechnical Investigation and Testing: Field Testing – Standard Penetration Test*. London: BSI, BS EN ISO 22476-3.

British Standards Institution (2006). *Geotechnical Investigation and Testing: Sampling Methods and Groundwater Measurements – Technical Principles for Execution*. London: BSI, BS EN ISO 22475-1:2006.

British Standards Institution (2007). *Eurocode 7: Geotechnical Design – Ground Investigation and Testing*. London: BSI, BS EN1997-2:2007.

British Standards Institution (2009). *UK National Annex to Eurocode 7: Geotechnical Design – Ground Investigation and Testing*. London: BSI, UK NA to BS EN1997-2.

British Standards Institution (2009). *Geotechnical Investigation and Testing: Field Testing – Mechanical Cone Penetration Test (CPTM)*. London: BSI, BS EN ISO 22476-12.

British Standards Institution (2010). *Code of Practice for Site Investigations*. London: BSI, BS 5930:1999+A2.

ICE & CECA (2003). *ICE Conditions of Contract Ground Investigation Version*, second edition. London: Thomas Telford.

Site Investigation Steering Group (1993). *Specification for Ground Investigation*. London: Thomas Telford (new edition 2011).

46.10.1 Useful websites

Association of Geotechnical Specialists (AGS) publications; www.ags.org.uk/site/publications/pubcat.cfm

British Drilling Association (BDA); www.britishdrillingassociation.co.uk/

British Standards Institution (BSI); http://shop.bsigroup.com/en/

BSI Committee B/526/3 Site investigation and ground testing (lists standards proposed and in development in this area); http://standardsdevelopment.bsigroup.com/Home/Committee/50001991?type=m&field=Ref

It is recommended this chapter is read in conjunction with

- Chapter 13 *The ground profile and its genesis*
- Chapter 95 *Types of geotechnical instrumentation and their usage*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 47
Field geotechnical testing

John J. M. Powell Independent consultant and Geolabs Ltd, formerly BRE, Watford, UK
Chris R. I. Clayton University of Southampton, UK

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Information relating to ground conditions can be obtained in a number of ways, for example through remote sensing, boring and drilling, and sampling and laboratory testing. This chapter deals with common methods of testing *in situ* using penetration testing, vane testing, plate load testing, pressuremeters and *in situ* permeability tests. Equipment and test methods are briefly described, and potential difficulties and corrections noted. References to further information, sources of interpretation methods and to standards are given.

47.1 Introduction

Before geotechnical design can start, the engineer needs to collect and generate a wide range of information on site ground conditions. This is typically done through a walkover survey and desk study, by making boreholes, taking samples and testing them in the laboratory, and through field testing. BS EN 1977-2 (British Standards Institution, 2007b) and BS 5930 (British Standards Institution, 2010) clearly outline the best practice in planning this process. This chapter deals with some of the field geotechnical testing options available to those planning investigations. This is an area in which a number of standards and specifications are anticipated to be published in the next few years; engineers need to check the current status of the standards or specifications that they are using.

Field geotechnical testing makes a number of vital contributions to the site characterisation and ground modelling process, through:

- index and classification information;
- profiling;
- parameter determination.

Whilst data for this can be obtained from laboratory tests or from field geotechnical tests, the use of both types of test is generally advisable.

The engineer should carefully consider what tests to use and when. **Table 47.1** (Lunne *et al.*, 1997; their Table 1.1) and Table 2.1 in BS EN 1997-2 (British Standards Institution, 2007b) give useful general guidance to the parameters that might be obtained from various field tests and also the ground conditions they might be used for. Field geotechnical testing can have the following advantages:

- It can be used when sampling is impossible (e.g. in granular soils), or where sampling disturbance will significantly affect laboratory test results.
- It can test larger volumes of the ground than is possible in a laboratory test.
- The ground is tested as it exists in the field, i.e. at the *in situ* effective stress level, and with a representative particle size and fracture distribution.
- A near continuous record can be obtained with depth from some tests, allowing classification, and the development of a very good soil profile.
- *In situ* tests are often faster than sampling and laboratory testing, and may therefore be cheaper.

On the other hand, field geotechnical testing can have some significant disadvantages:

- In many tests, the soil type being tested is not seen, and cannot be described.
- Results may be affected by test rates that are often high, compared with laboratory loading rates, and especially when compared with construction rates of loading.
- Boundary conditions with respect to stress, strain and drainage are often poorly controlled, limiting the applicability of such tests (for example, to the determination of undrained strength or stiffness parameters).
- Deformation of the soil around an *in situ* test may be dissimilar to that around the proposed construction (e.g. horizontal, rather than vertical, under a foundation), and coupled with non-uniform imposed stress and strain fields, and lack of drainage control, this may make interpretation uncertain.
- Some disturbance may occur during boring, where this is necessary, or during insertion of any apparatus.

In general, it is sensible to determine key parameters using a combination of laboratory and field geotechnical testing and this approach is consistent with that in BS EN 1997-2 (British Standards Institution, 2007b).

The adoption of the geotechnical Eurocodes and associated standards should lead to greater harmonisation of testing and ground investigation (GI) practices in the future (see Chapter 10 *Codes and standards and their relevance*). In the sections below reference will be made to both current and forthcoming EN (European) standards; in the latter case many of the documents

		Soil parameters													Ground type						
Group	Device	Soil type	Profile	u	φ'[1]	s_u	I_D	m_v	c_v	k	G_o	σ_h	OCR	σ-ε	Hard rock	Soft rock	Gravel	Sand	Silt	Clay	Peat
Penetro-meters	Dynamic	C	B	-	C	C	C	-	-	-	C	-	C	-	-	C	B	A	B	B	B
	Mechanical	B	A/B	-	C	C	B	C	-	-	C	C	C	-	-	C	C	A	A	A	A
	Electric (CPT)	B	A	-	C	B	A/B	C	-	-	B	B/C	B	-	-	C	C	A	A	A	A
	Piezocone (CPTU)	A	A	A	B	B	A/B	B	A/B	B	B	B/C	B	C	-	C	-	A	A	A	A
	Seismic (SCPT/ SCPTU)	A	A	A	B	A/B	A/B	B	A/B	B	A	B	B	B	-	C	-	A	A	A	A
	Flat dilatometer (DMT)	B	A	C	B	B	C	B	-	-	B	B	B	C	C	C	-	A	A	A	A
	Standard Penetration Test (SPT)	A	B	-	C	C	B	-	-	-	C	-	C	-	-	C	B	A	A	A	A
	Resistivity probe	B	B	-	B	C	A	C	-	-	-	-	-	-	-	C	-	A	A	A	A
Pressure-meters	Pre-bored (PBP)	B	B	-	C	B	C	B	C	-	B	C	C	C	A	A	B	B	B	A	B
	Self-boring (SBP)	B	B	A[2]	B	B	B	B	A[2]	B	A[3]	A/B	B	A/B[3]	-	B	-	B	B	A	B
	Full displacement (FDP)	B	B	-	C	B	C	C	C	-	A[3]	C	C	C	-	C	-	B	B	A	A
Others	Vane	B	C	-	-	A	-	-	-	-	-	-	B/C	B	-	-	-	-	-	A	B
	Plate load	C	-	-	C	B	B	B	C	C	A	C	B	B	B	A	B	B	A	A	A
	Screw plate	C	C	-	C	B	B	B	C	C	A	C	B	-	-	-	-	A	A	A	A
	Borehole permeability	C	-	A	-	-	-	-	B	A	-	-	-	-	A	A	A	A	A	A	B
	Hydraulic fracture	-	-	B	-	-	-	-	C	C	-	B	-	-	B	B	-	-	C	A	C
	Cross-hole/ down-hole/ surface seismic	C	C	-	-	-	-	-	-	-	A	-	B	-	A	A	A	A	A	A	A

Applicability: A = high; B = moderate; C = low; – = none
1. φ' depends on the soil type.
2. Only when a pore pressure sensor is fitted.
3. Only when a displacement sensor is fitted.
Soil parameter definitions: u is the *in situ* static pore pressure; φ' is the effective internal friction angle; s_u is the undrained shear strength; m_v is the constrained modulus; c_v is the coefficient of consolidation; k is the coefficient of permeability; G_o is the shear modulus at small strain; σ_h is the horizontal stress; OCR is the overconsolidation ratio; σ-ε is the stress–strain relationship and I_D is the density index.

Table 47.1 The applicability and usefulness of *in situ* tests
Reproduced from Lunne *et al.* (1997); Taylor & Francis Group

are finalised but publication has yet to happen. Once they are published then they will replace any existing British Standards.

47.2 Penetration testing

Penetration testing is widely used in geotechnical practice, and can provide valuable data in almost all ground conditions. Penetration testing can be classified in a number of ways, e.g.

- dynamic (impact driving) vs quasi-static (pushing);
- requiring a borehole, or not;
- 'continuous' or intermittent.

47.2.1 The Standard Penetration Test

The Standard Penetration Test (generally known as the 'SPT') is probably the most widely used geotechnical test in the world. At the time of writing it is standardised in the UK in BS EN 1997-2:2007 (British Standards Institution, 2007b) and BS EN ISO 22476-3 (British Standards Institution, 2005b) – note that there are amendments available to both these documents. In the USA it is standardised in ASTM D1586 (ASTM, 2008a).

The test is dynamic, and intermittent, being carried out in a borehole, typically at 1.5 m intervals of depth. When the

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required test depth has been reached, a standard 50 mm outside diameter 'split-spoon' penetrometer (see **Figure 47.1**) is lowered to the bottom of the borehole on rods, and is driven into the soil using repeated blows of a 63.5 kg weight falling through 760 mm (note that this test is used worldwide, and that the exact dimensions may differ slightly in individual country specifications, the adoption of the EN ISO standard should bring practices into closer alignment). The SPT *N* value is the number of blows required to achieve a penetration of 300 mm, after an initial seating drive of 150 mm; corrections can then be applied to obtain a 'corrected' *N* value for a standard hammer energy and (in coarse-grained soil) overburden pressure (see below).

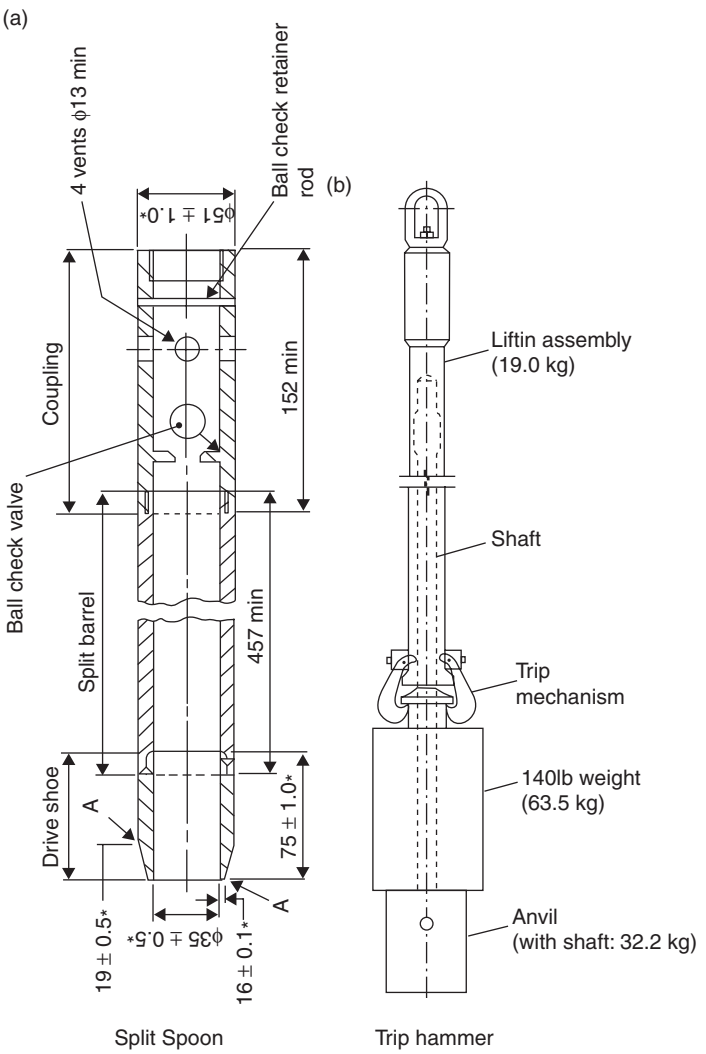


Figure 47.1 Standard Penetration Test (SPT): split-spoon sampler (left) and automatic trip hammer (right). Corners marked 'A' may be 'slightly' rounded

(Left) Reproduced with permission from BS 1377-1 © British Standards Institution 1990; (Right) Reproduced with permission from CIRIA R143 (Clayton, 1995)

Despite widespread criticism the SPT continues to be popular worldwide because:

1. It uses simple equipment.
2. It can be used in almost any ground conditions (including clays, sands, coarse granular soil and fractured weak rock).
3. It provides a sample, allowing basic visual identification of the ground in which it is conducted.

It can be used, through empirical correlations, to estimate many of the strength and stiffness parameters needed for design.

Apart from the ground conditions in which the test is made, the result of the Standard Penetration Test is influenced by three main groups of factors:

- The drilling or boring technique used to reach the test depth;
- The SPT equipment used for the test;
- The test procedure.

The most significant of these factors are described below (the wider adoption of the EN ISO standard, if adhered to, should improve the situation; however, these factors need to be considered when using historical information). For a more complete review see Clayton (1995) or Clayton *et al*. (1995).

In coarse-grained soil, *borehole disturbance* can be very large. Experience has shown that UK practice, using a 'shell' and light percussion boring can reduce the SPT *N* value by a factor of up to 5. Hollow-stem auger drilling, when used badly, is thought to be capable of loosening the soil so that *N* is reduced by a factor of about 3. The SPT *N* value can also be increased through the compaction of granular soil ahead of the drilling tools. Control of soil loosening can be achieved by:

1. Ensuring that the borehole is full of water at all times, so that piping does not occur as a result of the upward inflow of groundwater;
2. Using a small diameter borehole, so that the depth of borehole disturbance (typically about 1.5 to 3 times the borehole diameter) is restricted, and preferably less than the 150 mm seating drive (BS ISO EN 22746-3 requires that the borehole diameter is always reported).
3. Avoiding the use of a casing (for example, by using drilling mud to support the bottom of the borehole), thus reducing the concentration of groundwater inflow.
4. Reducing the vigour with which the driller advances the borehole, for example, by restricting the rate at which the hollow-stem auger plug is pulled from the centre of the auger, immediately prior to the SPT.

In most clays where SPT is appropriate, it is considered that the borehole disturbance will not be as severe as in silts and sands. The effects may be large when testing in soft and

sensitive soil, but the SPT should be considered inappropriate in such materials due to the potential for disturbance and also as it will yield too low an *N* value to be of great use in design.

SPT *equipment* is known to have a number of important effects, the two most significant of which are due to:

- the hammer design and use;
- the split-spoon design.

A wide variety of hammer designs are used across the world and most can be used in a variety of ways. They will deliver different amounts of energy per blow, and can be more or less consistent, blow-by-blow, depending upon the design. Since the SPT *N* value is inversely proportional to the energy delivered per blow, this means that it will vary according to hammer type and use. Therefore, the energy imparted by the SPT hammer must be measured and reported. BS EN ISO 22476-3 (British Standards Institution, 2005b) requires that all SPT hammers are calibrated regularly. The International Reference Test Procedure (IRTP) (ISSMGE, 1999) suggests that SPT *N* values should be corrected to 60% of the free-fall energy, N_{60}, and this is now considered in British practice through BS EN ISO 22476-3 (British Standards Institution, 2005b). It has been estimated (Clayton, 1995) that hammer energy variations around the world can lead to variations in the *N* value of -25% to +35%.

In the UK it is routine practice to use an automatic trip hammer with a relatively large anvil. Measurements suggest that this will typically deliver about 70% of the free-fall energy, producing an *N* value of the order of 15% lower than that of the 60% standard energy. Reading *et al.* (2010) show quite clearly the wide variations in energy imparted by 'standard' hammers in the UK, in some cases simply due to lack of maintenance but also related to equipment type.

When very short rod lengths are used between the hammer and the split spoon there will be some reduction in energy transfer. BS EN ISO 22476-3 (British Standards Institution, 2005b) suggests that in sands a correction to the *N* value should be considered.

The design of the split spoon can also have an effect on penetration resistance, if varied from standard. The user should be wary of two particular variations. In a few countries (including the UK) the practice has been to substitute a 60° solid cone for the split-spoon cutting shoe, when testing in gravel. Evidence from the UK suggests that in sands this can approximately double the measured penetration resistance. The use of a solid cone should be avoided if possible, and if used should be clearly reported as an SPT(C) test in accordance with BS EN ISO 22476-3 (British Standards Institution, 2005b).

Another potential problem occurs where, as in some countries, the split spoon is fitted with a liner, to allow easy sample collection. The omission of a liner from a spoon designed to take one has been found to cause a decrease in the *N* value of about 10–30%.

The *test procedure* is fairly uniform in the UK, where blow counts are typically recorded by the rig foreman for six 75 mm sections of drive (the first two for the seating and the remaining four are the test drive) – an amendment to BS EN ISO 22476-3 (British Standards Institution, 2005b) originally stipulated 150 mm drive lengths but now allows for 75 mm drives, as per UK practice. However, procedures can vary when testing either in loose or weak soil or in dense soil and weak rock. In the former case, penetration under the self-weight of the rods and hammer historically may or may not have been recorded but now, under BS EN ISO 22476-3, it has to be reported. In the latter case, the full 450 mm of penetration may not be achieved, and in this case it is now stated in BS EN ISO 22476-3 (British Standards Institution, 2005b) that it is acceptable to report the number of blows and the penetration achieved; a clear note identifying this variation is required in the report.

As with other *in situ* penetration tests, the SPT is useful for profiling, for classification and for parameter determination. When used for profiling, ideally more tests should be carried out than is common in UK practice, say at 1 m centres. As with the dynamic penetration test (described below), the SPT split spoon could be driven continuously, but this is not normal, as friction and adhesion on the drive rods tend to increase penetration resistance at depth. Classification is possible through the use of a sample (albeit of very low quality in clays, and unable to provide representative gradings in coarse granular soils) combined with the measured penetration resistance. This is based upon the relative density in granular soils, the consistency in fine-grained soils and the strength in weak rocks (see Clayton, 1995, for example).

The SPT is a small-diameter dynamic, rapid test to failure. The principal *soil properties affecting the SPT N* value are therefore:

- effective angle of friction (granular soils);
- relative density (granular soils);
- effective stress level (granular soils);
- grain size (coarse granular soils and silty granular soils);
- undrained shear strength (cohesive soils);
- cementing (weak rocks, granular soils);
- jointing (weak rocks).

Because interpretation depends upon the soil type, this must be known. Fortunately, as noted above, in most ground conditions a small sample will be retrieved from the split spoon.

The effective stress level needs to be taken into account when interpreting tests carried out in granular soil. For example, an SPT carried out from the original ground level would be expected to be higher (and sometimes very considerably higher) than one carried out in a general excavation (below

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the original ground level) during construction, as a result of the reduction in the vertical effective stress that has occurred. Modern practice is to correct N as measured in granular soils to its equivalent value under a vertical effective stress of 100 kPa (1 ton/ft^2), which is termed 'N_1'. N corrected for both hammer energy and for effective overburden pressure is termed '$(N_1)_{60}$'.

Soil grain size is an important consideration when interpreting SPT data. The standard correlations and interpretations used in granular soil are for sands. Evidence suggests (Tokimatsu, 1988) that once the mean particle size (D_{50}) exceeds about 0.5 mm, penetration resistance climbs rapidly, approximately doubling at D_{50} = 10 mm. Correlations obtained for sands cannot be assumed to hold true for gravels, and larger diameter penetrometers have therefore been used in investigations of, for example, the liquefaction potential of gravels. Coarse-grained materials are not the only issue – the fines content is thought to contribute to lower penetration resistance (due to the development of pore pressures during driving) in silty sands.

In clays, the penetration resistance seems to be primarily controlled by the undrained shear strength. Unless there is a very significant drilling disturbance (which may well be a factor in soft and sensitive soils), this will not change significantly during drilling. Therefore the SPT N value is not corrected for overburden pressure in clays. N_{60} is used in design. Care must always be taken when looking at correlations or design procedures as to what form of N value (corrected or not, and, if corrected, specifically what corrections were used; different forms of correction can be found in the literature for the same thing, see BS EN 1997-2). Guidance on interpretation both for geotechnical parameters (see **Table 47.1**) and direct design methods can be found very widely in the literature (e.g. Clayton, 1995; see also conferences listed in 47.6 Further reading)) and in Annex F of BS EN 1997-2 (British Standards Institution, 2007b) with the associated caveats in the UK National Annex (British Standards Institution, 2009a), but care must always be taken as to their applicability for the ground under investigation.

47.2.2 Dynamic probing

Because it does not require a borehole, dynamic probing provides a simple, rapid and relatively inexpensive way of profiling a site. It is a dynamic, intermittent test. It is suitable for small depths of exploratory work and to fill in or extrapolate data from more expensive and sophisticated tests, using site-specific correlations. The equipment is generally light and compact and is ideal for work on sites with restricted access.

A dynamic probing test (DPT) consists of driving a solid steel cone vertically into the ground, via an anvil and extension rods, with successive blows of a free-fall hammer. Lightweight hammers may be raised by hand, but in most cases a motorised device, incorporating an automatic latch and release mechanism, is used. **Figure 47.2** shows a motorised dynamic probing rig.

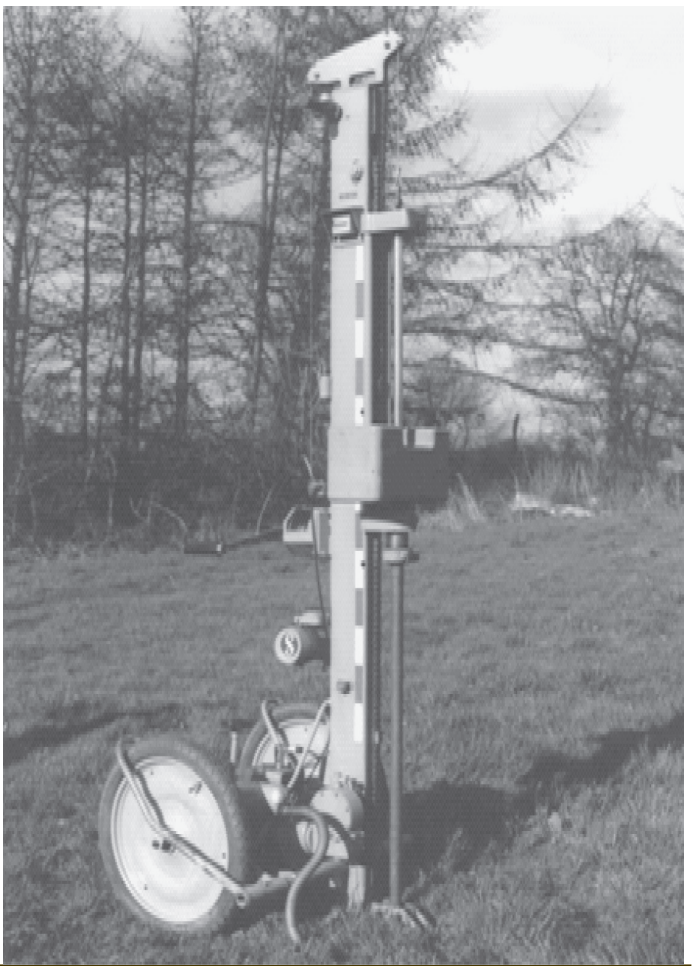


Figure 47.2 Motorised dynamic probing rig

The number of blows required to drive the cone a fixed distance (10 or 20 cm depending on the test specification) is recorded. The cone may be fixed to the extension rods, or loose-fitting, in which case it will be lost when the rods are withdrawn at the end of the test (sacrificial cone). Driving is halted every metre to add a further extension rod; at this point the torque required to rotate the rods in the ground is measured to assess the ground friction acting on them, which also has the effect of relieving friction for subsequent tests. The torque value can be used to correct the blow count to compensate for friction.

A variety of equipment has been developed and standardised over the years, with varying sizes of cones and hammer weights. Dynamic probing is now standardised in the UK through BS EN ISO 22476-2 (British Standards Institution, 2005a), which lists five configurations of equipment with varying cone size and drop weight (DPSH-A is not used in the UK). These configurations are generally the same as those that have been common in the UK and other countries (based on ISSMFE, 1989) for many years, except DPM, which has now changed to using a larger cone [The

configuration for DPM is now that of DPM15, given in earlier versions of BS5930 (British Standards Institution, 1999). The range gives increasing specific work per blow from CPL through to DPSH.

The different equipment configurations detailed in BS EN ISO 22476-2 (British Standards Institution, 2005a) are reproduced in **Table 47.2**. Standard ranges of blows are given as 3–50 for DPL, DPM and DPH and 5–100 for DPSH; it is recommended that if the minimum is not reached then the results cannot be used for quantitative purposes.

Figure 47.3 shows the acceptable shapes of the fixed and sacrificial cones. In the UK many companies use sacrificial cones of the same shape as the fixed cone. In general DPL, DPM and DPH have the same hammer drop height but different hammer weights and DPH and DPM use the same extension rod mass. The DPSH-B test is designed to closely resemble the Standard Penetration Test (SPT), as can be seen in **Table 47.2**, but differs from the other tests because it counts the number of blows per 20 cm penetration, rather than 10 cm. BS EN ISO 22476-2 (British Standards Institution, 2005a) recommends that the imparted energy should be measured for each new device, in a similar way to the SPT. In addition it recommends the actual energy E_{meas} transmitted to the drive rods is found from calibration when the test results are to be used for quantitative evaluation purposes, i.e. parameter determination.

47.2.2.1 Results and analysis of data

The probing results are recorded as blows for 10 cm (N_{10}) or 20 cm (N_{20}) penetration and should fall within the standard range of values stated in BS EN ISO 22476-2 (British Standards Institution, 2005a). Note that the values will be specific to the configuration used and the annexes of the standard suggest that the use of an extra subscript, N_{10L} or N_{10M} for example, would avoid any confusion when presenting results. With BS EN ISO 22476-2 (British Standards Institution, 2005a), it is now a requirement to measure the torque on the rods at least every metre and this has to be recorded and reported. The effect of rod friction on the $N_{10/20}$ values can be significant and now it is a requirement in the standard that this shall be considered, but there is no mention of how this should be done. Examples of the effects are given in the annexes to the standard.

The N_{10} values can be converted into unit cone resistance (r_d) or dynamic cone resistance (q_d), and this can allow different configurations of equipment to be brought to a common basis; both parameters can be derived from pile-driving formulae; see BS EN 22476–2 Annex E, thus:

$$q_d = \frac{M}{M + M'} r_d \tag{47.1}$$

$$r_d = \frac{Mgh}{Ae} \tag{47.2}$$

		Test specification			
Factor	**DPL**	**DPM**	**DPH**	**DPSH-A**	**DPSH-B**
Hammer mass, kg	10 ± 0.1	30 ± 0.3	50 ± 0.5	63.5 ± 0.5	63.5 ± 0.5
Height of fall, m	0.5 ± 0.01	0.5 ± 0.01	0.5 ± 0.01	0.5 ± 0.01	0.75 ± 0.02
Mass of anvil and guide rod (max), kg	6	18	18	18	30
Rebound (max), %	50	50	50	50	50
Rod length, m	1 ± 0.1%	1 ± 0.1%	1 ± 0.1%	1 ± 0.1%	1 ± 0.1%
Mass of rod (max), kg	3	6	6	8	8
Rod eccentricity (max),mm	0.2	0.2	0.2	0.2	0.2
Rod OD (d), mm	22 ± 0.2	32 ± 0.2	32 ± 0.2	32 ± 0.3	32 ± 0.3
Rod ID, mm	6 ± 0.2	9 ± 0.2	9 ± 0.2	9 ± 0.2	9 ± 0.2
Cone apex angle	90°	90°	90°	90°	90°
Cone area (nominal) (A), cm2	10	15	15	16	20
Cone diameter new (D), mm	35.7 ± 0.3	43.7 ± 0.3	43.7 ± 0.3	45.0±0.3	50.0±1.0
Cone diameter worn (min), mm	34	42	42	43	49
Mantle length, mm	35.7 ± 1	43.7 ± 1	43.7 ± 1	90 ± 2	50.5 ± 2
Number of blows per x cm penetration (Nx)	N10 :10	N10 :10	N10 :10	N20 :20	N20 :20
Standard range of blows	3–50	3–50	3–50	5–100	5–100
Specific work per blow (Mgh/A), kJ/m	50	98	167	194	238

Table 47.2 Details of dynamic probing test specifications
Data taken from BS EN ISO 22476-2:2005 (British Standards Institution, 2005a)

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where r_d and q_d are resistance values in Pa, M is the mass of the hammer in kilograms, g is the acceleration due to gravity in m/s^2, h is the height of fall of the hammer in metres, A is the projected area of the cone in m^2, e is the average penetration in metres per blow ($0.1/N_{10}$ from DPL, DPM15, DPM and DPH, and $0.2/N_{20}$ from DPSH), and M' is the total mass of the extension rods, the anvil and the guiding rods in kilograms.

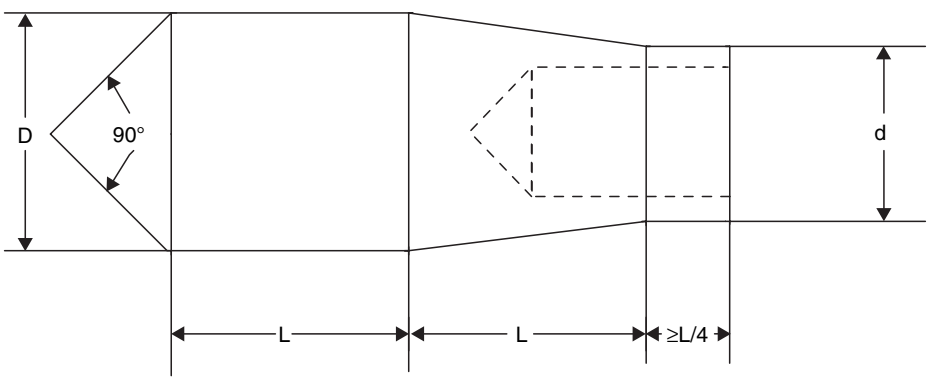
The value of r_d represents the work done in penetrating the ground and q_d modifies the r_d value to take account of the inertia of the driving rods, anvil and hammer during impact. The inclusion of the hammer weight, height of fall and cone size permits a comparison of the different configurations by way of r_d, but the inclusion of the size and mass of the extension rods in the calculations of q_d better normalises the results from the different equipment configurations. **Figure 47.4** shows typical data for a stiff clay site when three equipment configurations (DPL, DPM15 and DPH) were used. The results show how the different profiles for N_{10} are largely the same when plotted as r_d and q_d.

47.2.2.2 Rod friction

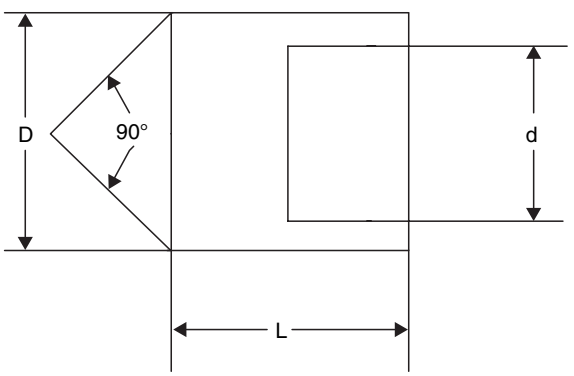
Friction on the rods will affect the recorded values of N_{10} or N_{20}. Experience has shown that torque readings in excess of 200 Nm generally mean that the driving rods have been forced off line and that further driving would permanently bend them, probably resulting in the driving rods no longer complying with BS EN ISO 22476-2 (British Standards Institution, 2005a). To avoid this, tests should be terminated when a torque reading reaches 120 Nm.

BS EN ISO 22476-2 (British Standards Institution, 2005a) provides for the introduction of drilling mud (bentonite slurry) to reduce rod friction. A specially perforated rod carries the cone and the drilling mud is introduced down the hollow rods and out through the perforations. For stiff clay sites, the mud can be poured into the rods to be sucked out or forced out by gravity into the annulus behind the probe cone. The level of bentonite should be topped up after the addition of each extension rod. For soft clay, a pressure just above the mean *in situ* stress needs to be applied to force the mud into the space behind the probe cone.

In its most common use, dynamic probing is used to look at variations within a profile and to map their extent laterally; however, in many countries considerable experience has been built up to convert the results into other parameters (see **Table 47.1**) particularly in coarse-grained deposits. For example,



Fixed cone



Sacrificial cone

D = cone diameter
d = extension rod diameter
L = cone body length

Figure 47.3 Acceptable shapes for fixed and sacrificial DPT cones

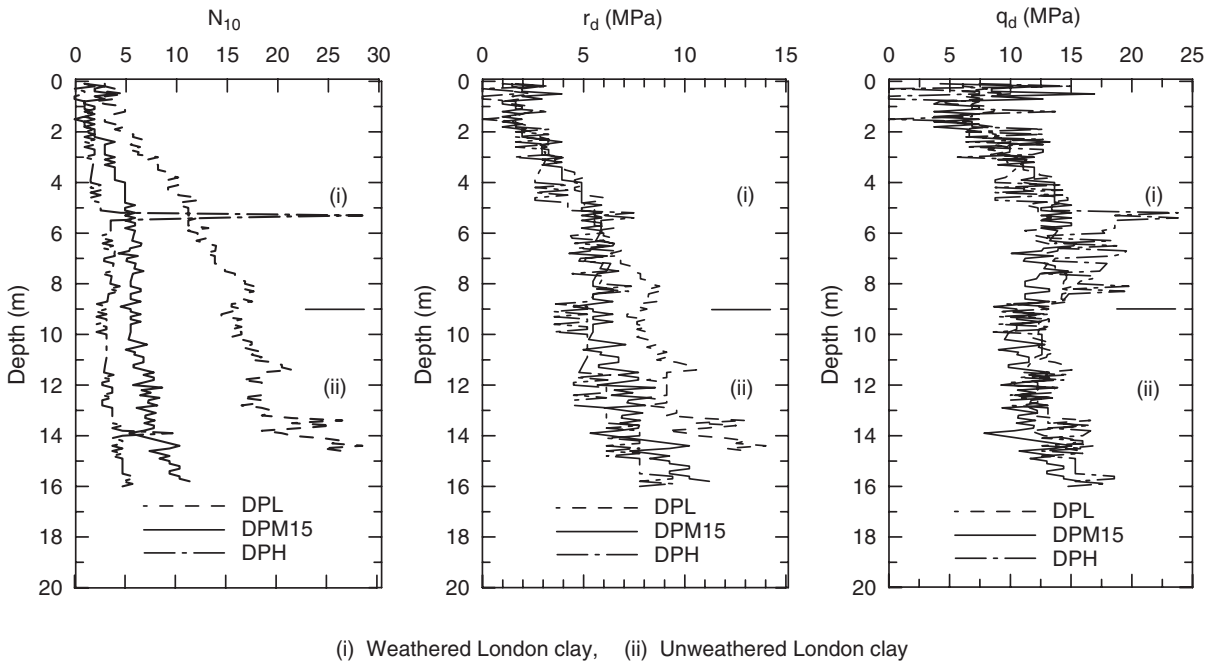


Figure 47.4 Results (N_{10}, r_d and q_d values) for three configurations of dynamic probing (DPT) for a stiff clay site

in the Annexes to BS EN ISO 22476-2 (British Standards Institution, 2005a) there are examples of the various factors that can affect DPT results and in BS EN 1997-2 Annex G (British Standards Institution, 2009a) some examples are given of interpretation of DPT results in coarse-grained materials. Other examples can be found in the literature (see Further reading). It is vital to ensure that any correlations used have been validated for the local soil types and DPT configuration; in the future with better calibration and energy measurements it may be possible to transfer correlations between configurations. Little has been published on their interpretation in clays but Butcher *et al.* (1995) give examples of the potential for interpretation in clays to determine geotechnical parameters. BS EN ISO 22476-2 (British Standards Institution, 2005a) does point out that because of hammer fall energy losses, it is recommended to know by calibration the actual energy E_{meas} transmitted to the drive rods when this test is used for quantitative evaluation purposes.

47.2.3 The cone penetration test (CPT)

Among the many *in situ* test devices available, the electric static cone penetrometer (CPT) and the piezocone (CPTU) are among the best. CPT is a quasi-static test that is near continuous; it is generally undertaken without a borehole but can be used with boreholes to advance past obstructions or to increase the depths achieved. Tests are rapid, can generate detailed ground profiles, can classify the ground, and the results can be used to calculate a wide range of geotechnical parameters that accurately reflect ground properties. The test is covered by an International Reference Test Procedure (IRTP) (ISSMGE, 1999) BS 1377 Part 9 (British Standards Institution, 2007a), BS EN ISO 22476-1 (British Standards Institution, in preparation, f; when published, this will replace the IRTP and BS 1377 Part 9) and BS EN 1997-2 (British Standards Institution, 2007b). In some countries older-style cones may be encountered that use remote mechanical systems for force measurement (CPTM) – these are covered by BS EN ISO 22476-12 (British Standards Institution, 2009b).

47.2.3.1 Test equipment

The diameter of the standard 60° cone is 35.7 mm (giving a cross-sectional area of 10 cm^2) and the area of the friction sleeve is 150 cm^2. The cone resistance and sleeve friction are normally measured via strain-gauged load cells. **Figure 47.5** illustrates the main components and terminology of cone penetrometers. The CPT and CPTU systems generally include an inclinometer, to warn if the cone is going off vertical, which can easily happen if hard obstructions are encountered.

Although a cone penetrometer with a 10 cm^2 base area cone and an apex angle of 60° is specified in the International Reference Test Procedure (ISSMGE, 1999) and BS EN 22476-1 (British Standards Institution, in preparation, f) as the reference test (being the same size as the original mechanical cone), other sizes of cone exist. Apart from the 10 cm^2 device, the 15 cm^2 cone is the only one routinely found in land-based practice but the use of 2 and 5 cm^2 devices can be found in the offshore environment (along with the larger sizes).

The standards detail all necessary requirements with regard to shape, dimension and area, and their tolerances, as well as operational procedures. Provided the equipment dimensions

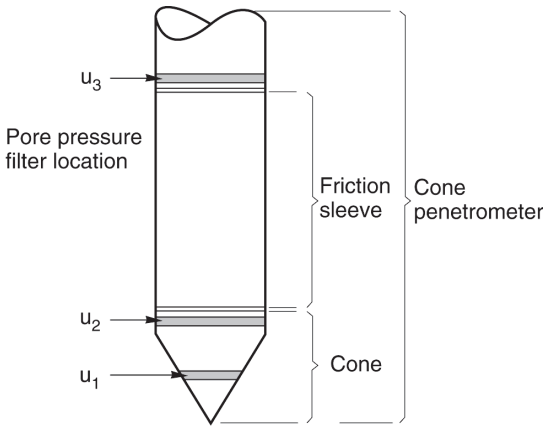


Figure 47.5 Main components of the CPT cone, also showing the positions for pore pressure measurement

are kept within these tolerances, the likely errors in measured parameters, simply from wear, will be kept to less than a few per cent.

The addition of a pore water pressure measuring system within the CPT (CPTU) has greatly enhanced its capability for both profiling and parameter determination. **Figure 47.5** shows that the pore pressure may be measured at several locations: 'u_1' (on the cone), 'u_2' (behind the cone) and 'u_3' (above the friction sleeve). Generally, it is recommended that the pore pressure is measured just behind the cone shoulder (at the 'u_2' position) for the following reasons:

1. good protection from damage, relative to 'u_1';
2. relatively easy saturation;
3. gives good profiling detail;
4. generally good dissipation data;
5. correct location to determine q_t (see corrections below).

However, the other locations may be useful in certain conditions (see Lunne *et al.*, 1997).

The pore pressure measurement system should be designed in such a way that it is easy to saturate and to keep saturated. This is because successful CPTU profiling depends very much on good saturation of the pore pressure measurement system, particularly in clays.

In order to attain satisfactory accuracy it is essential that the instrument is calibrated regularly and over load ranges appropriate for the ground being tested. On every project, and at regular intervals on a large project, the engineer should check that the calibration information being used is current and appropriate. It is also important to take, report and review sensor base readings before and after each full penetration – this is a requirement in the IRTP and BS EN ISO 22476-1 standard (British Standards Institution, in preparation, f). Calibration enables the assessment of the stability of the sensors and any change due to damage, especially of the load cells. In both cases significant changes in the base readings for the load cells can be converted into measured load and then into an assessment of the potential inaccuracy in the results. The IRTP and BS EN ISO 22476-1 (British Standards Institution, in preparation, f) introduce the idea of accuracy classes (in the former) and application classes (in the latter) and these set requirements for the accuracy of the equipment as a function of soil type and likely end use of the results, e.g. profiling or property determination. They require that the CPTU is used when geotechnical parameters are to be determined from the results. These classes are a critical item in the specification of appropriate tests, and in their interpretation. **Table 47.3** shows how the classes relate to the requirements for equipment and usage.

47.2.3.2 Test method

The cone, on the end of a string of rods, is pushed into the ground at a constant rate and near-continuous or intermittent measurements are made of the resistance to penetration of the conical tip. Measurements are also made of the resistance to penetration of the friction sleeve, above the cone. When using a piezocone, pore pressure measurements are also made.

The cone is normally pushed into the ground at a rate of 20±5 mm/s. A wide range of systems have been developed over the years to carry out the pushing operation, both on land and at sea. On land, pushing usually consists of hydraulic jacking against reaction systems specially built for the purpose; but sometimes the pushdown of an anchored drill rig is used. Hydraulic jack stroke lengths of 1–1.2 m are typically used, but shorter lengths may be adopted when working in limited headroom. Offshore systems exist that can either operate continuously down a drill string or by pushing from a jacking unit on the seabed.

During penetration, measurements are taken from all sensors at a fixed frequency so as to give discrete readings for forces, pore pressures and inclination, as appropriate (i.e. for the accuracy or application classes). Typically the minimum frequency is once every second or 2 cm of penetration; faster rates can be used to increase the detail of the ground profile (see Lunne *et al.*, 1997). In this way near-continuous profiles of the measured parameters are generated. A typical mixed profile is shown in **Figure 47.6**.

At any point during a test, penetration can be stopped and the CPTU left stationary, with measurements of pore pressure taken as a function of time (many standards also require cone resistance and sleeve friction to be recorded); this is referred to as a pore pressure 'dissipation test'. These tests can be used to determine the coefficient of consolidation. When penetration pore pressures are left to dissipate it is possible to estimate the *in situ* static pore pressure–depth relationship.

47.2.3.3 Cone parameters

The measured total force, Q_C, acting on the cone end, divided by the projected area of the cone, A_C, gives the measured cone resistance, q_c. The total measured force acting on the friction sleeve, F_S, divided by its surface area, A_S, gives the measured

Application class	Test type	Measured parameter	Allowable minimum accuracy[1]	Maximum length between measurements	Use	
					Soil[2]	Interpretation and evaluation[3]
1	TE2	Cone resistance	35 kPa or 5%	20 mm	A	G, H
		Sleeve friction	5 kPa or 10%			
		Pore pressure	10 kPa or 2%			
		Inclination	2°			
		Penetration length	0.1 m or 1%			
2	TE1 TE2	Cone resistance	100 kPa or 5%	20 mm	A	G, H*
		Sleeve friction	15 kPa or 15%		B	G, H
		Pore pressure[4]	25 kPa or 3%		C	G, H
		Inclination	2°		D	G, H
		Penetration length	0.1 m or 1%			
3	TE1 TE2	Cone resistance	200 kPa or 5%	50 mm	A	G
		Sleeve friction	25 kPa or 15%		B	G, H*
		Pore pressure[4]	50 kPa or 5%		C	G, H
		Inclination	5°		D	G, H
		Penetration length	0.2 m or 2%			
4	TE1	Cone resistance	500 kPa or 5%	50 mm	A	G*
		Sleeve friction	50 kPa or 20%		B	G*
		Penetration length	0.2 m or 2%		C	G*
					D	G*

NOTE For extremely soft soils even higher requirements for accuracy may be needed.

1. The allowable minimum accuracy of the measured parameter is the larger value of the two quoted. The relative accuracy applies to the measured value and not the measuring range.
2. According to EN ISO 14688–2:
   - A Homogeneously bedded soils with very soft to stiff clays and silts (typically $q_c < 3$ MPa);
   - B Mixed bedded soils with soft to stiff clays (typically $q_c \leq 3$ MPa) and medium dense sands (typically 5 MPa $\leq q_c < 10$ MPa);
   - C Mixed bedded soils with stiff clays (typically 1.5 MPa $\leq q_c < 3$ MPa) and very dense sands (typically $q_c > 20$ MPa);
   - D Very stiff to hard clays (typically $q_c \geq 3$ MPa) and very dense coarse soils ($q_c \geq 20$ MPa).
3. G profiling and material identification with low associated uncertainty level;
   - G* indicative profiling and material identification with high associated uncertainty level;
   - H interpretation in terms of design with low associated uncertainty level;
   - H* indicative interpretation in terms of design with high associated uncertainty level.
4. Pore pressure can only be measured if TE2 is used.

Table 47.3 Application classes from EN ISO 22476-1
Data taken from BS EN ISO 22476-1 (British Standards Institution, in preparation, f)

sleeve friction, f_s. In CPTU, the pore pressure is measured as discussed above. The friction ratio, R_f, is obtained by dividing the sleeve friction by the cone resistance and is expressed as a percentage.

Pore pressure measurements can be used to correct measured results for geometrical effects (see below). However, a pore pressure measurement may not always be achievable, for example in unsaturated soils and above the groundwater table.

47.2.3.4 Effects of penetration rate, pore pressure and verticality

The results of a test are to some extent dependent on how fast the penetrometer is inserted into the ground. The speed of penetration should be 20 ± 5 mm/s according to the International Reference Test Procedure (ISSMGE, 1999) and BS EN ISO 22476-2 (British Standards Institution, 2005a). The rate of penetration, especially when using the piezocone in fine-grained soil, affects pore pressure generation and there may in addition be viscosity effects on q_c and f_s. The penetration rate may also influence creep and particle crushing. Typically a tenfold increase in the rate causes a 10–20% increase in the measured cone resistance in stiff clays and 5–10% in soft clays (e.g. Powell and Quarterman, 1988). If the standard is followed, the rate should not be a problem in clays and sands; however, in intermediate soils the effect of rate changes may be more significant for the measured parameters. The rate may change most noticeably when the pushing equipment is reaching capacity and may slow down.

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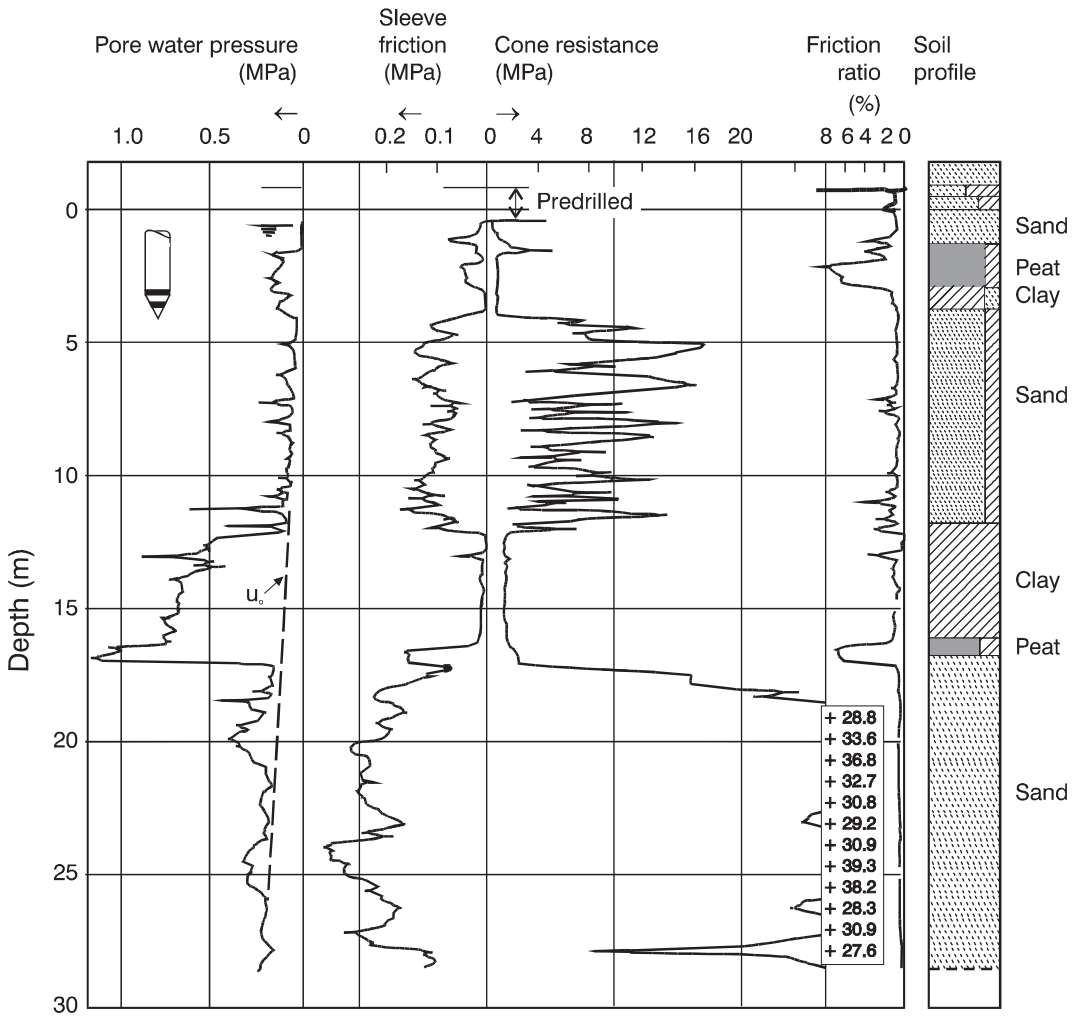


Figure 47.6 Example profile of measured and derived parameters from a CPTU profile

As has been noted, the most common onshore insertion method is to push the cone penetrometer into the soil using a rig with hydraulic jacks, with a stroke length of about 1 m. At the end of each stroke, the pushing stops and the jacks return to their start position. This is referred to as 'intermittent testing'. In some soils this causes 'unrepresentative readings' every time the test is stopped and a new rod added. These effects can be even more pronounced after pauses for dissipation tests when, because of the drop in pore water pressure and resultant consolidation, local increases in both q_c and f_s can be observed; there may also be a delay in the pick-up of the pore pressure response. This behaviour could be misinterpreted as a local ground feature if not anticipated.

The measured cone resistance is affected by pore water pressures acting downwards, on the top of the cone (**Figure 47.7**). The magnitude of the effect will vary from cone to cone depending on the geometry and will result in significantly different (40%) measurements of q_c in the same soft fine-grained soil. This variability of results may be avoided by using the CPTU since the pore water pressure effects may be corrected for by using:

$$q_t = q_c + u_2 (1 - a) \tag{47.3}$$

where q_t is the corrected cone resistance, q_c is the measured cone resistance, u_2 is the pore water pressure measured just behind the cone and a is the area ratio (that area affected by the pore water pressure ranges from 0.35 to 0.95, typically 0.7-0.85, see Lunne *et al.*, 1997).

This correction is a requirement in the IRTP and BS EN ISO 224476-1 (British Standards Institution, in preparation, f) for some 'accuracy' or 'application' classes and when the results are to be used to interpret geotechnical parameters. Friction sleeves are also affected by pore water pressure effects but correction is more difficult because it requires measurement of both u_2 and u_3, as shown in **Figure 47.7**. The effects can be reduced significantly if the end areas A_{st} and A_{sb} are equal.

Another correction that should be considered is for errors in depth measurement due to deviation of the cone from the

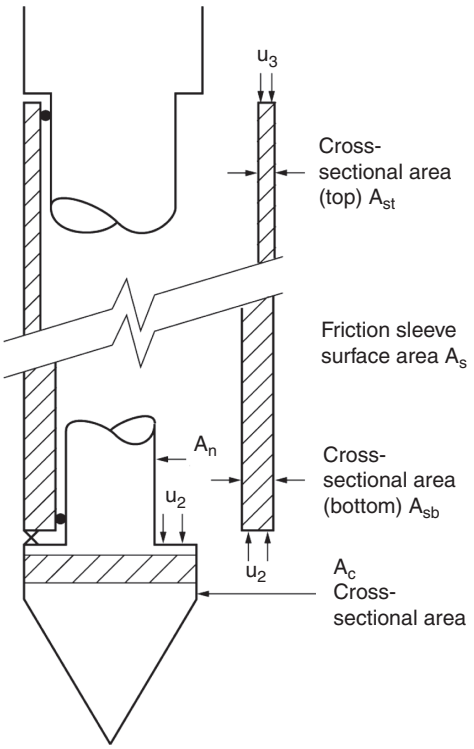


Figure 47.7 Effect of pore pressure on cone resistance

vertical. Most electric cone penetrometers contain simple slope sensors to measure the non-verticality of penetration. This is useful in avoiding damage to equipment due to sudden deflection. Where penetrations are deep, inclinations in excess of 45° are not uncommon, especially in stratified soil, and can result in wrongly identifying the depth of changes of the strata. This correction is a requirement in both the IRTP and BS EN ISO 22476-1 (British Standards Institution, in preparation, f) for some 'accuracy' or 'application' classes.

The CPT and /CPTU test probably has the widest range of ground parameters that can be derived from its results, mostly by correlation (see **Table 47.1**). The applicability of any correlation used must be ensured if confidence in the resulting parameters is to be achieved. Deriving 'site-specific' correlations is often useful. Results from CPT and CPTU tests can also be used directly in design, e.g. pile capacity, settlement predictions, for assessment of liquefaction potential or the effect of ground improvement. A number of sources can be used – see Lunne *et al*. (1997), BS EN 1997-2 Annex D (British Standards Institution, 2007b), as well as various technical conferences involving CPTs, some of which are listed in section 47.6 Further reading.

47.3 Loading and shear tests

Loading and shear tests are of most benefit when representative samples cannot be taken, for example in made ground, very soft and sensitive clays, granular or stony soils, or fractured rocks. Some (e.g. the vane test) are relatively easy and low cost. Others (e.g. large-diameter plate tests and the more sophisticated pressuremeter tests) are expensive, can be time consuming, and are not used during routine investigations.

47.3.1 The vane test

The vane shear test is used to determine the undrained shear strength of soft soils (although it is used in stiffer materials in some countries). It is standardised in BS 1377 part 9 (British Standards Institution, 2007a), BS EN 1997-2 (British Standards Institution, 2007b) and the anticipated BS EN ISO 22476-9 (British Standards Institution, in preparation, d) – when published, this standard will replace BS 1377 Part 9. The test most commonly used consists of pushing a cruciform vane mounted on a solid rod into the soil and rotating it from the ground surface whilst measuring rotation and torque. In different countries, vane tests may be carried out either by pushing the device from ground level, or from the base of a borehole, or both. The use of a borehole, brings with it the risk of strength reduction as a result of borehole drilling disturbance, and this must be recognised and dealt with. As a result of developments in the offshore industry, where measurement at the surface would be unrealistic, equipment is now available for both land and offshore work where the torque and rotation are measured electronically (down-hole) close to the vane head.

In its conventional form (**Figure 47.8**), the field vane has four rectangular blades and with a height-to-diameter ratio (H/D) of two. In the UK the dimensions of field vanes are controlled by BS 1377 (British Standards Institution, 2007a) and BS EN ISO 22476-9 (British Standards Institution, in preparation, d). BS 1377 gives suitable overall dimensions based on experience and these are shown in **Table 47.4**.

By implication, BS 1377 considers that the field vane will not be suitable for testing soils with undrained strengths greater than about 75 kPa. However, in some countries the vane test is used in stiffer soils and BS EN ISO 22476-9 (British Standards Institution, in preparation, d) recognises this; see **Table 47.5**.

The test is generally not suitable for fibrous peats, sands or gravels, or in clays containing laminations of silt or sand, or stones.

Four types of vane are in use around the world. In the first, the vane is pushed unprotected from the bottom of a borehole or from the ground surface. In the second, a vane housing is used to protect the vane during penetration, and the vane is then pushed ahead of the bottom of the vane housing before the test is started. In the third, the vane rods are sleeved to minimise friction between the ground and the rods during the test. Finally, some vanes incorporate a swivel just above the blades, which allows about 90º of rod rotation before the vane is engaged. This simple device allows the measurement of rod friction (which can be significant when the vane is pushed from ground level) as an integral part of the test.

In all tests it is important that the vane is pushed ahead of any disturbance caused either by the vane housing or any boring

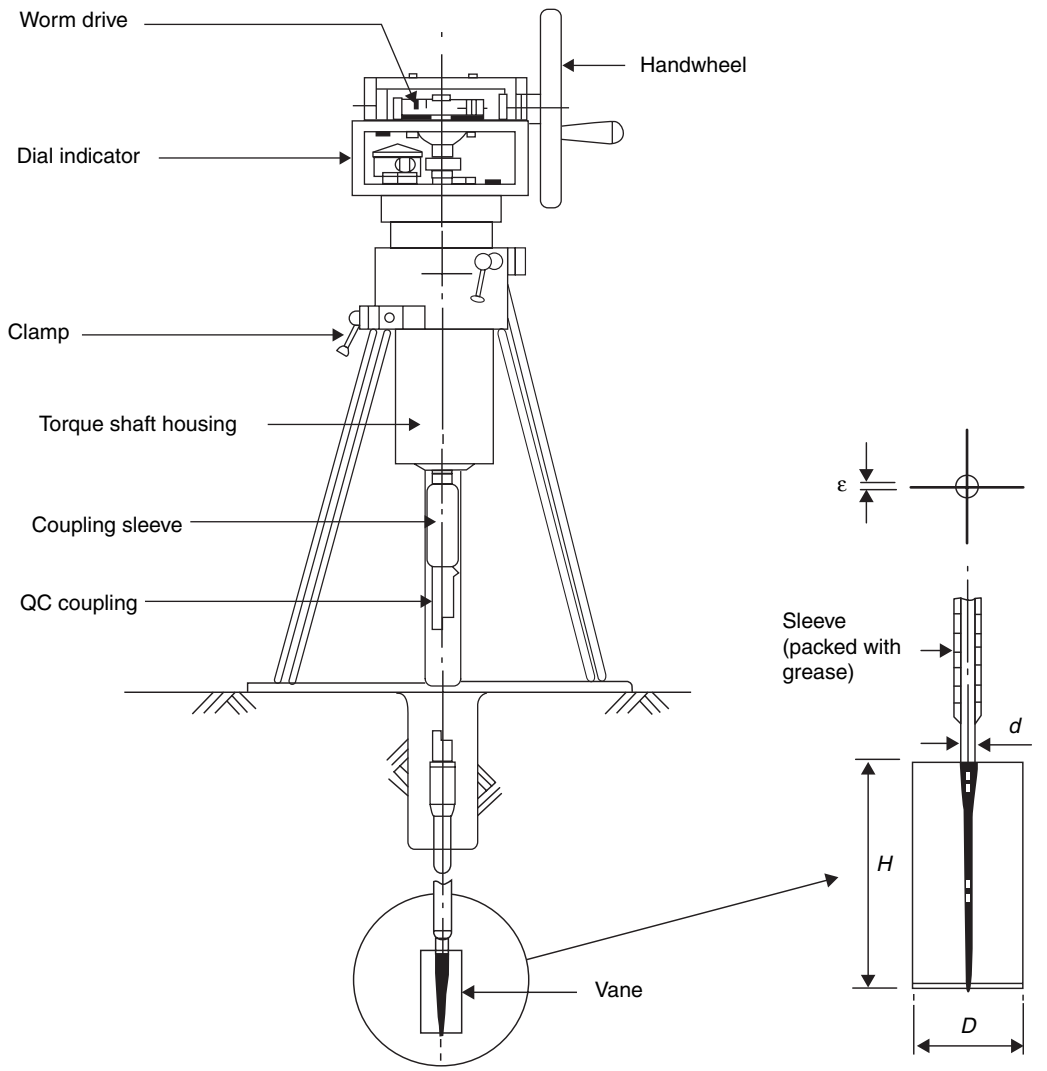


Figure 47.8 Field vane testing equipment
Reproduced from Clayton *et al.* (1995); Wiley-Blackwell

Undrained shear strength (kPa)	Vane diameter (mm)	Vane height (mm)	Rod diameter (mm)
<50	75	150	<13
50–75	50	100	<13

Table 47.4 UK specifications for vane blades (British Standards Institution, 2007a)

Soil type	Undrained shear strength (kPa)	Vane diameter suggestion[1] (mm)
Very stiff to stiff clays and silts	50–300	30–50
Medium stiff clays and silts	20–50	50–75
Soft clays and silts	10–20	75–100
Very soft clays	< 10	≥ 100

1. Still using H/D of 2.

Table 47.5 BS EN ISO 22476-9 vane dimensions
Data taken from BS EN ISO 22476-9 (British Standards Institution, in preparation, d)

operations. ASTM D2573 (ASTM, 2008b) specifies that the vane should be pushed five vane-housing diameters ahead of the vane housing before testing, and that when a borehole is used to get down to the test depth the vane should be advanced at least five borehole diameters ahead of the bottom of the borehole. BS EN ISO 22476-9 will have similar requirements (British Standards Institution, in preparation, d). During this process it is important that vane rotation follows the relevant standard.

Once the vane has been pushed to the required test depth, the test procedure is as follows:

1. Attach a torque wrench, or preferably a purpose-built geared drive unit, to the top of the vane rods, and turn the

rods at a steady but continuous rate. This should be fast enough to ensure the test is undrained but slow enough to ensure readings can be taken and potential rate effects are not significant. A rate of rotation of between 6 and 12° min is typically used. During this phase record the relationship between rod rotation and measured torque by taking readings of both at intervals of 15–30 s.

2. Once maximum torque is clearly achieved, or in the case of BS EN ISO 22476-9 a rotation of 180° is reached, the vane is then rotated rapidly through a minimum of 10 revolutions.
3. The shearing is then restarted at the previous slow rate, to determine the remoulded strength of the soil.

A torque–rotation curve is obtained from the measurements made during the test stages, but the reliability of this will depend on the accuracy of the rotation measurements (or assumed rotation); BS EN ISO 22476-9 (British Standards Institution, in preparation) gives the requirements for the accuracy of these measurements.

The vane test is routinely used only to obtain the 'undisturbed' peak undrained shear strength (based on the maximum or peak torque), and the remoulded undrained shear strength (using the torque in the retest) of soft soils, and thereby to give an assessment of a soil's sensitivity. BS EN ISO 22476-9 (British Standards Institution, in preparation) also defines a 'residual' undrained shear strength derived from the torque at 180° rotation in the initial phase). The test results should be reported as 'vane' shear strengths. BS EN 1997-2 (British Standards Institution, 2007b) and BS EN ISO 22476-9 (British Standards Institution, in preparation) use c_{fv} for the 'undisturbed' shear strength, c_{Rv} for the residual value and c_{rv} for the remoulded value. In BS EN ISO 22476-9 (British Standards Institutionin preparation), the sensitivity is c_{fv}/c_{rv}. In interpretation it is assumed, amongst other things, that the penetration of the vane causes negligible disturbance, that no drainage occurs during shear, and that the soil fails on a cylindrical shear surface, whose diameter is equal to the width of the vane blades.

The results of a vane shear test may be influenced by many factors, namely:

1. type of soil, especially when a permeable fabric exists or stones are present;
2. strength anisotropy;
3. disturbance due to insertion of the vane;
4. rate of rotation (strain rate);
5. time lapse between insertion of the vane and the beginning of the test;
6. progressive failure of the soil around the vane.

It is necessary to carry out the vane test rapidly, in an attempt to ensure that the shear surface remains reasonably undrained. The presence of sand or silt lenses or laminations within the test section will certainly make this assumption invalid, but (again in common with many *in situ* tests) it is not normally possible to know what type of material is about to be tested. Free-draining materials should not be tested. In higher plasticity clays, the shear strength is likely to be increased by viscous effects, and there is evidence (Parry and McLeod, 1967) that unreasonably high values can be obtained.

The presence of stones or fibrous peat will mean that the assumption of a cylindrical shear surface with a diameter equal to the vane blade width is invalid. Again, the shear strength will tend to be overestimated.

Perhaps the most serious problem can result from the disturbance induced in the ground by the insertion of the vane blades. La Rochelle *et al*. (1973) reported that thicker vane blades result in lower undrained shear strength values because of greater soil disturbance and also because of the induced increase in pore water pressure in the soil surrounding the vane. In a typical test, a torque is applied shortly after insertion, and this pressure does not have time to dissipate. Hence, the time interval between the moment of vane intrusion and the time of failure is also of importance in influencing measured strength. BS 1377 part 9 (British Standards Institution, 2007a) specifies that the area ratio (the volume of soil displaced, divided by the volume of soil within the assumed cylindrical shear surface), which is given by:

$$A_r = \frac{\left[8t(D-d)+\pi d^2\right]}{\pi d^2} \tag{47.4}$$

where t is the vane blade thickness, D is the vane diameter and d is the diameter of the vane rod, below any sleeve and including any enlargements due to welded joints,

shall not exceed 12%. The disturbance caused by vane insertion is a matter for concern, and engineers should ensure that the dimensions of the vanes that they use are recorded.

A torque–rotation curve is generally obtained from the measurements of rotation and torque, but the reliability of this will depend on the accuracy of the rotation measurements (or assumed rotation); BS EN ISO 22476-9 (British Standards Institution, in preparation, d) gives the requirements for the accuracy of these measurements.

It is usual to correct 'vane' undrained shear strengths before using them in design and several examples are given as guidance in BS EN 1997-2 Annex I (British Standards Institution, 2007b). The applicability of these corrections must always be checked for the ground conditions under investigation.

47.3.2 Plate testing

Techniques for carrying out the plate loading test have been described by ASTM D1194-72 (ASTM,), in BS EN ISO 22476-13 (British Standards Institution, in preparation, e), and in BS 5930 (British Standards Institution, 2010) and BS 1377 part 9 (British Standards Institution, 2007a). When published, BS EN ISO 22476-9 (British Standards Institution, in

preparation, d) will replace BS 1377 part 9. In the most common form of the test a relatively rigid metal plate (minimum diameter 300 mm) is bedded onto the soil to be tested, either using sand/cement mortar or plaster of Paris. A load is applied to the plate in successive increments of about one-fifth of the design loading, and held until the rate of settlement reduces to an acceptable level (**Figure 47.9**).

BS 1377 part 9 (British Standards Institution, 2007a) suggests that 'the load should preferably be maintained at each increment until the penetration of the plate has ceased. Tests on cohesive (fine-grained) soils should be continued at least until all the primary consolidation is complete, judged according to the settlement versus log time plot'. Clayton *et al*. (1995) suggest that each load be held until the rate of settlement is less than 0.004 mm/mm, measured for a period of at least 60 minutes, and that load increments are applied either until:

1. shear failure of the soil occurs; or more commonly
2. the plate pressure reaches two or three times the design bearing pressure proposed for the full-scale foundation.

The load is usually applied to the plate via a calibrated hydraulic load cell and a hydraulic jack. The hydraulic jack may either bear against beams supporting concrete blocks, or (preferably) a reaction may be provided by tension piles or ground anchors installed on each side of the load position. For lighter loads, a reaction against a suitable piece of plant is also possible.

At each load increment, a note is made of the load on the plate and dial gauge readings are made on a 'square of the integer' basis (i.e. 1, 4, 9, 16, 25 mm, etc.) after load application. This will ensure sufficient readings in the early stages of each load application when movement occurs most rapidly.

The results of these measurements are normally plotted in two forms: a time–settlement curve for each applied load, and a load–settlement curve for the entire test.

The number of tests required depends on both the soil variability and the consequences of poor data for geotechnical design. Tests should not normally be carried out in groups of less than three, and in order to allow assessments of variability any plate testing should be carried out at the

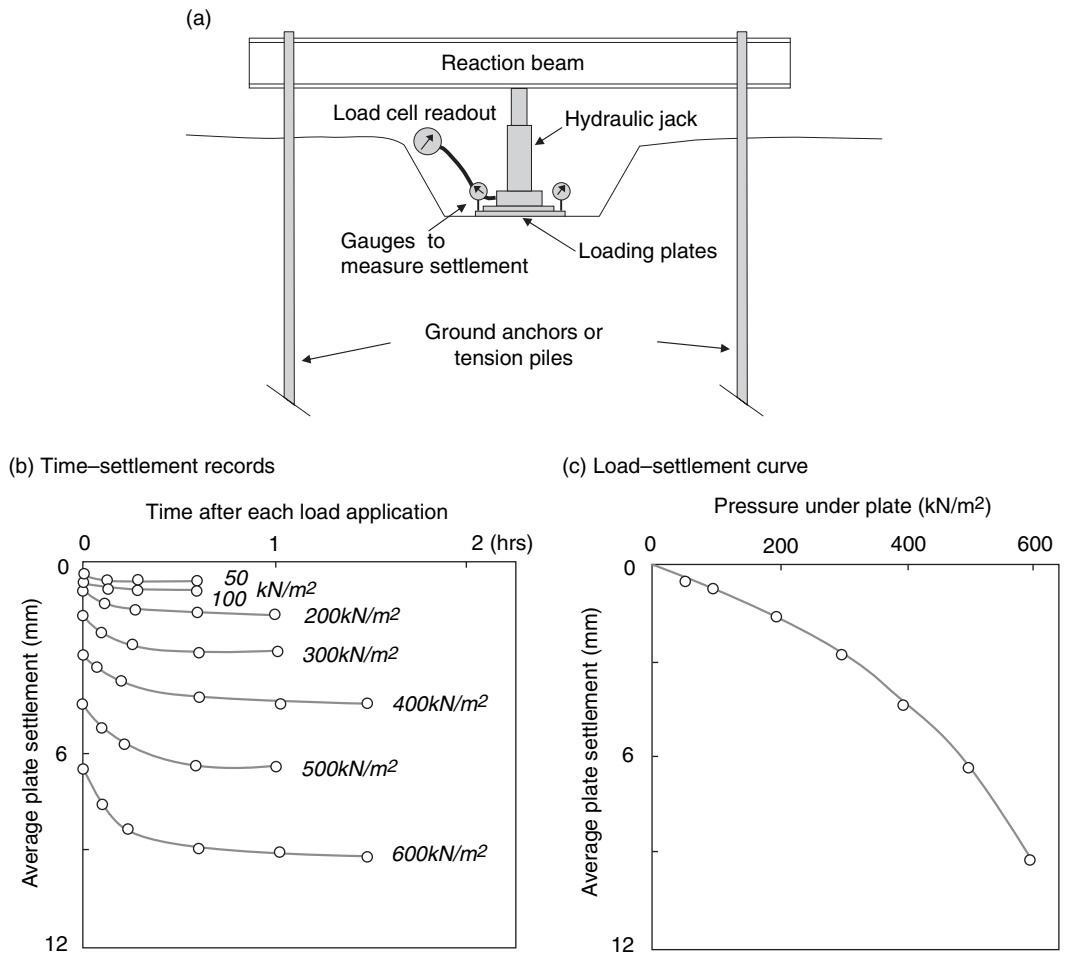


Figure 47.9 Plate load test equipment and results
(b,c) Reproduced from Clayton *et al*. (1995); Wiley-Blackwell

end of a site investigation, or as part of a supplementary investigation.

The size and location of plate tests should be assessed on the basis of *in situ* testing and visual examination of the soil or rock to be investigated. As a general rule of thumb, the plate diameter should never be less than either six times the maximum soil particle size or six times the maximum intact rock block size. The use of this rule of thumb ensures that enough discontinuities or inter-particle contacts exist in the stressed zone to give representative results, but it does not aid in extrapolating results when tests are carried out only at the proposed foundation level.

It is quite clear from the literature that the extrapolation of settlement from small plates to large loaded areas on granular soils is rather unreliable, and therefore the plate loading test on granular material should be regarded as giving a modulus of compressibility value for the soil immediately beneath the test location. Elastic stress distributions indicate that the soil will only be significantly stressed to a depth below the plate of about 1.0–1.5 times the width of a square or circular loaded area.

Plate tests on rocks appear to present a rather more attractive proposition, partly because reliable methods of predicting the settlement on rocks are almost non-existent, and also because the sorts of structure for which good estimates of the settlement on rock are required will normally justify the high expenditure necessary. Most civil engineering structures will be founded in the upper, more weathered rock zones. Ward *et al*. (1968) and Hobbs (1975) have shown that in these zones it is the compressibility of the discontinuities, and not the intact rock, which controls the compressibility of the rock in the mass. The compressibility of joints and bedding planes can be assessed visually, based on experience, but actual values under working loads can only be obtained satisfactorily from a (sufficiently large) *in situ* loading test.

A logical approach to the problem is to use a plate of sufficient size to determine a modulus value, and either to carry out tests at different levels, or to correlate tests on different materials with their weathering grades. However, the cost of large borehole plate tests is normally prohibitive. For plate tests intended to give elastic moduli values for soils or rocks BS EN 1997-2 Annex K.2 (British Standards Institution, 2007b) uses the equation for a uniformly loaded rigid plate on a semi-infinite elastic isotropic solid,

$$E = \frac{\pi q B}{4} \frac{(1-\nu^2)}{\rho} \tag{47.5}$$

where E is the elastic modulus, q is the applied pressure between the plate and the soil, B is the plate width, ρ is the settlement under the applied pressure q and ν is Poisson's ratio.

For granular soils and soft rocks Poisson's ratio will normally be between 0.1 and 0.3, and so the term $(1 - \nu^2)$ has a relatively small effect. Where plate tests are carried out in the stressed zone of a proposed foundation the value of q can be taken as the vertical foundation stress to be applied at the level of the plate test, or alternatively, a safety margin can be incorporated by taking q to be 50% (for example) higher than the estimated applied stress.

Where plate tests are intended to give values for the shear strength or bearing capacity in fine-grained soils, the load is not applied in stages. The plate is pushed downwards to give a constant rate of penetration, and the undrained shear strength is deduced from the ultimate bearing capacity; see BS EN 1997-2 Annex F (British Standards Institution, 2007b).

Two enhancements have sometimes been used in conjunction with larger, more expensive, plate tests, such as are used for major investigations (reactor foundations or underground caverns) in weak rocks:

- Multi-point borehole extensometers may be placed under the plate, in order to allow the determination of strain levels at various distances away from the loading (for example, see Marsland and Eason, 1973 and Barla *et al*., 1993). Stress changes at the measuring points must be determined from elastic theory, even though this may be rather unreliable.
- An oil-filled pad (similar to a flat jack) may be placed between the plate and the rock in an attempt to remove the concentration of stresses at the plate edge due to its rigidity. When this is done, the estimates of stress change are improved, but an interpretation of surface movements should be made on the basis of the settlement of a fully flexible loaded area on an elastic half space.

Smaller plate loading tests are routinely carried out down-hole in some countries during investigations that rely upon correlation with a visual description. This combination has proved particularly valuable above the water table in hard, gravelly or unsaturated and saprolitic soils, all of which can be very difficult to sample and test. The plate test is carried out in a large-diameter hole, which is formed using an auger-piling rig. Another adaptation of the plate test is the 'skip test'. Here a heavy-duty waste-disposal skip is used to simulate the relatively low levels of loading produced, for example, by low-rise housing. This type of test is now the subject of a standard: BS 1377 part 9 (British Standards Institution, 2007a). Settlements are measured using levelling.

47.3.3 Marchetti dilatometer test (DMT)

The Marchetti dilatometer test (DMT) [DD CEN ISO/TS part 11 (British Standards Institution, 2006), BS EN 1997-2 (British Standards Institution, 2007b), ASTM D6635 (2007) (ASTM, 2007)] is a versatile tool for *in situ* ground testing. It is relatively quick and robust, can generate profiles of information and the results can be interpreted for a wide range of geotechnical parameters or used directly for design. Despite this, it is relatively rarely used in the UK, and for that reason will only be discussed briefly.

The blade (**Figure 47.10**) is pushed vertically into the ground, using rods. It is 250 mm long, 94 mm wide and 14 mm thick, with a tip angle of 16º. It has a flat, circular steel membrane mounted flush on one side. The blade is normally jacked into the ground using available field equipment such as the rigs normally used



Figure 47.10 Marchetti dilatometer blade

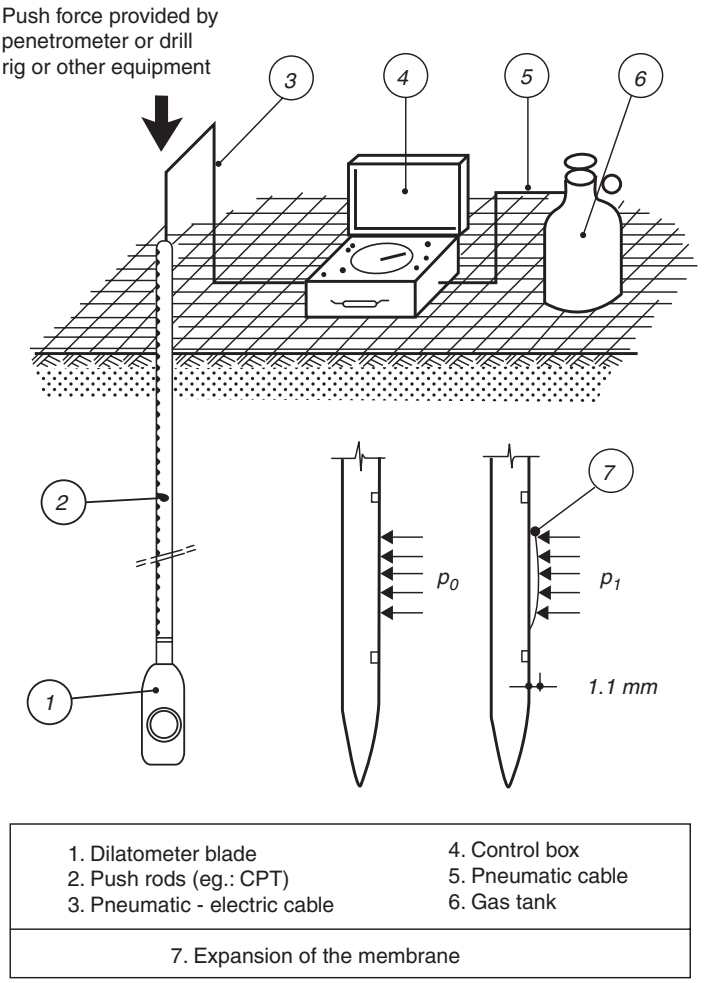


Figure 47.11 Layout of test equipment for DMT testing

for the cone penetration test (CPT), or drill rigs. The general layout of the dilatometer test is shown in **Figure 47.11**. At regular intervals (typically every 0.2 m) penetration is halted, and a test is performed by inflating the membrane by gas pressure.

The blade is connected to a control unit at the ground surface by a tube that transmits both the gas pressure and an electrical signal, running through the push rods. After penetrating to a test depth, the operator inflates the membrane using the control unit, and takes, in about one minute, two (or three) readings:

- the pressure required to just move the membrane against the soil; this 'lift-off' pressure, denoted the A pressure, is identified by the cessation of an audible signal;
- the pressure required to move the centre of the membrane 1.1 mm into the soil; this B pressure is identified by the audible signal re-starting;
- optionally, a third 'closing pressure', denoted the C pressure, can be taken by slowly deflating the membrane soon after B is reached until the membrane returns to the 'lift-off' position; this pressure is identified by the audible signal starting again after deflation.

The pressure readings A, B and C are corrected by calibration to take into account the membrane stiffness. They are then converted into pressures termed p_0, p_1 and p_2. Using p_0, p_1 and p_2, the *in situ* pore pressure and the vertical stress, three 'intermediate dilatometer parameters' are derived, namely:

- the material index, $I_{D;}$
- the horizontal stress index, K_D;
- the dilatometer modulus, E_D.

These 'dilatometer' parameters form the basis of an interpretation via a wide range of correlations to geotechnical properties such as soil type, shear strength, overconsolidation ratio, stiffness, density, etc. **Figure 47.12** shows a typical profile of results in a layered sand and clay deposit. The DMT is suitable for use in sands, silts and clays, where the grains are small compared to the membrane diameter (60 mm), with a very wide range of strengths, from extremely soft clay to a hard soil or soft rock. It is not suitable for gravels, although the blade is robust enough to pass through gravel layers of no more than about 0.5 m thickness. The DMT has also been quoted as being used for a variety of direct applications such as the settlement

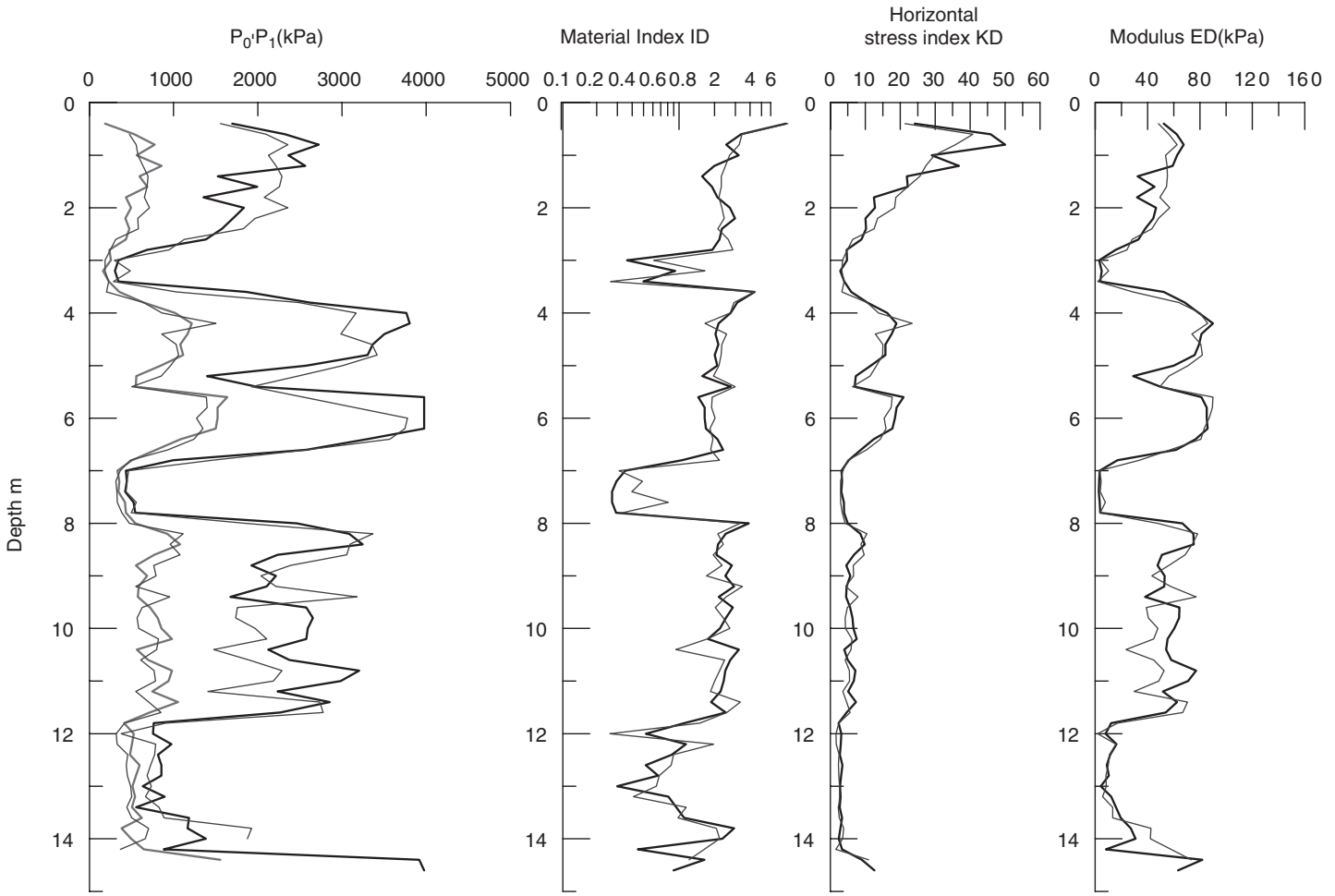


Figure 47.12 Typical profile of DMT results in layered sand and clay

of shallow foundations in clays and sands, the axial capacity of piles, the lateral behaviour of piles, compaction control, the liquefaction of sands and the detection of slip surfaces.

Further information can be found in Marchetti *et al.* (2001) and experience in UK soils is described in Powell and Uglow (1988b). More information is available in the *in situ* test conferences listed in section 47.6 Further reading. As with the interpretation for any *in situ* test, it is important to ensure that the source of the correlations used is fully appreciated and applicable to the ground conditions under investigation.

47.3.4 Pressuremeters and dilatometers

Although a number of different designs and test methods exist, all pressuremeters consist of a cylindrical cell, which is installed vertically and applies uniform radial pressure to the ground via a flexible membrane, connected by cabling and tubing to a control and measurement system at the ground surface. Higher pressure devices of this type, designed for use in hard soils or rocks, are sometimes referred to as 'dilatometers'. The aim of a pressuremeter test is to obtain the stiffness, and in weaker materials the strength, of the ground, by measuring the relationship between the applied radial pressure and the resulting deformation. BS 5930 (British Standards Institution, 2010) and BS EN 1997-2 (British Standards Institution, 2007b) give some guidance.

Three principal types of pressuremeter are in use:

- *Borehole pressuremeter* A borehole is formed using any conventional type of drilling rig capable of producing a smooth-sided test cavity. The pressuremeter has a slightly smaller outer diameter than the diameter of the hole, and can therefore be lowered to the test position before being inflated. Borehole pressuremeters have been widely used around the world, but they are not in common use in the UK. A typical test arrangement for a borehole pressuremeter test is shown in **Figure 47.13** and will be covered by BS EN ISO 22476-5 (in preparation, g). The most commonly used version of this type worldwide is the Ménard pressuremeter (MPM), and will be covered by BS EN ISO 22476-4 (British Standards Institution, in preparation, a).
- *Self-boring pressuremeter (SBP)* Borehole disturbance can have a very great effect on the soil properties determined from *in situ*

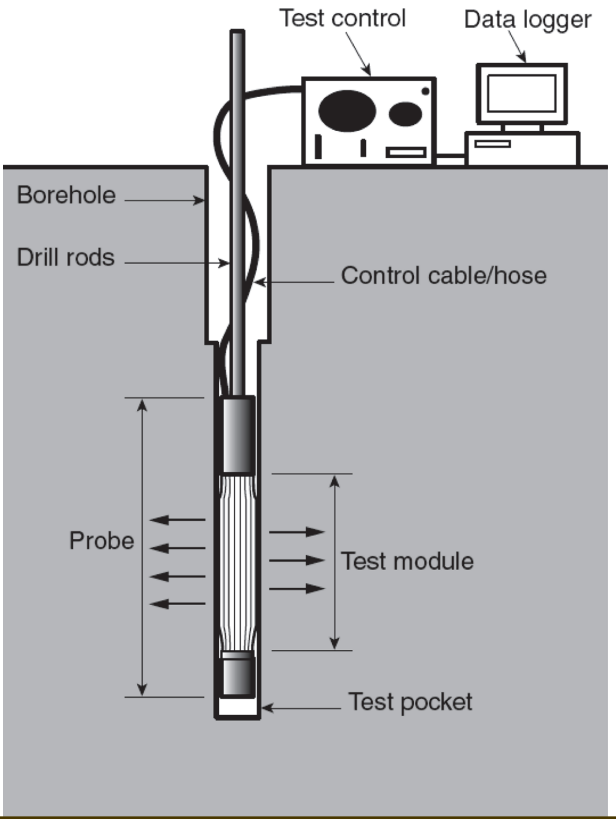


Figure 47.13 Borehole pressuremeter equipment

testing, as already noted in the case of SPT. A self-boring pressuremeter incorporates an internal cutting mechanism at its base; the probe is pushed hydraulically from the surface, whilst the cutter is rotated and supplied with flush fluid. The soil cuttings are flushed to the ground surface via the hollow centre of the probe, as the pressuremeter advances. Self-boring pressuremeters have been widely used in the UK for high-value investigations in stiff clays. They will be covered by BS EN ISO 22476-6 (British Standards Institution, in preparation, b).

- *Full-displacement pressuremeter* (FDP) Displacement pressuremeters are pushed into place and have, to date, been used only rarely in conventional, onshore, site investigations but are seeing increased interest. The push-in pressuremeter (PIP) was primarily intended for offshore investigations, where it was used with wire-line drilling equipment. The cone-pressuremeter (CPM) is a full displacement device mounted above a CPT and is seeing increased interest. Full-displacement pressuremeters will be covered by BS EN ISO 22476-8 (British Standards Institution, in preparation, c).

Because of the wide variety of this type of device, and their sophistication, a detailed description of pressuremeters is beyond the scope of this chapter (see Clarke, 1995 for more details).

A pressuremeter is inserted to a predetermined depth and then expanded. Expansion is either pressure-controlled (most common) or strain/expansion controlled. Pressure-controlled tests expand the membrane using increments of pressure each of which is held for a short period whilst the expansion is recorded. Pressure increments continue until either the maximum system pressure or the maximum expansion of the membrane has been reached. In expansion-controlled tests the rate of radial expansion is controlled and these tests therefore generally require more sophisticated systems (SBPs are generally of this type as are many of the full-displacement systems). At the completion of a test, a plot of applied pressure against radial expansion can be generated, examples of which are shown in **Figure 47.14**. The three types of device generate very different relationships between the pressure and radial strain, although these are essentially different parts of the same pressure–expansion curve.

The principal differences between the three classes of pressuremeter described above lie in the stresses applied to the probe at the start of the test. Borehole pressuremeters start from a horizontal total stress level close to or equal to zero. Self-boring pressuremeters should start the test at approximately the horizontal total stress level in the ground before insertion. Displacement pressuremeters (because they push soil aside during installation) start with a horizontal total stress, which is usually much greater than originally existed in the ground. The increase in horizontal total stress applied during a test is intended to take the soil to failure, although in rock this may not be achievable because of limited system pressure. Unload/reload iterations can be performed during any of the tests.

The interpretation of pressuremeter test data may be semi-empirical (typically the case for borehole pressuremeters and Ménard tests) or based on first principles (because the boundary conditions are well defined, a more rigorous theoretical analysis than other *in situ* tests can be performed, based on the expansion of a cylindrical cavity. There are many methods for interpreting these tests and they can even derive full stress–strain curves of the material from the test results). But essentially, all types of pressuremeter give data that is normally used to calculate, or estimate:

- the *in situ* horizontal total stress;
- the stiffness, normally the horizontal Young's modulus, or shear modulus;
- the strength (where maximum system pressure and volumetric strain permit);
- the drained parameters in sands (although more difficult than testing in clays).

All three types of device are useful and should be considered with regards to cost, reliability and consistency of the data generated.

SBP tests require significant experience if they are to be used successfully. Potentially they can give the best results but are expensive. MPM and pre-bored tests are simpler but require 'consistent' drilling and installation. Softening can occur around the borehole walls. FDP tests can generally be installed with 'repeatable' disturbance but they need further development for some areas of interpretation.

In principle, self-boring pressuremeters should provide the best estimate of the *in situ* horizontal stress and shear modulus,

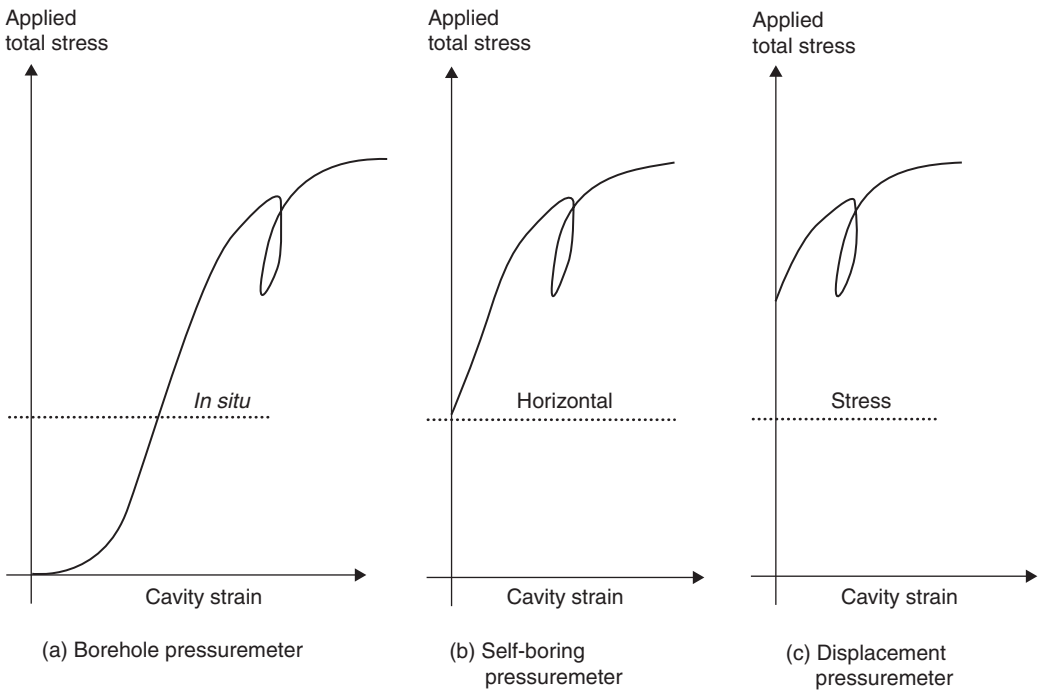


Figure 47.14 Applied pressure/radial expansion curves for three types of pressuremeter
Reproduced from Clayton *et al.* (1995); Wiley-Blackwell

provided (as should be the case) that installation disturbance is controlled. Borehole pressuremeters and dilatometers are the most rugged of the devices, but drilling disturbance and bedding between the cell and the borehole wall make the interpretation of *in situ* stress rather more uncertain, and this can lead to significant underestimates of stiffness. A displacement pressuremeter imposes very significant strains on the soil, and may de-structure it; however, the disturbance is generally 'repeatable'. Experience has shown that, when installed well, in clays the strength derived from all three types is similar and, provided the soil is not cemented, very similar results can be obtained for the stiffness from unload/reload iterations. The pre-bored devices cannot measure as small a strain level as the other devices. When interpretation methods exist, pre-bored and displacement pressuremeters can give reliable indications of the *in situ* horizontal stress (see **Table 47.1**).

The Ménard pressuremeter test generally relies on determining parameters specific to the test method, see BS EN ISO 22476-4 (British Standards Institution, in preparation, a), and these are then used in 'rules' during design, etc., see BS EN 1997-2 Annex E (British Standards Institution, 2007b). As with all these tests the 'rules' should be validated for the soil types under investigation.

More information on the interpretation of pressuremeter tests both for geotechnical parameters as well as their use in direct design can be found in the Further reading section at the end of this chapter.

47.3.5 Field geophysics

Field measurement of the seismic shear wave velocity can be an effective way to determine the shear modulus of made ground, natural ground and fractured rocks, since the relationship between the shear wave velocity and the very small strain shear modulus is a simple one:

$$G_0 = \rho V_s^2 \tag{47.6}$$

where G_0 is the shear modulus at very small strain levels, V_s is the measured shear wave velocity and ρ is the bulk density of the ground.

Suitable methods for determining the required values include:

- continuous surface wave testing;
- down-hole testing (which could be, in suitable ground, a version of CPT or DMT);
- cross-hole testing.

Surface wave testing is discussed briefly in Chapter 45 *Geophysical exploration and remote sensing* of this manual. A brief introduction to these techniques, together with examples of their use and interpretation, can be found in Clayton *et al.* (1995), McDowell *et al.* (2002) and Clayton (2011).

47.4 Groundwater testing

A good knowledge of the groundwater regime and permeability of the ground is essential for any realistic geotechnical

engineering design. The pore pressure regime on a site can be determined using other forms of *in situ* testing (see the section above on cone penetration testing, for example). However, on most sites it will be necessary to install piezometers to measure groundwater levels in the soil. On rocky sites, packer tests can be used to determine the permeability and its spatial variability.

The permeability of a granular, layered, anisotropic or fissured soil is unlikely to be obtained with sufficient accuracy from laboratory tests on specimens from normal diameter boreholes, and therefore *in situ* permeability tests are in widespread use. They can be carried out in soils or rocks, in open boreholes, using piezometers, or in sections of drillhole sealed by inflatable packers. Two common types of test are:

1. rising and falling head tests;
2. packer or Lugeon tests.

47.4.1 Rising and falling head permeability tests

The rising or falling head test is generally used in relatively permeable soils, such as sands and gravels – see BS 5930 (British Standards Institution, 2010) and BS EN ISO 22282-2 (British Standards Institution, in preparation, h). It is usually carried out in a cased borehole or a simple piezometer set in a sealed borehole section, such as a Casagrande piezometer. Where the groundwater level is below the bottom of the borehole or piezometer, the test is started by adding water from a bowser. Where the groundwater level is above the base of the borehole, it is preferable to start the test by bailing water out, since this will reduce the effect of any smear caused by boring. Water-level measurements are then taken at suitable time intervals until the water level returns to equilibrium (see **Figure 47.15**).

The data are interpreted using Hvorslev's basic time lag method (**Figure 47.15(c)**). This requires that the equilibrium water level is known, so that H_0, the driving head at the start of the test, can be calculated. If the equilibrium value is unknown it can be found by trial and error, since for the correct value a plot of $\log_e (H/H_0)$ against (linear) time will yield a straight line. When the time equals the basic lag, then:

$$T = \frac{A}{Fk} \text{ so that } k = \frac{A}{FT} \tag{47.7}$$

where k is the permeability of the soil, T is the basic time lag and F is the shape factor of the test section (normally a pocket of sand).

For a cylindrical piezometer or standpipe sand pocket, or a cased borehole, of length L and diameter D

$$F = \frac{2\pi L}{\log_e \left[\frac{L}{D} + \sqrt{\left(1 + \left(\frac{L}{D} \right)^2 \right)} \right]}. \tag{47.8}$$

If $\log_e (H/H_0)$ is plotted as a function of time, the value of the basic time lag can be found from the straight line at $\log_e (H/H_0) = -1.0$.

The test interpretation assumes that the soil will not swell or consolidate, that the geometry of the test pocket is known, that it contains material that is very much more permeable than the *in situ* soil under test, that there is no smear on the soil wall, and that other test errors, such as those due to air in the soil or pipes, do not occur.

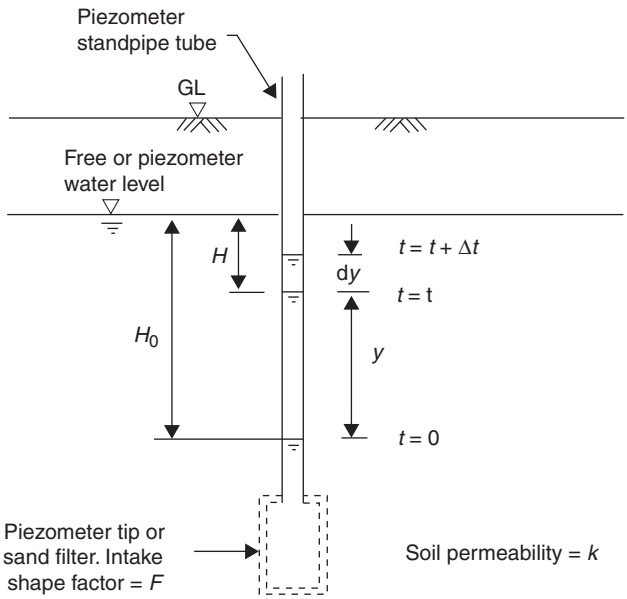
The time necessary to conduct a rising or falling head test increases dramatically as soil permeability reduces. Test times are prohibitively long in clayey silts and silty clays. At the same time, the assumption that there is no contribution from swelling or consolidation to the measured rate of flow becomes unreasonable. Laboratory testing can be used, providing that undisturbed samples can be taken, and these are large enough to contain representative fabric (e.g. laminations, varves and fissures).

47.4.2 Packer tests

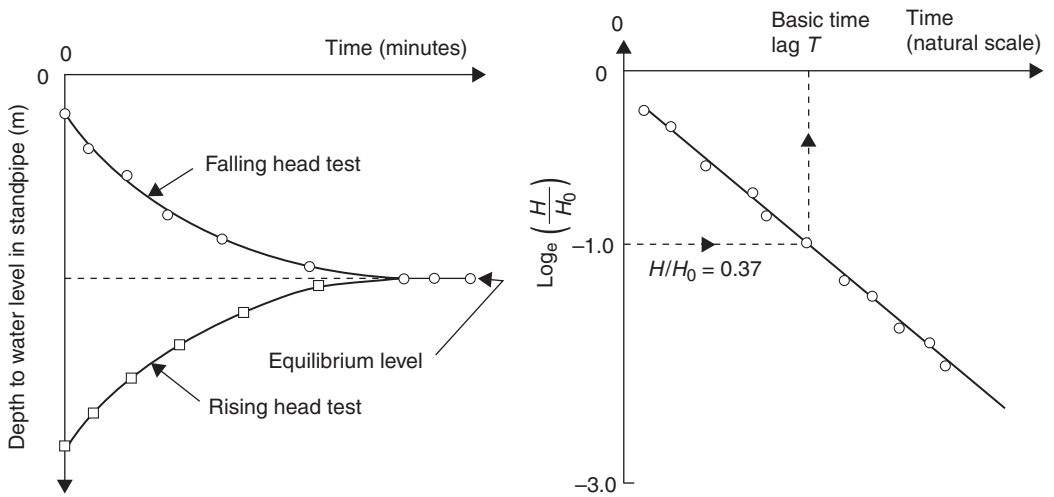
Packer tests, also sometimes known as Lugeon tests, are used to determine the permeability of fractured rock masses – see BS 5930 (British Standards Institution, 2010) and BS EN ISO 22282-6 (British Standards Institution, in preparation, i, j). A test may be carried out in the base of a drillhole using a single inflatable packer to seal off the test section, or after the hole is complete, testing may be carried out at a variety of depths using a double packer to seal the top and bottom of the test section.

The construction of a packer is critical if leakage is to be avoided, and the greater the length of a packer, the more effective the test. A test is carried out by lowering the packer or packers to the required depth and inflating them using gas pressure supplied from a nitrogen bottle. When expanded, the length of each packer should be at least five times the borehole diameter. The test section is often about 3 m long. Packers are supported on drill rods, which are also used to supply water under pressure to the test section. At the top of the borehole the rods are connected via a water swivel to a system for measuring pressure and volume take, and a water pump.

Testing is carried out in constant pressure stages, which cycle up to a maximum pressure and then down again. A maximum pressure should be specified to ensure that hydraulic fracture is avoided. A test is normally carried out using stages such as 1/3, 2/3, 1, 2/3 and 1/3 of the maximum allowable gauge pressure. At each stage, the pressure is held constant and the volume measured over two or more periods of 5 min. Permeability is calculated from the length and diameter of the test section, the measured volume of flow and the net dynamic head applied to the test section. Alternatively, for deeper test sections, the results can be quoted in terms of 'lugeons', where one lugeon is the water take measured in litres per metre of test section per minute at a pressure of 10 kg/cm^2 (= 1 000 kN/m^2).



(a) Rising head test – definition of variables



(b) Rising and falling head test results

(c) Method of plotting results to find basic time lag (T)

Figure 47.15 Rising and falling head permeability tests
Reproduced from Clayton *et al.* (1995); Wiley-Blackwell

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It is recommended this chapter is read in conjunction with

- Chapter 13 *The ground profile and its genesis*
- Chapter 17 *Strength and deformation behaviour of soils*
- Chapter 18 *Rock behaviour*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 48

Geo-environmental testing

Nick Langdon Card Geotechnics Ltd, Aldershot, UK
Cathy Lee (nee Swords) Card Geotechnics Ltd, Aldershot, UK
Jo Strange Card Geotechnics Ltd, Aldershot, UK

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Under current UK legislation and planning regulations, the assessment of a site has to address potential contamination and associated environmental risks and waste management issues. Typically this will involve a qualitative and/or quantitative risk assessment which requires collection and interpretation of appropriate geo-environmental data and relevant environmental surveys. Data are required relating to contamination sources, including the chemical composition of soils, groundwater and soil gas, in addition to potential receptors which may be impacted. The quality of this data depends on the use of good practice in the selection, collection, handling, storage and analysis of samples. The value of the risk assessment further depends on the processing and interpretation of the data. In the context of a ground investigation, there are significant benefits in combining the collection of geo-environmental data alongside geotechnical requirements, provided it is done in an appropriate and targeted manner.

48.1 Introduction

Geo-environmental testing commonly forms part of ground investigations tasked with providing information to support both geotechnical design and contamination issues such as risk assessment and remediation design. Current guidance for undertaking investigation of potentially contaminated sites is provided in BS 10175:2011. The understanding of the ground conditions is paramount to the success of an investigation, and geo-environmental testing forms a major part in obtaining data to develop that understanding. The need for geo-environmental testing can be driven by various factors and the scope is usually determined from the initial desk study supplemented by site observation. The desk study should identify where samples are required for testing because of potential chemical hazards in the ground. Obtaining, transporting, handling, preparing and testing of samples must be addressed when planning and undertaking an investigation.

Typically geo-environmental testing may be undertaken to enable qualitative and quantitative risk assessment of pollutant linkages based on current UK guidance and assessment criteria such as SGVs and GACs, derived using the CLEA model which undertakes contaminant exposure modelling using HCV. Testing may also be required to undertake modelling to derive remediation targets and classification of material for waste disposal. The suite of analysis applied to samples should be bespoke to the site conditions and the purpose of the testing, albeit likely to be based around a generic suite(s) to assist the laboratory and minimise costs. However the validity and value of geo-environmental testing will be questionable if it is not done with full understanding of the reasons behind it, the objectives in undertaking such testing and the factors that influence the quality of data and its assessment.

48.2 Philosophy

The philosophy behind geo-environmental testing is distinctly different from that found in conventional geotechnical testing. Firstly, testing is done with a view to some comparison with criteria set out by national regulators and not for the purposes of pure investigation of a particular quality, such as shear strength or particle size. Secondly, the accuracy to which these determinations are made depends on equipment, sample preparation and the laboratory in a way that even the most leading edge geotechnical testing is not. The testing of blank samples and the routine re-evaluation of results from commercial analytical laboratories is not a process normally seen in geotechnical testing. Thirdly, some criteria set by the regulators, particularly with regard to water standards, cannot be achieved routinely by commercial laboratories. Finally, the testing is regularly specified to prove the absence of a particular contaminant. This is akin to undertaking a plastic limit test on a sand to prove it is a non-plastic material.

Added to this subtle difference in philosophy are the myriad changes in guidance from regulators and the steady improvement in the ability to measure very small concentrations of any particular contaminant. When this is set against a material being tested, usually Made Ground or a dynamically changing fluid source, which are thoroughly non-homogeneous and potentially unrepeatable, the treating of test results in isolation as ‘absolute’ is nonsensical.

Once results are obtained they are routinely processed statistically in a manner that is only now in 2011 being introduced into geotechnical testing in the UK by Eurocode 7.

The objective is to quantify the risk to human health, the risk to buildings and construction materials, the risk to controlled waters, and waste classification. Once these are established and a remediation methodology adopted, there is a requirement for

verification testing against criteria set out in a RIP. The nearest analogy is earthworks control testing, but with tens of separate variables – rather more than are usually tracked in an earthworks contract.

The consequence is that sampling and testing protocols and the impact of variations need to be clearly understood.

48.3 Sampling

Given the variability within soils sampled for geo-environmental purposes, sampling is not an exact science. However, the aim of sampling is to obtain *appropriate* samples for testing: 'targeted' to assess what is likely to be the worst contamination, or 'representative' for waste classification or verification.

Targeted sampling will be based on background information from the desk study (see Chapter 43 *Preliminary studies*) plus observations of the specific ground conditions including visual and olfactory evidence. Representative sampling may be undertaken at regular intervals within an exploratory hole or soil mass (**Figure 48.1**).

Composite samples are rarely used for geo-environmental purposes, unless to test bulk, basically homogeneous, materials. The inherent human factor involved in sampling decisions cannot be eliminated, but good sampling practice can minimise the scope for errors.

48.3.1 Protocols

48.3.1.1 Soils

There are many and varied ways of getting soil samples from the ground. Generally, methods are typically variations on those techniques used to obtain the samples used for geotechnical purposes, as discussed in Chapter 46 *Ground exploration*. However, these variations may be critical to the validity of the samples for the required testing or to the protection of the environment. Some typical examples of good practice are identified below.

- Drilling of boreholes in certain geological conditions may result in the transfer of contaminants to deeper levels by introducing a pollutant migration pathway. In this case, clean drilling techniques may be required in contaminated soils using dual string casing with appropriate seals.
- It is often necessary to add water to cable percussion boreholes to aid drilling. The quantities of water should be kept to a minimum and only clean water used.
- Lubrication of the drilling tools should be kept to a minimum during drilling and only vegetable oils should be used, if absolutely necessary, and their use recorded.
- In the case of rotary boreholes, drilling fluids need careful consideration and recording.
- During window sampling, plastic core liners can also be used to limit the potential for cross-contamination (**Figure 48.2**).
- All samples must be taken using clean equipment, including trowels, spades, gloves and, where necessary, power washers and detergents (e.g. Decon 90) should be used. Equipment cleaning must be done between holes and samples, if necessary, to minimise cross-contamination.



Figure 48.1 Sampling from trial pit

Typically geo-environmental samples are small disturbed samples, but collection into a plastic bag is not appropriate. Samples should be compacted into a container to minimise headspace and sufficient sample volume taken to enable the required testing. Typically a 1 kg plastic tub and a 250 ml glass jar are ample, but larger samples may be needed for some testing suites. Certain organics tests require glass containers and VOCs may require a McCartney vial.

Once a material has been identified for sampling, a full description is required, ideally to BS 5930:1999 and full records kept of the process. Photographs can also be useful. In addition, records are needed of anything that may require the laboratory to take precautions, such as broken glass, possible asbestos or potential biological contamination (details may be found on the Health Protection Agency website). Such samples should be double-bagged, sealed and clearly labelled with reference to the potential hazard.

48.3.1.2 Water

Groundwater (and gas, see section 48.3.1.4) monitoring installations may be installed in boreholes or window sample holes, and not within trial pits. Correct borehole installation is critical for monitoring purposes and construction details will be determined by site conditions. Relevant issues are listed below.

- The well size must be appropriate for the required sampling method.
- The response zone should be placed such that it monitors groundwater in the correct position in the aquifer. This will take account of the potential for free product, i.e. hydrocarbon in liquid or non-aqueous phase, and/or tidal/seasonal fluctuations. In the case of free product, the well screen should extend above the water table such that free product may be captured.

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- Where more than one aquifer is present (e.g. perched waterbody and a deeper aquifer) the response zone must not cross both water bodies. Additionally, a seal (typically bentonite or similar) must be placed between the two water bodies to limit the potential for cross-contamination.
- Well construction materials should not interfere with the prevailing groundwater. PVC casing must be threaded and no glue or other adhesive used to seal the joints, and the filter material should comprise washed chemically inert granular material.

On completion of the installation, a full written log of the well design should be maintained and the well developed to settle the gravel pack, bail out any non-representative water and to cleanse the strata around the borehole. This involves bailing or pumping the well until the recovered water is free from suspended solids. This may require removal of 10 or more borehole volumes. The well should ideally be left for one week prior to purging and sampling.

If required, gas monitoring should be completed prior to water sampling. A record of site conditions should be made, and the well purged to ensure representative water conditions are sampled. Typically at least three times the well volume should be purged prior to sampling. Alternatively, measurements of geochemical parameters such as conductivity, pH, temperature, dissolved oxygen and redox can be continuously monitored during purging until these parameters stabilise. It is good practice to record these parameters during purging and sampling.

Groundwater samples can be taken using a bailer (stainless steel or Teflon) or by using pumping techniques, depending on conditions. Samples should be taken with minimal disturbance and aeration of the sample to avoid chemical changes in the water. All equipment should be cleaned thoroughly between sampling points and dedicated bailers (and string) or tubing should be used for each well.

Sample containers appropriate for the type of testing, as provided by a laboratory, and of sufficient volume, should be used. Containers for VOC analysis should be filled first and all containers should be filled until a positive meniscus forms on the top of the container to avoid air/headspace in the sample.

48.3.1.3 Surface water

When taking surface water samples the following points should be considered:

(i) samples of the water surface should be avoided unless sampling for floating free product;

(ii) sample containers must be submerged below the surface and solid material such as plant debris should be removed;

(iii) avoid taking samples from the bank of the river or stream;

(iv) samples should be taken downstream of the sampling point in the direction of the flow of water;



Figure 48.2 Sampling from open window sampler

(v) sample the flowing parts of the watercourse where possible, avoiding areas of stagnant or standing water.

48.3.1.4 Gas

Gas wells should be installed in accordance with CIRIA 665 and BS 8485:2007 and the final design should be based on the ground conditions encountered. Note that:

(i) it is vital that an adequate bentonite seal is placed around the top of the borehole and should be a minimum depth of 0.5 m;

(ii) pipework comprising threaded joints should be used. Glue or adhesive should not be used to seal pipework.

Typically, bulk gas is measured *in situ* (see section 48.4.2.3), but for laboratory analysis, especially if trace gases are of interest, a sample may be required. There are numerous sampling options; commonly used containers are Gresham tubes (sealed steel containers into which a pressurised bulk gas sample can be collected) or a Tedlar bag (typically 1 litre sample for trace gases) filled via the pump on monitoring equipment (**Figure 48.3**).

There are a number of occurrences in gas monitoring wells which must be borne in mind during sampling as they can impact the results obtained. Firstly, gas can layer in wells, as methane is lighter than air and carbon dioxide is heavier than air. In addition, during construction the soil gases within the well reflect that of atmospheric air. Over time, carbon dioxide and methane, etc., will diffuse into the well. In high permeability soils, equilibrium can be reached fairly quickly. Wells in low permeability soils may take longer to equilibrate. This should be taken into account when planning the monitoring programme.

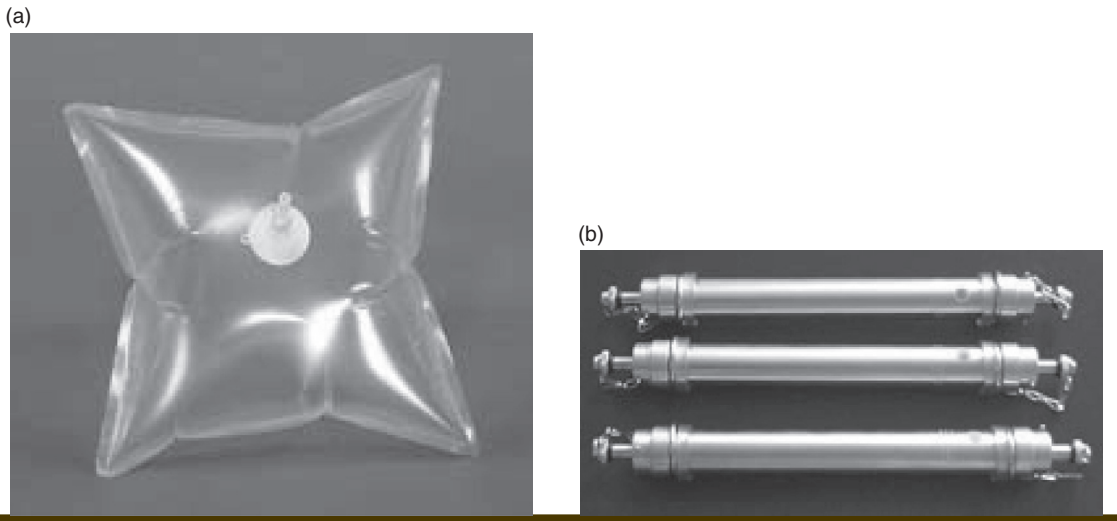


Figure 48.3 (a) Tedlar bag and (b) Gresham tubes

48.3.2 Handling and transport

All sample containers should be labelled accordingly with indelible marker – but not only on the lid. Sample containers should be placed immediately in a cool box containing frozen ice packs (and preferably maintained below 4°C) and dispatched to the laboratory. All samples should be carefully packed to prevent damage during transportation.

Each sample will be recorded on 'chain of custody' documentation, which is required to accompany the samples when transported. The chain of custody form is one of the most important documents in verifying quality assurance during environmental sampling and is often the key weakness in legal cases. This documents the transport and arrival dates of samples at the laboratory and the condition in which they were received. Each laboratory has a standard version.

All details requested on the form should be filled in and a copy retained for the project file for tracking purposes. Once samples have been received at the laboratory, a receipt should be issued, along with confirmation of the testing schedule, as appropriate.

48.3.3 Storage

Sample holding times can significantly affect the quality of results obtained. This is particularly important for certain tests which have limited holding times such as the testing of BOD or volatile organics. Analyses performed outside of the recommended holding time will usually be considered invalid or not representative. Laboratories will advise on holding times for specific analyses. If appropriate, sample containers with preservatives may be used.

48.4 Testing methods

48.4.1 Purpose

Geo-environmental testing not only allows quantification of pollutants in various media, but it also enables an understanding of the geochemical environment in the soil and groundwater to predict the fate and transport of pollutants. It is important that the scope for geo-environmental testing is considered in relation to the findings of the desk study and with due regard for future requirements or more detailed assessments.

It is absolutely critical that the testing method produces results which are relevant for the assessment required, and that limits of detection are below the appropriate assessment criteria. Inappropriate testing is a waste of time and money and can lead to, at best, an over conservative assessment and, at worse, wrong interpretation.

48.4.2 *In situ* testing

In situ testing is primarily used as a support tool in geo-environmental assessments to complement laboratory testing. The benefit of *in situ* testing is that it can provide a large volume of data in a cost effective manner. Real time results also enable 'on the spot' decisions to be made. However, *in situ* testing is not a replacement for laboratory testing, due to issues associated with accuracy, detection limits and reliability.

48.4.2.1 Soils

In situ testing of soils is typically used to fill data gaps in conventional laboratory testing and is also a useful method to delineate areas of contamination for remediation or verification purposes.

There are various *in situ* testing methods currently available, but generally they fall into three main categories, as discussed below.

Hand-held field testing kits

Portable hand-held field testing kits are available for a wide range of contaminants such as hydrocarbons and heavy metals (**Figure 48.4**). All have limitations which need consideration before use. Since many hand-held testing kits are sensitive to

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Figure 48.4 *In situ* testing

various parameters, have a range of limitations and involve the use of dangerous or controlled substances (e.g. solvents, X-ray technology), they must only be operated by appropriately trained individuals.

Mobile laboratory units

Mobile laboratories are a useful resource, particularly on large scale projects, where accurate real time data are needed. Mobile laboratories analyse samples in a similar way to that undertaken in the laboratory and are manned by a suitably qualified laboratory chemist. A broad range of contaminants can be measured depending on the specification of the mobile laboratory. Chemical test results from mobile laboratory units can be UKAS accredited. However, mobile laboratories are generally only economic on larger projects or projects with strict program constraints (**Figure 48.5**). Detailed pre-use discussions with the regulatory bodies are advisable to ensure that the testing methods have been agreed up-front and that any concerns are addressed.

Modified probes

In situ probes can provide real time data in a fast, efficient and economical way to characterise large areas of contamination in soils and groundwater. They also enable decisions to be made



Figure 48.5 Mobile laboratory

regarding targeting of site investigation or remediation and can provide data to give a rapid estimate of the volume of contamination within the sub-surface.

Cone penetration testing, typically used for geotechnical applications, has been modified to detect an array of contaminants in the ground. Techniques have been developed which use fluorescence to identify petroleum hydrocarbons via a modified cone driven using standard CPT equipment in both soils and groundwater. Free phase hydrocarbon product can be detected as well as residual concentrations in soil and it can also provide an indication of the type of petroleum hydrocarbon present (e.g. diesel, petrol, oil). Volatile organic contaminants such as chlorinated hydrocarbons and BTEX compounds can also be detected using MIPs.

48.4.2.2 Waters

In situ testing of waters is also routinely undertaken for various physicochemical parameters to provide information on the geochemical conditions in the waterbody. These parameters are used to determine if conditions are suitable to promote the natural degradation of the contaminants and thereby support the remediation decision-making process. The *in situ* geochemical parameters that are usually measured in groundwaters and surface waters are summarised in **Table 48.1**.

These parameters can be measured at the laboratory. However, *in situ* testing of these parameters is preferred since some parameters, such as dissolved oxygen and pH, are very sensitive to exposure to atmospheric air and changes in pressure – which can result from bringing water samples from depth to ground surface. For these reasons, taking a representative reading from groundwater at depth overcomes various challenges.

Hand-held meters come equipped with probes on long cables that can be lowered into a groundwater monitoring well. These meters can provide an accurate *in situ* reading, though are generally expensive.

Hand-held probes require a water sample to be brought to the surface and decanted into a vessel from which the measurement

Parameter	Description	Reason for measurement
pH	Measure of acidity or basicity of the waterbody	Can affect microbial activity, solubility of contaminants, speciation of metals and can interfere with reduction/oxidation processes
Redox potential	Describes the oxidation state of the waterbody	To determine if conditions in the waterbody are likely to be oxidising or reducing, either of which can impact upon the degradation of contaminants. For example, the natural degradation of some chlorinated hydrocarbons can be enhanced by reducing conditions. For other contaminants (such as benzene), reducing conditions would inhibit natural degradation
Temperature	Temperature of the waterbody	Can affect microbial activity and solubility of contaminants
Electrical conductivity	This measures the ability of the water to conduct an electric current	This can indicate if the water is freshwater, brackish or saline
Total dissolved solids	This is a measure of the concentration of dissolved ions in the water	This is essentially measured by converting measurements for electrical conductivity using a conversion factor. This helps to determine if the water is freshwater, brackish or saline
Dissolved oxygen	This a measure of the amount of oxygen dissolved in the waterbody	Provides an indication of whether conditions in the waterbody are aerobic or anaerobic

Table 48.1 Summary of typical geochemical parameters measured in water bodies

is taken. These are relatively inexpensive and provide indicative information on aquifer conditions, but the disturbance caused during sampling reduces confidence in the results obtained, which should not be relied upon for decision making.

Current best practice combines the use of low flow sampling techniques with the use of a flow-through cell. The low flow sampling results in minimal disturbance and aeration of the aquifer during sampling, and the flow-through cell minimises contact with atmospheric air as readings are taken.

48.4.2.3 Gas

Typically bulk gas measurements are taken using *in situ* measurement techniques in boreholes with specialist equipment capable of measuring gas concentrations and flow. In terms of assessing gas risks for construction purposes, monitoring for concentrations alone is worthless – without the corresponding flow reading.

Prior to monitoring, clean atmospheric air should be run through the instrument. Flow measurements should be taken first, then measurement of gas components at regular intervals until steady state conditions are reached. Records should be kept of atmospheric pressure and air temperature. On completion, the groundwater level should be recorded.

Certain conditions can affect the validity of monitoring results. Wind blowing across the vent outlet of the flow meter can seriously affect the recorded flows, and layering of gas within the borehole should be allowed for, with sufficient time spent to observe changes in gas proportions as gas is extracted from the full depth of the well.

Gas measurements are not limited to boreholes and where an understanding of direct surface emissions is necessary, flux box testing may be of use. This places a sealed container on the ground and emitted gases are trapped within the container for subsequent measurement. A FID is recommended for measuring trace surface gases.

In landfill applications, where information on changes in concentrations within the gas may be critical (e.g. for flaring or utilisation), *in situ* monitoring may be valuable. In-line measurement and datalogger equipment, such as a GasClam, can be used as an alternative to a hand-held monitor for taking regular measurements.

48.4.3 Laboratory testing

Laboratory testing gives an accurate measurement of concentrations of specific chemicals or substances in a sample such as soil, water, leachate or gas. Typically only a small percentage of the sampled material is tested and the portion used for each test is randomly selected. Therefore the validity of the testing is dependent on good sampling practice, combined with the selection of suitable methods. It should also be noted that most samples for chemical testing require sub-division and preparation in the laboratory, either by crushing, drying, solvent extraction or leaching. Therefore all laboratory testing involves a delay between samples arriving and the testing procedure commencing. Typically, in the UK, most testing can be completed within five to eight days. A more rapid turnaround, where possible, incurs additional costs.

48.4.3.1 Methodologies

The choice of method is important. It can affect the result and what it means. Some screening tests are quick, inexpensive and provide the required data, and may be more appropriate than a slower, expensive and more complex, accurate test. The method selected should reflect the future purpose of the data and be able to achieve a LOD below the relevant assessment criteria. If the LOD is too high, all samples may have to be assumed to exceed permitted concentrations, leading to false negatives and overconservative risk assessments. Conversely, too stringent LODs can result in unnecessary expense. It is outside the scope of this manual to discuss all the various

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methods available. All laboratories can provide a list of the sample methodologies they offer, the quality accreditations associated with each method and the limits of detection. These should be checked for any new laboratory used. It should be noted that the LOD is also a function of the quality procedures and statistical manipulation of the data to comply with quality accreditations. The LOD does not necessarily reflect the resolution of the testing machine, although it can mean that the laboratory cannot test to a sufficient resolution, especially where scientific advances refine the assessment criteria and the laboratories have to adapt testing methods as a result. The reason for LODs in excess of assessment criteria should be checked with the laboratory before test scheduling, if they are critical to the assessment.

It is recommended for consistency that the same laboratory is used for the whole of a project, but should this not be possible, the testing methods should be kept consistent between laboratories, so that data are comparable.

48.4.3.2 Suites

For practical purposes, geo-environmental testing is usually broken down into standard suites, such that similar substances are tested together (e.g. metals, PCBs, hydrocarbons, pesticides). Standard suites can also be added together to produce generic suites, as may be offered by a laboratory, or bespoke suites for particular customers or projects. Agreeing a suite of tests is generally much more cost-effective than specifying individual tests on each sample.

However, the appropriateness of suites needs to be checked for each project and, where necessary, parameters added or subtracted from generic suites, based on the contaminants of concern identified from the conceptual site model. Typically, a site without any known contaminative history would be tested for a basic generic suite including metals (e.g. As, Cd, Cr, Cu, Ni, Zn, Pb, Hg, Se, V), basic inorganics (e.g. pH and sulphate), and basic organics (e.g. PAH and phenols). Cyanides, TPH, solvents, pesticides, herbicides, PCBs, asbestos, or any other relevant potential contaminants would be added, depending on site-specific or surrounding conditions. For construction projects where material is to be disposed of off-site, a generic plus targeted suite of testing will be necessary for all Made Ground and waste acceptability criteria testing may also be required.

In the case of unusual substances, the specialist analytical chemists employed by the laboratories can often provide advice on the type and suitability of tests available.

For some testing the methodology must comply with specifications set out in guidance documents and regulations. For example, waste acceptability testing requires testing to a specific method involving several stages of sample preparation.

48.4.3.3 Accreditation

A UKAS accredited laboratory should be used for all geo-environmental testing requirements. Additionally, MCERTS accredited testing procedures should be used, where available, for all testing which will be used for legal or regulatory purposes. However, it is noted that only soils are covered by MCERTS at present and certain tests or determinands may not yet be accredited. This will vary between laboratories.

48.4.4 Complementary testing

In addition to chemical testing, geo-environmental assessment and detailed groundwater quantitative risk assessment in particular require some basic geotechnical information. Typically such information relates to porosity, permeability and moisture content.

48.5 Data processing

48.5.1 Assessment criteria

48.5.1.1 Human health

Current best practice for the assessment of geo-environmental data for human health risk is laid out in the model procedures within CLR 11, and within those procedures, the processing of soil data requires the Chartered Institute of Environmental Health (CIEH) statistical approach. It is necessary to understand the basis and limitations of how such statistical tests are applied so that the scope of geo-environmental testing is suitable to complete such assessments. The current approach uses the calculated mean at the 95% confidence value for comparison against SGVs. These calculated values are derived from the CLEA model which uses a toxicological approach to calculate acceptable soil concentrations for various land uses and assumes a soil organic content of 6%, which is representative of typical topsoil. At the time of writing, only a relatively small number of SGVs have been published for three specific land uses. The SGVs are also generic and conservative values being based on UK defaults for standard receptors and exposure pathways. For anyone wishing to carry out assessments for different parameters or different soil organic matter (SOM), CIEH in conjunction with LQM has published a set of GACs for 40 determinands derived using the CLEA model for SOM at 1%, 2.5% and 6%. However, these too are generic and, if values are needed which are specific to a particular site, GACs have to be derived from first principles using the CLEA model.

48.5.1.2 Waste classification

The scope and, in some cases, the methods of environmental testing required for waste classification are similar, but different to that required for environmental risk assessment. The drivers for testing are the Hazardous Waste Directive (HWD) and the Landfill Directive, which are translated into law in England and Wales via the Hazardous Waste (England and Wales) Regulations (2005) and the Landfill (England and Wales) Regulations (2002), respectively. (Note that Scotland has separate waste management regulations.)

The HWD defines hazardous wastes as those possessing one or more of 14 identified hazardous properties. All wastes are listed in the European Waste Catalogue (EWC) (2002) which includes a list of those automatically deemed to be hazardous,

known as 'absolute entries'. Some wastes may be either hazardous or non-hazardous and are known as 'mirror entries' in the EWC. All other wastes are classified as non-hazardous waste, but this category includes a further sub-division of inert waste.

These classifications correlate with the three categories of landfill, i.e. landfills licensed to accept hazardous, non-hazardous or inert waste. Within the hazardous waste category, some wastes are also unacceptable for disposal to landfill if they exceed certain limits. In addition, under the Landfill Regulations, certain wastes are automatically prohibited from disposal to landfill, e.g. liquid, tyre, clinical, flammable, explosive, corrosive, oxidising and chemical waste.

The process of waste classification is summarised in **Figure 48.6**.

Once a waste has been identified and confirmed as acceptable for disposal, the process continues with assessing if it is hazardous. Once waste has been assessed as either hazardous or non-hazardous, it may require testing for acceptability for landfill. 'Absolute entry' substances only need testing for acceptability to landfill. Material that is non-hazardous and not inert does not need acceptability testing. However, in most geo-environmental/geotechnical contexts, the waste will be a natural soil or Made Ground which is a mirror entry, EWC code 17 05 03* or 17 05 04 – soils and stones. The asterisk (*) denotes the hazardous waste containing dangerous substances. The waste will require testing to determine its component substances and to demonstrate whether it is hazardous or not. This process is based on assessing the waste materials against limit values for various chemicals, determined using the hazard codes and risk phrases allotted to various substances in the waste – as indicated in the current classification, labelling and packaging (CLP) regulations of chemicals.

The acceptability testing is commonly referred to as WAC. This is a suite of soil and leachate tests, the latter being carried out on leachates prepared in a specified two stage process (EN 12457–1 to 3).

It is crucial that testing of the soil for total contaminant concentrations is carried out before or alongside WAC testing. WAC testing alone is generally meaningless as the test is mostly a leachability test and assessment limits relate specifically to inert or hazardous waste categories. The waste category cannot be determined without data on total soil concentrations, see **Figure 48.6**.

WAC testing can be carried out as a full suite, where the waste type is unknown and data are needed for expediency, or limited to a reduced suite specific to inert or hazardous wastes, if known. WAC testing is not needed for non-hazardous waste.

48.5.1.3 Waters

Water test results are generally assessed against a number of criteria depending on the aquatic sensitivity of the site. For a site which sits within a groundwater protection zone or which otherwise provides a potable water supply, standards relevant to drinking water apply. Typically, these are EU or WHO standards. For groundwater or surface water which does not impact on a potable supply, environmental quality standards are more appropriate, but not necessarily less stringent.

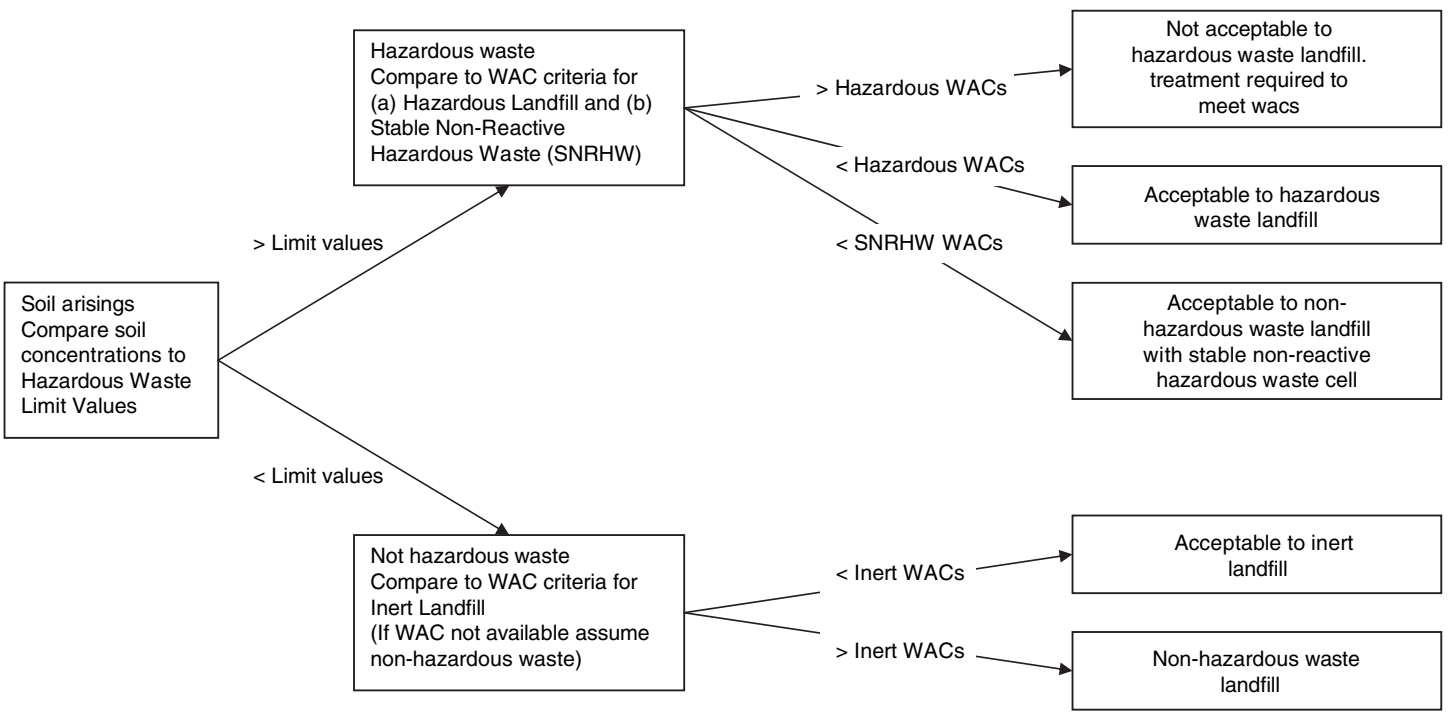


Figure 48.6 Flow chart for waste classification

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Unlike with soil assessment, there is no compulsory statistical analysis required. Where there is soil contamination which may pose a risk to groundwater, it can be assessed initially by comparing leachate test results against the relevant water quality standards. For more detailed quantitative assessment, there are two UK models in common use and advocated by the Environment Agency. The remedial targets methodology is a deterministic groundwater contaminant fate and transport model which estimates the maximum allowable soil and/or groundwater concentration to limit contamination of controlled waters to a stated maximum. ConSim is a probabilistic contaminant fate and transport model which calculates the probability of exceeding a groundwater concentration from a known source.

48.5.1.4 Building materials

Some chemical testing is carried out to assess the suitability of soils for underground construction, such as pipes, concrete, membranes.

For protecting concrete in the ground, specific testing for pH and sulphates is required, with assessment in accordance with the identified groundwater and soil conditions and associated flow charts and tables published in BRE Special Digest 1 (2005). The results of testing for pH, sulphate and occasionally magnesium in soil and groundwater are used to determine the design sulphate class and the aggressive chemical environment for concrete (ACEC) site classification.

If there are high phenols in the soil or groundwater in contact with structural concrete, specialist advice on the design of concrete to resist such conditions should also be sought.

Water supply pipes are particularly sensitive to contamination in the ground. As a rule, plastic pipes will not be permitted within contaminated soil due to the potential for damage to the pipes and/or possible tainting of the water supply. UK Water Industry Research Ltd (UKWIR) has published specific guidance on investigation and analysis to support selection of pipe materials. In addition, each water company may have specific requirements for testing and assessment of soils into which water supply pipes are to be laid. These can be quite onerous, and the water company should be contacted if plastic pipes are proposed to ensure that conditions are acceptable.

In situations where a barrier membrane may be installed on the ground, or be in contact with ground-borne liquids or gases, the durability and resistance of such membranes must be checked against the chemicals present in the ground. This information may be available on datasheets or from the manufacturer. For unusual chemicals, or low specification membranes, this information may not be available.

48.5.1.5 Verification

Verification testing is carried out to confirm whether remediation works have achieved the target concentrations. These are generally a set of values agreed with the regulator, i.e. the local authority or the Environment Agency. Soil concentrations at or below those values are acceptable and concentrations above are unacceptable. Typically these values are based on the result of quantitative risk modelling. The use of SGVs and water quality standards may not be appropriate criteria for remediation and may be overly stringent. Verification data for soil is relatively limited in availability and is compared directly to the target concentrations. Statistical analysis of such data are rare, but is occasionally used if a limited number of tests out of a large batch marginally exceed limits or if such analysis is appropriate given site conditions and usage. However the use of such an approach has to be agreed in advance with the regulator.

Groundwater validation data are generally collected over a period of time, typically ranging from three months to five years, to assess trends as well as direct comparison of concentrations against targets.

48.5.2 Rogue data

All analytical results should be reviewed by a suitably qualified person to check for potential errors or unusual results; this would include:

- looking for transcription errors or missing data;
- considering if the data are within the expected or valid ranges for that contaminant or the site (based on previous monitoring rounds);
- understanding the site conditions (e.g. potential for contamination, soil type). Do the results match the observed contamination such as free product?
- looking at potential conflict between results (e.g. low PAH concentrations but high aromatic TPH concentrations, or the presence of ammonia and high dissolved oxygen);
- assessing other field measurements which may explain spurious results (e.g. high suspended solid content).

Identified rogue data should be checked with the laboratory and, if necessary, samples re-tested, or re-sampling undertaken to confirm the results. The majority of anomalies will be readily explained by re-testing or by the application of common sense; others will actually appear correct, only because of the absence of an explanation to the contrary. Contamination concentrations can, however, radically vary across a relatively small area, such as found in a 'hot spot'. In some circumstances, such as with volatile material, concentrations will also vary significantly with time and exposure.

48.6 Quality assurance

48.6.1 Site model

It is a requirement for a CSM to be developed for an individual site as part of a planning condition in the UK. The source, pathway and target for identified groups of contaminants have to be set out and one of the most simple processes for any QA checking is to compare the CSM with the type and amount of testing done in any investigation. The identifying of the potential for a contaminant plume of a specific hydrocarbon (which is not complemented by testing of the water for a specific range

of hydrocarbons across a site in a number of places) instantly places the quality of the investigation into doubt.

48.6.2 Sampling strategy

The distribution patterns of sampling positions are much loved by statisticians and academics alike. Unfortunately they assume a homogenous media in which the contaminant lies – and a client with a very large testing budget. Both are rare. The locating of previous contaminating processes from historical maps and plans becomes crucial and the numbers of tests becomes a matter of judgement, not statistical random sampling. That said, it is easier to identify the wrong strategy rather than specify a generic and therefore inadequate ideal. The following should act as a guide:

- Do not take samples only from shallow depth when fluid contaminants such as diesel might have spread downwards into natural horizons. Many environmental investigations sample only the top 500 mm of a site.
- Do not just test the man-made soils, but test the underlying natural soils to establish if contamination has migrated and impacted other soils.
- Do not just focus testing only on potential 'hotspots' such as around buried tanks or process building, but look to establish the potential for 'background' or 'unaffected' levels.
- Do not ignore the groundwater; remediation of this can be many times more expensive than dealing with soils.
- Do not ignore soil gas as a remediation issue in itself and an indicator of other contamination.
- Samples can be too few in number, particularly on small sites and can overwhelm the interpretation on larger sites without a clear assessment of the causes of distribution.

48.6.3 Laboratory

Laboratories are commercial organisations and it should be expected that despite accreditation and various levels of quality assurance procedure their performance and reliability will vary significantly. It is important to consider that procedures are insufficient and that human and mechanical errors will occur at times in the best of laboratories.

As a principle, a single laboratory should handle the testing from one site from investigation to remediation. Indeed a known and planned throughput can see economies in scale.

On large projects it is recommended that the laboratory is visited and inspected and testing requirements, timescales and alternative options of analysis discussed. In certain circumstances, it might also prove necessary for a backup laboratory to be identified and compatibility of analytical techniques and sample preparation thoroughly checked.

Laboratories should be able to provide appropriate accreditation across the widest range of testing suites.

A principal safeguard is the 'believability' check by an experienced geo-environmental engineer who should be able

QC sample location	Duplicate QC sample	Blank QC samples	Standard/spiked QC samples	Errors or variability detected
Waterbody	Sampling duplicate (i.e. repeat entire sampling procedure)	Not possible	Not possible	Total of: purging/short term natural variability plus errors below
Sampling equipment	Equipment duplicate (i.e. repeat use of sampling equipment)	Equipment field blank1	Equipment field standard/spike1	Total of: sampling equipment/some short term natural variability (in the case of duplicate), plus errors below
Prior to treatment (e.g. filtering/preservation)	Pre-treatment duplicate (split sample prior to treatment, then treat both samples identically)	Pre-treatment field blank	Pre-treatment field standard/spike	Total of: field treatment (filtering/preservatives), plus errors below
Prior to bottling	Post-treatment duplicate	Post-treatment field blank	Post treatment field standard/spike	Total of: ambient conditions plus errors below
Prior to transport	Not possible	Trip blank	Trip standard/spike	Total of: handling and transport plus errors below
Sample bottles	Not possible	Bottle bank (i.e. place deionised water in bottle and submit for analysis)	Bottle standard/spike (i.e. place standard or spiked sample in bottle and submit for analysis)	Total of: bottle material and preparation plus errors below
Delivery to laboratory	Laboratory duplicate	Laboratory blank	Laboratory standard/spike	Total laboratory errors(1)
Type of errors detected	Random only	Random and systematic gains only	Random and systematic gains and losses	

(1) Only possible if equipment is removable. For dedicated sampling equipment, this QC sample becomes less important. N.B.: this table only relates to the sampling process. Further QC samples should be prepared in the laboratory in order to detect errors during the laboratory handling and analytical process.

Table 48.2 Summary of QC sample types

Data taken from Environment Agency (2000) © Crown Copyright

QC sample type	Advantages	Disadvantages
Duplicate	• Sampling process itself can be duplicated (sampling duplicate), providing information on errors in the entire sampling/analysis process. • Relatively easily performed. • Can be applied for all determinands.	• Only detects random errors; systematic errors are not detected.
Blank	• Easily performed. • Can be applied to all determinands. • Detects some random and systematic errors.	• Cannot be applied to initial sampling. • Only detects gains in determinands; losses are not detected.
Standard/spike	• Detects all random and systematic errors from point of QC sampling.	• Requires laboratory prepared standard solution. Can be more difficult to perform. • Each sample usually applies to only one determinand.

Table 48.3 Comparison of QC sample types

to critically appraise the results received and identify rogue data.

However, on certain projects it may be necessary to undertake quality control sampling to provide a measurement of data errors and variability in sampling. There are a number of ways of taking quality control samples including duplicate samples, blank samples and standard/spiked samples. The types of quality control samples and the types of errors they detect are summarised in **Table 48.2**.

A summary of the advantages and disadvantages of the three quality control sample types are presented in **Table 48.3**.

48.6.4 Checking and review

The number of data points and the statistical manipulation of them can obscure the random error from the checking and review process and it is important that there is a specific independent data verification check done before conclusions are drawn from the processed raw data. Ideally this is done as part of the initial calculation process by staff of similar experience. More senior staff tasked with the review of the processed data can only realistically select random samples and follow these through set processes. As with all checking operations, the process is as valuable as the experience of the individual carrying out the check.

It should be expected that the final review should be signed off by an appropriately professionally qualified individual holding chartered status as an environmentalist or similar relevant profession.

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48.7.1 Further reading

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48.7.2 Useful websites

CABERNET (Concerted action on brownfield and economic regeneration network); www.cabernet.org.uk/index.asp?c=1124
CL:AIRE (Contaminated land: applications in real environments); www.claire.co.uk
ContamLinks, portal for quality information about the assessment, management and remediation of contaminated land; www.contamlinks.co.uk/index.htm
Environment Agency; www.environment-agency.gov.uk
Environment Agency publications; http://publications.environment-agency.gov.uk
EUGRIS, portal for soil and water management in Europe; www.eugris.info/index.asp
Health Protection Agency; www.hpa.org.uk
NICOLE (Network for industrially contaminated land in Europe); www.nicole.org/index.asp
Portal for contaminated land information in the UK, CIRIA; www.contaminated-land.org/index.html

It is recommended this chapter is read in conjunction with

- Chapter 8 *Health and safety in geotechnical engineering*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Glossary

BOD	Biological oxygen demand
BTEX	Chemical compounds benzene, toluene, ethylbenzene and xylene. BTEX are constituents of petrol
CLEA	Contaminated land exposure assessment
CLR	Contaminated land report
CPT	Cone penetration test
CSM	Conceptual site model
FID	Flame ionisation detector
Free Product	Hydrocarbons in a liquid phase i.e. 'free' or non aqueous
GAC	Generic assessment criteria – an assessment value derived by non-government agency using a contaminant fate and transport model – usually CLEA in the UK
Hazardous waste	Waste which, because of its quantity, concentration or characteristics, may be hazardous to human health or the environment when improperly treated, stored, transported or disposed of
HCV	Health criteria value – typically in the form of an index dose (ID) or tolerable daily intake (TDI), this is the estimate of the daily intake of a contaminant that can be experienced over a lifetime without appreciable health risk (or cancer risk in the case of an ID)
Inert waste	Waste which is neither chemically nor biologically reactive and will not decompose
LOD	Limit of detection – lowest value to which a test result is reported
MCERTS	Monitoring certification scheme introduced by the Environment Agency
MIP	Membrane interface probe
Non-hazardous waste	Waste which is neither hazardous nor inert

PAH	Polyaromatic hydrocarbon
Pollutant linkage	Interaction between a contaminant source, transfer pathway, and vulnerable receptor. The linkage only exists if all three are in place
Purging	Process of removing stale water from a borehole to ensure representative sampling
QC	Quality control
Redox	Reduction oxidation potential of water – measure of how likely chemical reactions are to occur
RIP	Remediation implementation plan
SGV	Soil guideline value, an assessment value derived by the Environment Agency using a CLEA model
TPH	Total petroleum hydrocarbons
UKAS	United Kingdom Accreditation Scheme
VOC	Volatile organic compound a human health assessment value
WAC	Waste acceptability criteria
XRF	X-ray fluorescence

Chapter 49

Sampling and laboratory testing

Chris S. Russell Russell Geotechnical Innovations Limited, Chobham, UK

The development of a good ground model relies upon a successful ground investigation with appropriate sampling techniques and laboratory testing. The involvement of all stakeholders in the process, open communication and supervision are essential for high quality parameter determination for the foundation or temporary works design. Various sampling techniques and their validity for parameter determination are discussed in a process-based fashion along with an insight into the various laboratory tests available. The list of laboratory tests is not exhaustive but lists many of the common test types (and some specialist considerations) associated with modern design requirements from low-rise buildings to advanced construction. The reader is given the main principles of sampling and sample disturbance with reference to the effects on laboratory testing and parameter determination which may be used as a basis for investigations worldwide.

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49.1 Introduction

From previous chapters and sections the engineer will have created and planned the site investigation and have a list of requirements needed to satisfy the construction program from the initial desk study phase through to the finished construction (see Chapter 4 *The geotechnical triangle*). These plans will have been laid out in a 'best practice' manner but should allow for some deviation especially when dealing with the 'unknowns of nature' in what lies beneath the surface. Plans that are too rigid may lead to problems later when dealing with the running of the project and certainly may cause problems in the ground investigation phase. The budget for ground investigations are always only a few percent of the overall project cost but it is here that major savings can be made in the overall design phase if thought and care is taken.

The design of the ground investigation will lead on from the desk study which will have identified the expected stratum and ground conditions. The ground investigation will verify these conditions and identify any deviations which may require further attention or categorisation leading to a ground model suitable for the engineering or design purposes intended. Laboratory tests are routinely used to calibrate ground models, however, *in situ* conditions (sometimes dominated by discontinuities or complex horizons) may require more expensive and time-consuming field tests for full categorisation (see Chapter 47 *Field geotechnical testing*). This sounds simple on paper but is in fact crucial for the design and construction stage. Failings here could have disasterous consequences. If in doubt seek advice. It should be noted that this chapter pays attention to the physical properties of the ground and so does not address testing for chemical properties or ground contamination (refer to Chapter 48 *Geo-environmental testing*). For further information with regard to the task/project in hand, you may wish (amongst others) to refer Chapter 13 *The ground profile and its genesis*; Section 3 *Problematic soils and their issues*; Chapters 43 *Preliminary studies* to 46 *Ground exploration*.

49.2 Construction design requirements for sampling and testing

The categorisation of the site should be comprehensive and provide the best possible parameters for the foundation design whether it be for simple load-bearing calculations for a strip footing through to finite element analysis which are often required for more complex or fragile construction. The required parameters drive the testing schedules for laboratory testing in order to gain accurate knowledge of the physical (and chemical – refer to Chapter 48 *Geo-environmental testing*) properties of the site (the ground model) to indicate uniformity/non-uniformity of the ground (both laterally and with depth). It is this coverage which is required for any form of foundation design or physical modelling. Soil tests should be identified to provide the parameters required for design.

Basic categorisation can be carried out on site through trial pitting, but drilling and sampling will be required for depth profiles to be identified. Each site or contract should be treated as unique and specifications must be reviewed for each contract in light of this. The complexity of the construction and the nature of the ground play a most important role in the parameters required. Interestingly the process of desk study followed by ground investigation and subsequent laboratory testing has required much thought for the layout and sequence of this chapter. It is the output in parameters required for the design that will dictate the sample types which need to be taken during the initial ground investigation phase, or a two-phase ground investigation may be required if the construction is complex or fragile. In this respect the order of description in this chapter has had to be reversed as the parameters for design will dictate the test types and therefore the sample quality which will finally control the sampling methods and preparation of the samples for the tests. In other words, think about what you need before you try and achieve it. The cost savings are in the planning of this and it can be expensive (apart from commercially embarrassing) to get this wrong. The parameters (and some associated tests) you may require can be summarised as follows.

49.3 The parameters and associated test types

In their simplest form, the results of laboratory testing may be used to categorise our area of interest and identify uniformity or non-uniformity of ground characteristics both laterally and vertically whilst more advanced parameters are based around the stability or reaction of the soil with regard to the loading (or unloading) of the construction, both in the short and long term. These values are related to soil strength and stiffness (possibly anisotropy), compressibility and permeability. For the short-term requirements and very low permeability materials we may be more interested in undrained scenarios whilst for the long term or high permeability materials we would be more interested in the fully drained conditions. These properties all vary with different stress states and again the ground investigation should be targeted to gain knowledge of these so that laboratory testing can be representative of the *in situ* conditions and be used with confidence in the foundation design stage. Test standards are listed in the appendix to this chapter for most common laboratory tests and some history/background of these tests and their basic methodology can be found in the *Manual of Soil Laboratory Testing* (Head, 1986).

49.4 Index tests

These are the most common and routine of all site characterisation techniques and are used as a profiling tool both vertically and laterally to build the ground model suitable for the project in hand. They may be carried out using both disturbed and undisturbed material and are relatively cheap and quick to perform. Index tests may be used to support 'expected' basic ground behaviour and identify where more expensive tests (or increased parameter interest) may be required for design. Such routine tests generally comprise moisture content determinations, particle-size distribution and Atterberg limits. When combined these provide very useful profiling tools which can also be used to calibrate ground models and verify the results of any further laboratory tests to be carried out. They provide information which can be combined with drilling logs to identify and corroborate with the height of the water table, variation of soil type and the expected soil behaviour, so giving clarity to the ground model.

49.4.1 Moisture content

This is the simplest and cheapest of the soil tests to be carried out in laboratories and can be carried out on both undisturbed and disturbed samples. The test consists of a small sample of soil being weighed before and after drying to determine the ratio of solid particles to water. Interestingly though, it is the moisture content of soils or weak rocks which often dictates or certainly can dominate their engineering behaviour. The 'engineering' of moisture content only improves the workability or placement of some materials (say for compaction in a landfill liner or dam core) but can cause catastrophic failure through loss of shear strength/cohesion if calculated or carried out incorrectly. Moisture content profiles of depth and lateral distance can be used to indicate zones of differing soil properties. Clays with 'high' moisture contents are often soft (or softened) compared to their drier counterparts (with identical mineral composition). They are also more likely to compress and collapse. The effect of desiccation and the possibility of ground heave can be identified through moisture content profiling to nearby vegetation or drawdown situations. Higher localised moisture contents can even be used to identify the location of broken drains and water pipes which may be the cause of undermined foundations (amongst other engineering problems).

'Natural' moisture content determinations require the material to be tested to be sealed in its natural state. This may appear obvious, but many people get it wrong. The sample should be taken from the ground at whatever depth without the influence of outside water/drill fluid/evaporation and should be immediately sealed in a fully airtight fashion. If the bag is not sealed immediately it will change its moisture content and be unrepresentative (if it is raining, water may enter the sample, and if it is warm and sunny then water may evaporate from the sample). Cohesive materials recovered from rotary cores or where drill flushes have been in contact with the material should be sub-sampled away from the periphery of the material (in contact with the drill flush). The natural moisture content in these material types will be preserved in the centre of the core for a while due to their low permeabilities. Conversely it is almost impossible to gain a natural moisture content of many non-cohesive materials (especially gravels) as the high permeability of such materials prevents retention of the water during sample extraction from the ground. Thought should

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also be given to the materials scheduled for these tests, especially if they are likely to contain hydrated minerals such as gypsum. In such instances the oven drying temperature used in the laboratory determination should be below the level at which such minerals 'dehydrate' or release water from their crystal matrix (if present). If the oven temperature is above 110°C (hence 105–110°C for standard BS 1377 determinations) other volatile fluids will be evaporated other than water leading to erroneous test values. For materials containing (or suspected to contain) gypsum the oven temperature should not be more than 80°C. The drying stage is complete when successive weighings are within 0.1% at four-hour intervals (BS 1377:Part 2:1990). Saline pore waters can also lead to incorrect determinations and in such instances other tests may be more appropriate.

49.4.2 Atterberg limits

These are used to classify fine-grained soils and commonly identify two of the original seven limits defined by Albert Atterberg. The limits are based upon the moisture content of the soil and can be carried out on both undisturbed and disturbed samples. The plastic limit is the moisture content at which the soil changes from a semi-solid to a plastic state whilst the liquid limit is the moisture content at which the soil changes from a plastic to a viscous state.

Liquid limit determinations are carried out either by measuring the penetration of a calibrated cone into a known volume of fully mixed material at four increasing moisture contents (four-point cone method) or by 'bumping' material in a calibrated Casagrande system using a grooving tool again at four different moisture contents. There are alternatives to the four-point systems described here, but they are not ideal. The plastic limit (PL) is determined by the point at which soil can be 'rolled' in a calibrated way to form a thread 3 mm in diameter which has shears both transversely and longitudinally. This part of the test (PL) may yield variable results due to differing operators and levels of experience. This is a basic test but not a simple one to carry out!

The difference between the plastic (PL) and liquid limit (LL) is known as the plasticity index (PI). The relationship allows approximate determinations of compressibility, permeability and strength and is therefore very useful for soil classification. The derived plasticity index (PI) can also be used to determine the amount of clay present. High PI values indicate significant clay contents whilst low PI values indicate the dominance of silt particles. A PI of zero indicates the absence of both clay and silt and is termed 'non-plastic'. Generally the higher the PI value the greater the soil's potential to change volume. High PI values would signify a large volume change when wetted and large shrinkage when dried, etc. This general rule, however, does not take into account the presence of particles larger than 425 μm (removed by sieving before the test commences) and so the modified plasticity index (I_p') is often more appropriate, but only for overconsolidated clays. The calculation for the modified plasticity index was founded by the Building Research Establishment (BRE) and is:

Modified Plasticity Index (I_p') = PI × (% <425 μm/100%).

The National House Building Council (NHBC) has used this same calculation but has modified the percentages which identify high, medium and low volume change/shrinkage potentials. Values should be recorded as percentages as the general terminology of high, medium and low I_p' values are slightly different between the BRE and HSBC references. It can be seen very quickly that such tests can yield very good information and confidence in the material properties of fine-grained materials, especially when profiled.

49.4.3 Particle size distribution (PSD) analysis

This test can be carried out on both undisturbed and disturbed samples. A PSD determination is the mass of particles within designated size ranges expressed as a percentage of the complete sample mass. The range of sample sizes split the soil into its component groups ranging from clay to silt, followed by sand and gravel upwards in size (through to cobbles and boulders). For a complete analysis two distinct test types are performed. For particles larger than 63 μm (for British Standard Tests, 75 μm for ASTM standards) the material is graded by passing through sieves of decreasing sizes. The definitive method for these 'coarse' grains is by wet sieving whilst a quantative test may be carried out by dry sieving (for soils containing insignificant quantities of silt and clay). For wet sieving the particles less than the smallest sieve size are washed from the material and retained for the second part of the test. Particles less than the smallest test sieve are then graded by settling from suspension (with time) in either the hydrometer test or by pipette methods.

From the visual description of soils we make estimations of the percentages of the various sediment sizes which make up our sample. The PSD determination scientifically derives the exact percentages of each soil size fraction within the sample. It is therefore possible that a visual description may be slightly different to that recorded from a PSD analysis, but they can be used to calibrate each other and expose inaccuracies in logging and drilling records. The particle size distribution of a soil will also indicate the permeability and, possibly, compressibility characteristics to be expected from other tests which may allow test specifications to be designed for the material types in question.

Beware: it is not unknown for the sampling of some materials to be carried out badly, especially when retrieving granular material from depth. It is very easy to wash out the fines in the drill fluid or allow the sample tube to drain out water (carrying away fines in suspension) which will lead to an inaccurate PSD analysis. The author has even seen junior lab technicians pour the coarse material from the sample container/bag to be tested and leave the fine sediment in the bottom to be discarded. Both these instances would lead to a very inaccurate PSD analysis and an erroneous judgement of material properties.

Many other index tests exist and may be used in combination or with the main three tests listed above in order to complement soil characterisation. It is most important to remember that any test result is only as good as the representative sample taken and delivered to the laboratory and should be representative of the stratum from which it was taken. Coarse material should be taken in large quantities in order to be representative (see BS 1377:Part 2:1990 for required sample sizes for PSD and other analyses).

49.4.4 Compaction-related tests

These are a series of tests which identify the relationship of density (often with changing moisture content), with a known compactive force. These tests are mainly carried out on disturbed material or material to be classified for engineered fill (which by their nature are 'disturbed'). Compaction itself is a process where the density of the soil is increased by packing the soil particles closer together and so reducing the volume of air (without significantly changing the moisture content). The addition or reduction of moisture content for each test stage simply alters the strength characteristics of the soil and its 'compactibility'. These tests are common for field design of engineered fills as they will provide optimum moisture contents for the fill material with regard to compactive effort available (see Chapter 75 *Earthworks material specification, compaction and control*). Common forms of these tests for varying engineering uses and soil types are as follows:

- determination of dry density/moisture content relationship (2.5 kg rammer);
- determination of dry density/moisture content relationship (4.5 kg rammer);
- determination of dry density/moisture content relationship (vibrating hammer);
- determination of maximum and minimum dry density;
- determination of moisture condition value (MCV);
- determination of California bearing ratio (CBR);
- determination of chalk crushing value (CCV).

Please refer to the relevant current standards for full descriptions of the above methods with regard to the soil types to be tested. Of the tests listed above it is only the CBR which provides an empirical strength criterion (CBR value), and is often associated with pavement construction (see Chapter 76 *Issues for pavement design*).

49.5 Strength

This is defined as the limiting shear stress that a material can sustain as it suffers large shear strains (Atkinson, 2007). The response of the soil in both strength and stiffness are related to the 'state' of the soil. This state is related to the density of the material and stress level which is linked to pore pressure, and therefore, 'effective stress'. It is interesting that so many parameters and stress states are closely linked but all ultimately controlled by the principles of effective stress. 'All measureable effects of a change in stress, such as compression, distortion, and change of shearing resistance, are due exclusively to changes of effective stress' (Atkinson, 2007). The following is a list of laboratory tests including triaxial and direct shear types (amongst others). Triaxial tests are a family of tests whose subtleties are controlled by varying boundary, drainage and loading conditions, but 'appear' to use similar equipment.

49.5.1 Triaxial test types

For basic boundary, drainage and loading conditions see **Figure 49.1**.

- *Unconfined compressive strength* (UCS). This is a total stress test (no pore pressure measurement) and is carried out without radial confinement. It may also be termed 'unconfirmed' compressive strength, and unfortunately has the same acronym associated with the uniaxial compressive strength (UCS test) carried out on rocks. Although the applied stresses are all the same in these tests the standards, methods and test equipment used for soils and their rock equivalents are distinctly different.
- *Unconsolidated undrained triaxial test* (UU). Again this is a total stress test as pore water pressure is not measured. For this test a radial (confining) pressure (σ_3) is applied to the sample and is of a magnitude which relates to the depth of sample origin. Shearing rates are standardised for these tests and reference should be made to the relevant regional standards for further information. This test should not be confused with a UUP test (see below) which often goes under the same name.
- *Unconsolidated undrained triaxial test with pore pressure measurement* (UUP). An effective stress test which gives the undrained shear strength for the material. Be aware that the effective stress measured for such tests may not be representative of the mean effective stress of the material *in situ* due to the effects of sample disturbance. For more advanced testing the pore pressure may be measured both at the base and mid-height of the sample.
- *Isotropically consolidated undrained triaxial test* (CIU). As above but the sample is isotropically consolidated to a mean effective stress relevant to the *in situ* depth of the sample or a particular stress/depth condition which is to be modelled.
- *Isotropically consolidated drained triaxial test* (CID). This is for the 'drained' or long-term condition of the above test. For clay materials this may be a test of very long duration due to the very low permeability associated with such particle sizes and mineralogies, but for sands and free-draining materials it is usual to carry out drained triaxial tests rather than undrained shearing because the short- and long-term conditions should approximate (due to the high permeabilities). Undrained conditions are unusual for sands (unless in a fully confined state) within the ground unless one is trying to model a very specific ground or construction condition. Undrained shear tests in non-cohesive materials cause immediate dilation of the material which is often unrepresentative of field conditions.
- *Anisotropically consolidated triaxial tests* (CAUC, CAUE, CADC, CADE –see section 49.6).

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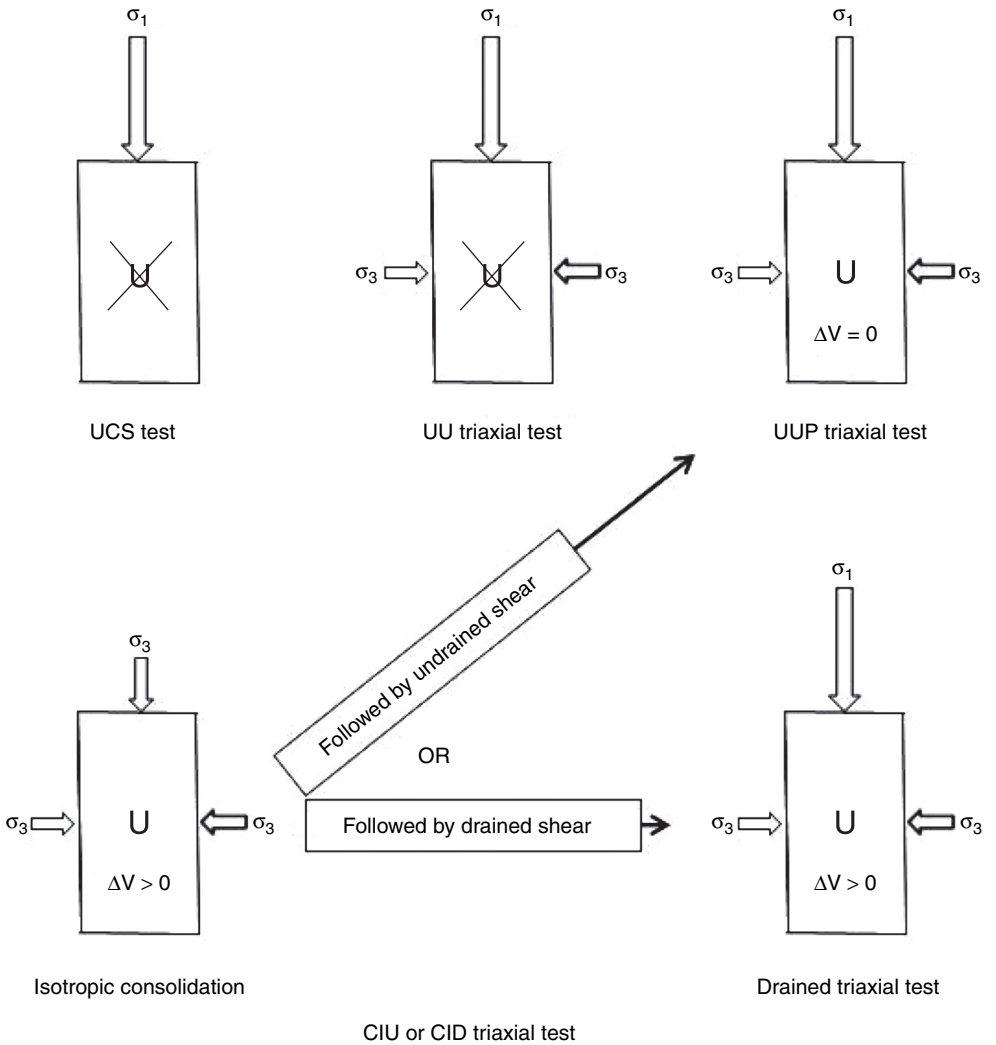


Figure 49.1 Triaxial test: boundary and loading conditions

Reference should be made to Chapter 17 *Strength and deformation behaviour of soils* for the interpretation of different strength parameters and 'ideal' triaxial tests. All of the triaxial tests listed above require undisturbed samples to be taken. Information and guidance is given later in this chapter as to the sampling types and methodologies which should be planned from the outset of the ground investigation. It can also be seen from the list of tests above that rates of testing are mentioned. For effective stress tests pore water equalisation is required in order to measure strength and stiffness correctly (see section 49.6).

Figure 49.2 shows a typical triaxial loading frame with a modern cell pressure and backpressure control system on the left and the transducer logging system on the right. Historically, pressures were applied manually and the measuring instruments were read manually by either reading dial gauges or writing down the specific outputs from digital readout units. Modern technology has advanced triaxial testing so that stress control and instantaneous measurements of transducers are becoming increasingly the norm. Load measurement can also be carried out in several different ways in laboratories and **Figure 49.3** shows the types of load measurement commonly available.

The original external load measuring devices commonly used were in the form of a load ring whose calibrated deflection could be read manually by an operator. These were superseded by load rings with integral digital readouts which can be manually read or their output logged by computers and read remotely. Electronic load measurement devices have become common now, but as with all previous versions the drawback of all of these is that they are generally not waterproof and therefore need to be mounted externally to the cell. This leads to the load transducer measuring the friction of the ram as it passes into the cell leading to errors in load measurement. Calibrations of ram friction can be made to minimise this error; however, slight non-concentric loading can lead to the

ram 'sticking' and giving 'false' load readings. It is possible to utilise rotating bushes where the ram passes into the cell but these are not without problems themselves. The ultimate load measuring device is the submersible load cell which is mounted on the end of the ram and remains in direct contact with the sample within the cell whilst any loading occurs. In essence any load which is applied to the sample will also be seen by the submersible load cell without any external effects and so is the ultimate load measuring instrument. The cost of such devices is often prohibitive in many institutions and so external load measurement is most likely to be the norm.

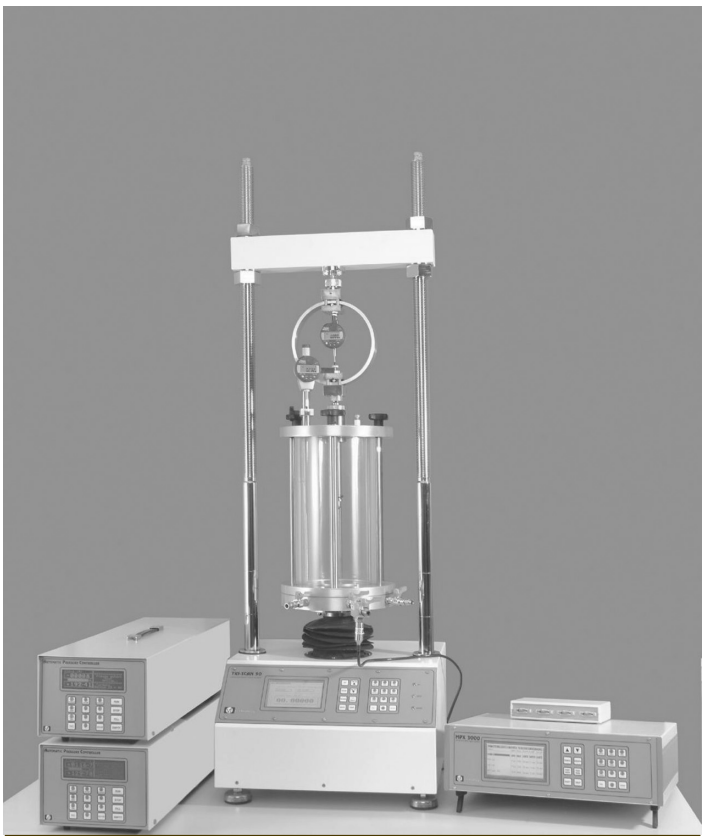


Figure 49.2 Triaxial test equipment
Courtesy of VJ Tech Ltd

It should be noted that testing of organic soils, especially peat, can produce rather 'unexpected' test results compared to those associated with sands and clays. This is due to the type, fabric, percentage and orientation of organic material which may be present in such samples. Undrained tests do not take into account the high compressibility of such materials and would yield low shear strengths whilst drained shearing stages will display very high strains and again possibly unrealistic shear strengths due to the complex nature of the material and the boundary conditions which exist in triaxial samples. These tests are certainly possible but care should be taken in the design and expectations of shear strength tests on such materials. More will be said about such materials as we move on.

Rock strength is dominated by its mineralogy and cementation along with the presence and orientation of discontinuities. Reference should be made to Chapter 18 *Rock behaviour* for further understanding and categorisation of the material. Commonly strength tests are carried out as uniaxial compressive strength tests (unconfined), but may also be carried out as confined or even effective stress tests using specialist high pressure/stress equipment. Reference is made to the main standards for such common test types in the appendix to this chapter. It should be noted that rock tests require very different equipment for preparation and testing than soil tests; however, there is a grey area where we might classify a soil as a weak rock and vice versa. In this instance, experience will prevail over which tests and equipment to adopt. As with all testing, these will require specialised personnel with significant skill



Figure 49.3 Triaxial load measuring equipment (Left: Load ring with manual dial gauge. Middle: Load ring with digital dial gauge. Right: Submersible electronic load cell)
Courtesy of VJ Tech Ltd

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and experience. The 'art' of these tests is in the sample preparation and the specific equipment used (along with sample orientation with regard to discontinuities and preferred fabric). Check with your nominated testing laboratory that they have the correct equipment and expertise to carry out such tests.

The cutting and facing of the sample if carried out incorrectly can reduce the strength of the test specimen by up to two thirds by the introduction of point loads and non-parallel faces. Cutting equipment should have very thin diamond blades and work by the rock core being moved (whilst rigidly supported) across the cutting blade, not vice versa as in concrete cutting equipment. All cutting marks must then be removed by facing. This involves the polishing of the sample surface until both ends are completely flat and parallel (tolerances can be found within the standards listed in the appendices at the end of this chapter). The samples must then be mounted on specially hardened platens (which are calibrated for flatness) which have a diameter either the same or no greater than 2 mm larger than the specimen diameter, one of which will have a spherical seat and be placed on the top of the specimen. The loading surfaces of the compression machine itself will be rigid, parallel and unable to rotate. One can appreciate that rock testing is a highly specialist form of material testing and there are few laboratories which can carry out this form of testing correctly.

49.5.2 Direct shear tests

Alternative strength tests such as the shearbox test are also common where phi angles and cohesion intercepts are required for design purposes. Such tests can be carried out on both undisturbed and disturbed samples (depending upon the desired engineering use for the material). The historical test is the shearbox which can yield values for phi and cohesion (see **Figure 49.4** for test mechanism).

Originally designed for testing sand, the equipment is now used commonly on non-cohesive and cohesive materials alike. The equipment comes in a range of sizes which provide testing for a range of particles from fine through to coarse. The shearbox consists of a 'hollow box' which is split horizontally and into which a sample is placed. The sample is consolidated to a desired normal stress and then sheared horizontally. The bottom half of the box is displaced whilst the top half of the box reacts against a load measuring device measuring the resistance to shear. These tests are usually carried out as a set of three tests where the first test is consolidated at half the calculated effective stress for the material, the second is at the calculated effective stress and the third is at double the calculated effective stress. The values from these tests should be used in total stress calculations only (although there is some argument). The only point at which we know the effective stress of the material (during the test) is at the end of the consolidation stage. As pore pressures are not measured throughout the following shear stage we are unable to verify the true effective stress of the material at failure and certainly along the plane of shear.

Consolidated peak strengths are obtainable from this equipment along with residual strengths.

Residual strengths in clays in particular are often in error from this equipment as it is not possible to form a perfectly flat shear plane. Residual values are reached only at very high strains compared to peak characteristics and the shearbox has limited travel. Some attempts to overcome this are in the test standards by reversing the direction of shearing until a consistent 'apparent' residual state is measured. For non-cohesive materials and silts this method may work, but due to the 'platey' nature of the clay particles, the perfect (flat and polished) shear plane can only develop by continued shearing in a singular direction (no reversals as per the standard shear box). This problem was overcome with the invention of the Ringshear apparatus (E. Bromhead). The normal loading and relative displacements induced during the test can be seen in **Figure 49.5** whilst a commercially produced ringshear apparatus is shown in **Figure 49.6**.

49.5.3 Ringshear test

Rather than linear movement of a block of soil, the ringshear rotates constantly and so the linear displacement (in a single direction) is limitless (or at least until all the soil has been 'squeezed' from the test annulus). Note that two opposing load measuring devices are used on this equipment. This is most necessary as the cell rotates and it is actually torsion which is measured. The use of two load measurements (being of matched stiffness) balance the top cap under rotation and are designed to prevent friction being measured from the central locating pin for the top cap.

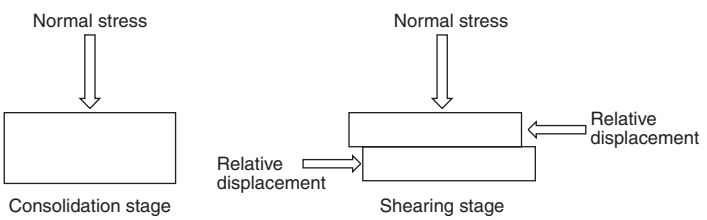


Figure 49.4 Loading and shear displacement in shearbox test

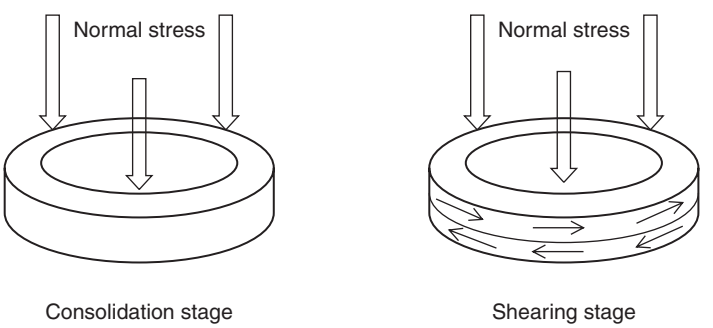


Figure 49.5 Loading and shear displacement in ringshear test

If we are interested in residual values (say for slope stability calculations) in clays, this is the test to perform. Peak strengths are not available from the 'Bromhead' or 'small' ringshear due to the fact that the sample must be remoulded as part of the test preparation. The material needs to be pressed into a tight annulus and benefits from having any structure or bonding destroyed. This allows for accelerated reorientation of the clay particles (shortening test times) and more repeatable test results through the homogeneity of the sample. This equipment is suitable for pure clays only. The presence of coarse particles may roll along the shear surface during the test and will destroy it by reorientating the clay particles. This will lead to higher (non-repeatable) residual values being obtained which may be disastrous where the true residual angle is actually lower still. For clays containing coarser material and non-cohesive samples the engineer should revert to the standard shearbox test (along with its known and well-documented limitations).



Figure 49.6 Ringshear apparatus
Courtesy of VJ Tech Ltd

49.6 Stiffness

There are many technical definitions of stiffness, but they all relate to the gradient of the line of stress plotted against strain and it is most important to consider that soils display a nonlinear stress–strain response (outside the highest levels of research). It should be remembered that the strength of the soil dictates its ultimate load-bearing capacity with large strains whilst stiffness identifies the compressibility (or strains) in the material at working loads (see **Figure 49.7**).

In general these parameters are not derived from the routine tests listed previously and require the highest quality undisturbed samples, specialist capabilities, instrumentation (**Figure 49.7**) and high levels of knowledge and experience. For stiffness of rock material reference should be made to Chapter 18 *Rock behaviour* for additional information. It is possible to fully instrument the following triaxial tests for the determinations of Young's modulus: UUP, CIU and CID. Shearing of samples may take place in either compression or

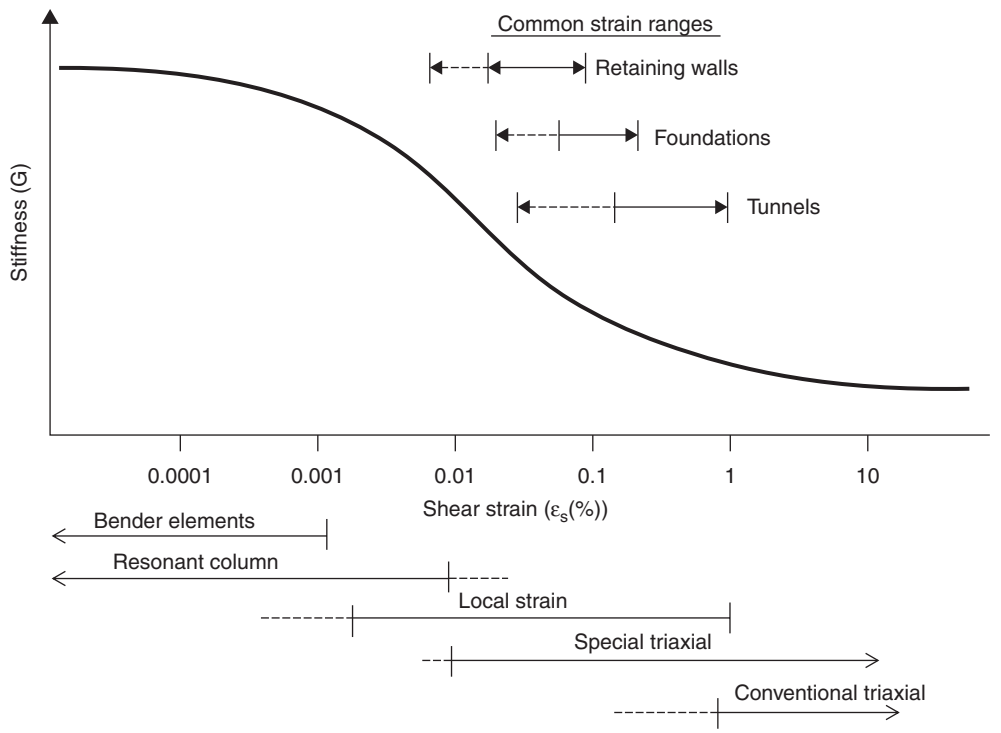


Figure 49.7 Idealised variation of stiffness with strain

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extension and so a suffix may be added to the above abbreviations in the form of a C (compression) or E (extension).

The highest level of triaxial effective stress test is known as a stress path test or anisotropically consolidated undrained (or drained) triaxial test (CAU or CAD) and again may have the C or E suffix depending upon the final shearing direction. High-resolution transducers are attached directly to the sample to measure axial strains for the determination of Young's modulus and when combined with a radial strain transducer can be used to determine Poisson's ratio, Shear modulus (G) for undrained shearing and Bulk modulus (K) for drained shearing. In addition it is possible to measure Gmax using bender elements which measure sample stiffness beyond the resolution of local small strain instrumentation (see **Figure 49.8** for typical advanced instrumentation).

These tests are designed to take the specimen through its recent stress history in order to minimise any effects of sample disturbance and return the specimen to its true *in situ* mean effective stresses or to a particular stress level required for modelling.

Figure 49.9 shows an advanced triaxial test with local axial and radial strain instrumentation, base/mid-plane pore water pressure measurement along with measurement of Gmax using bender elements in all three possible directions. This specialist

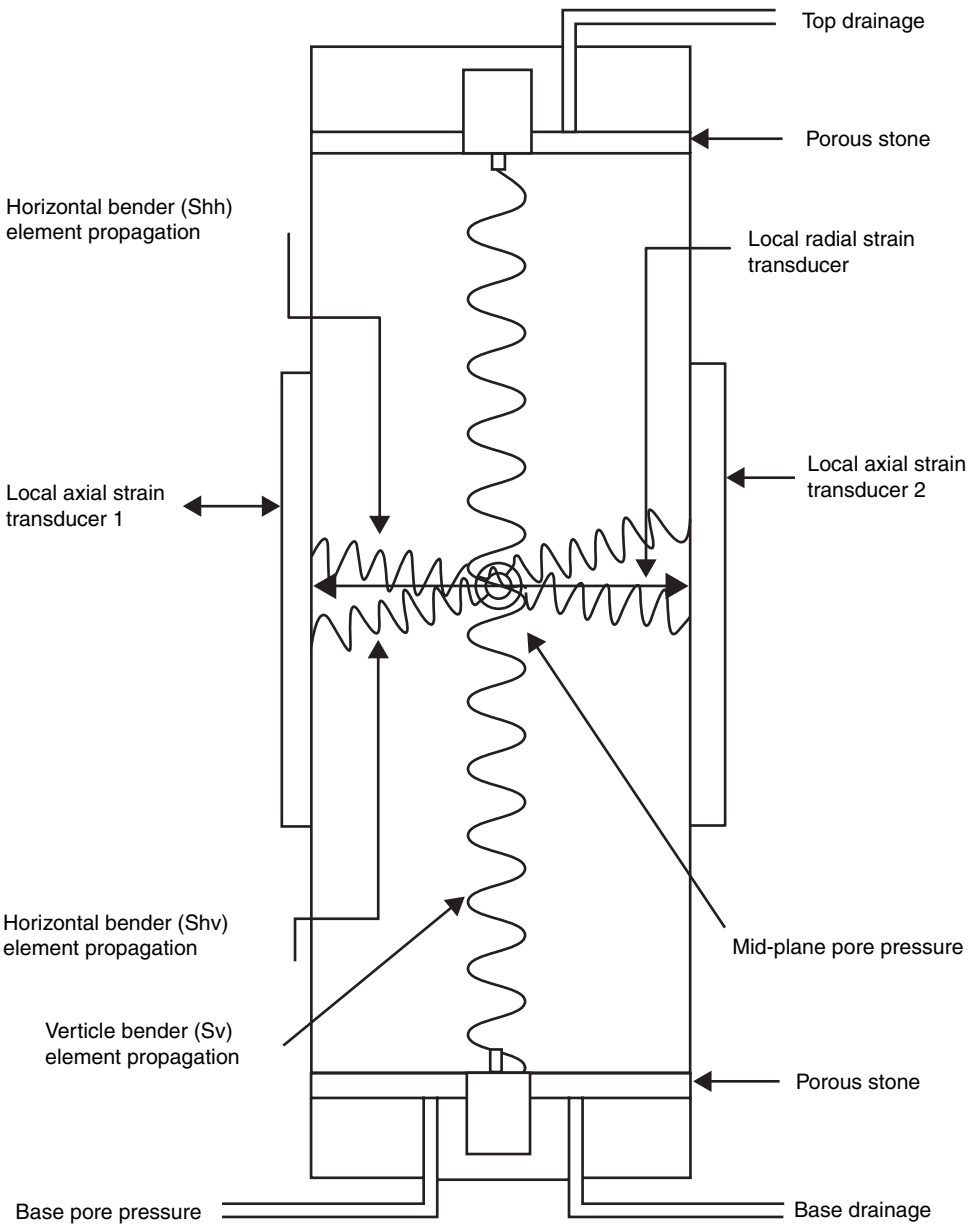


Figure 49.8 Typical advanced triaxial instrumentation for CAUC, CAUE, CADC and CADE test types



Figure 49.9 Triaxial stress (CAUC) tests with advanced instrumentation
Courtesy of Russell Geotechnical Innovations

instrumentation is used not only for the measurement of very small strain stiffness, but the utilisation of both base and mid-plane pore water pressure measurement allows verification of effective stress measurement.

The use of local axial and radial strain transducers allows the measurement of small strains directly on the sample and minimises the boundary effects (bedding and localised stress distributions) caused by the soil being in contact with the (much stiffer) end platens. Bender elements measure stiffness at even lower strains than local strain instrumentation and provide a completely non-destructive measurement of stiffness. They can be mounted in three orientations on a triaxial specimen and provide a very good indication of general sample condition and/or possible anisotropy.

During normal triaxial testing the pore water pressure is traditionally measured only at the base of the sample throughout the test. As the sample is in contact with far more rigid (usually metal) base pedestal and top cap, the areas at the specimen ends suffer boundary effects caused by the contact with the far stiffer material. For the calculation of shearing rates (time to failure) as per BS 1377:1990 we calculate that pore pressure dissipation is 95% only at the time of failure of the sample. The use of mid-plane pore pressure allows a second reference for pore pressure measurement and not only is unaffected by the metal pedestal (as it is in the central region of the sample), but can be used to display full pore water pressure equalisation throughout the sample (and not just at one end). This is ideal for drained shearing stages where the base and mid-plane pressures should remain the same if we are shearing at the correct rate. If the mid-plane pressure begins to deviate (increase) from the base measurement then excess pore pressures are being generated due to the sample being sheared too fast. For undrained shearing the pore pressures should react in unison and in the same direction (whether in compression or extension) again providing evidence that the correct shearing rates have been used. Interestingly, deviation between the base and mid-plane

may occur later during sample rupture as pore pressures may be generated/dissipated in different ways depending upon the inclination and orientation of any shear planes which may form in the specimen (and continued shear along rupture surfaces).

For rocks we can measure stiffness by carrying out a more advanced version of the uniaxial compressive strength test which also has additional high-resolution instrumentation which measures axial and radial strains. Because rock specimens fail at strains (usually an order of magnitude) lower than those of soils, specialist strain transducers (unsuitable for soil testing due to their very small range) are bonded axially and radially to the central third of the rock specimen. These are logged throughout the loading test and used to measure Young's modulus and Poisson's ratio.

Again, high technical competence is required for these tests. An ideal example is that the bonding agent used to bond the strain transducers to the sample must be able to not only prevent the gauges from creeping on the sample by bonding the gauge fully and remaining so throughout the test, but be of a lower stiffness than the sample itself so that they measure the natural strains evolving in the sample under load rather than the artificial strains caused by a stiffer bonding agent which may have locally filled the voids within the sample (causing a localised 'stiffer' response).

49.7 Compressibility

This is a term often used in soil mechanics and largely describes the relationship between stress and strain. It is the stiffness of the ground which determines the strains and displacements with changing stress and so by combining the stress level and stiffness of the material its compressibility can be determined. Laboratory tests associated with these parameters are:

- *Oedometer consolidation test*. Useful for Cv and Mv. The coefficient of consolidation (Cv) has the units M^2/year which is the 'scaled up' time (for consolidation) from the laboratory test to the full-scale field material being modelled. The coefficient of volume compressibility (Mv) has the units M^2/kN and is the slope of the porosity against the applied effective stress curve resulting from the test. Since porosity and void ratio are related quantities it also follows that Mv can be calculated from the void ratio against effective stress curve. It should always be remembered though that the value of Mv is dependent on the stress level applied.
- *Hydraulic cell consolidation test*. This is the full effective stress version of the above and is able to impose various loading conditions and drainage paths on the specimen. In addition the hydraulic cell can be used to measure the permeability of the sample at each consolidation stress along the drainage path used for the test.

Typical test equipment for these two types of test can be seen in **Figure 49.10** and a simplified line drawing of the loading and boundary conditions is given in **Figure 49.11**.

Reference should be made to **Figure 49.12** for an idealised consolidation curve. The first stage (primary consolidation) is the result of reorientation and re-packing of the soil particles with the expulsion of water from the voids. Secondary consolidation is the actual compression of the soil particles themselves with a further expulsion of water (mainly from the particles as they themselves compress). It is most important to identify the presence of organic materials, especially peats, and understand their compressibility characteristics as settlements may be orders of magnitude higher than those associated with non-organic soils. Both the oedometer and hydraulic cell can also be used for secondary consolidation or 'creep' monitoring. Secondary consolidation is the continued compression of a soil after primary compression is complete and is caused by viscous behaviour of the soil grain–water system or the physical compression of organic matter. For quartz sands we expect





Figure 49.10 Oedometer (left) and hydraulic cell (right)
Courtesy of VJ Tech Ltd

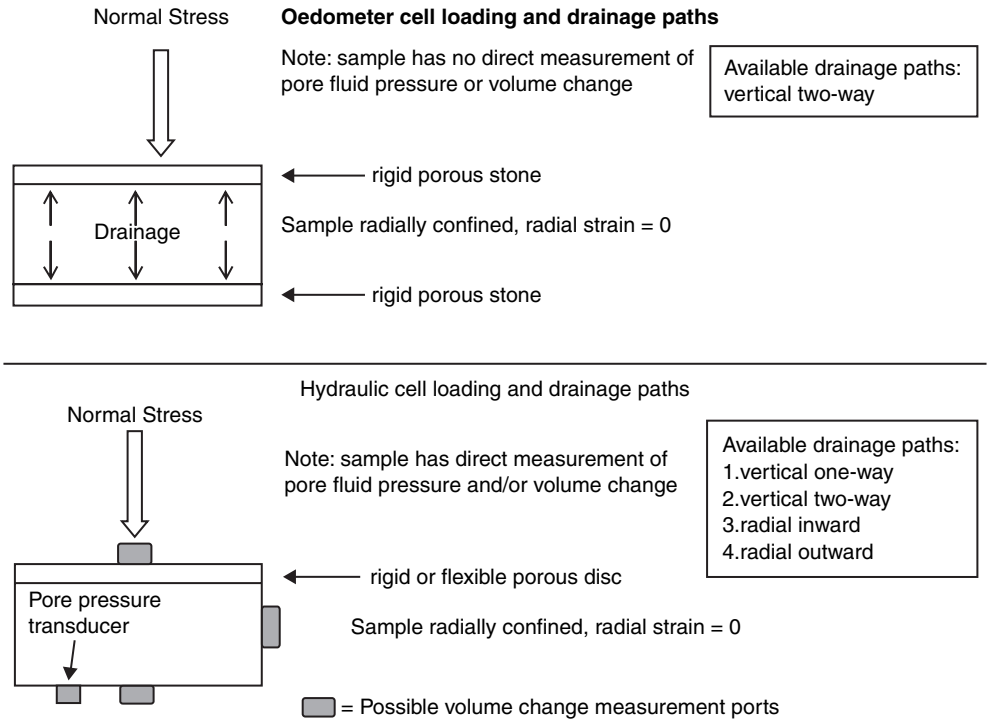


Figure 49.11 Oedometer (top) and hydraulic cell (bottom) loading and drainage paths

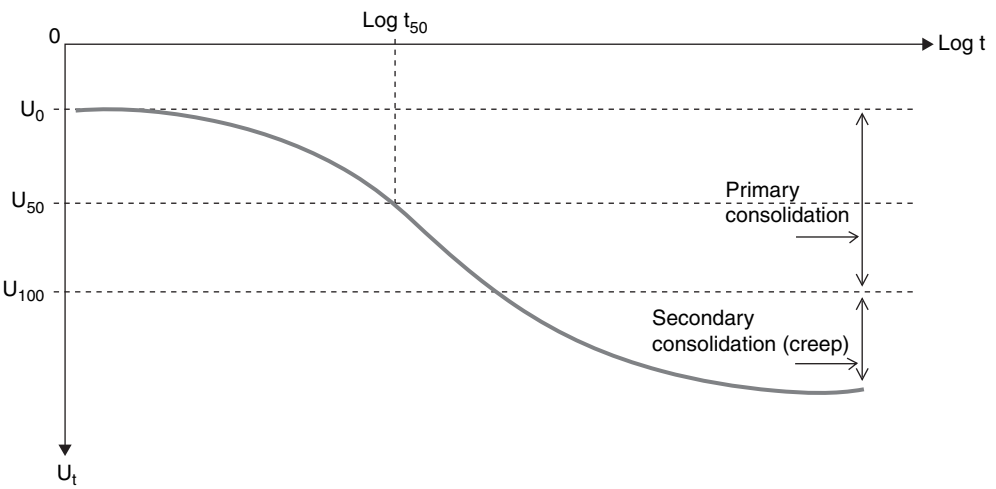


Figure 49.12 Log time consolidation curve

that creep (secondary compression) would be negligible as the sand grains are generally non-compressible. Other sand types (such as calcareous or carbonate sands) may show different behaviour or 'sudden collapse' if the normal stress exceeds the ultimate strength of the grains (and their asparites), leading to failure of the grains themselves. For peats and organic soils the soil 'solids' are themselves highly compressible and so a two-phase consolidation process is often seen with organic materials. Secondary consolidation also occurs in clays and can play important roles especially when dealing with very soft clays. When scheduling tests with the requirement for secondary consolidation parameters always bear in mind that these tests may take days, sometimes weeks per loading stage and have not only time but cost implications to match over the 'standard' tests where the interest is in primary consolidation only. It should also be noted that due to the fabric/structure of peaty materials they may show highly anisotropic behaviour especially with regard to drainage directions due to the orientation of the long axis of the vegetative material causing preferred drainage paths. In instances where these preferred paths have been identified it may well be preferable to schedule the use of the hydraulic cell and schedule a suitable drainage path/direction.

For rock materials we often assess compressibility from the intact strength of the rock material along with its discontinuity spacing and aperture (rock quality designation – RQD). It is possible to test rock samples for compressibility in high stress equipment; however, remember that you are only testing the intact material. *In situ* the bulk strength and compressibility of rock materials are usually dominated by their discontinuity spacing/orientation, aperture spacing/orientation and aperture contact areas. Other means (preferably field-based) should be used to assess these engineering characteristics.

It can be seen from the parameters gained from such tests that the soil structure and fabric (which controls drainage paths) are the controlling factors (material properties) that are being measured. Undisturbed samples of known orientation are the basic prerequisite for such tests. Re-moulded samples may be used if the construction requires design parameters to be used for such 'engineered' materials. Attention to sample quality and test preparation is paramount, especially as such small samples are tested and then the results scaled-up to the field model. Small errors magnify!

49.8 Permeability

For design purposes or interest in seepage problems we need to think about drained and undrained conditions or short- and long-term behaviour of the ground. This is dominated by the particle size, orientation and packing of the soil grains and whether they are cemented (in the case of hard pans or calcareous zones), cohesive or non-cohesive. Careful thought should be given to the sampling of such materials as the size, orientation and packing of the particles can lead to strong anisotropy *in situ* leading to high variations in permeability with flow direction. Sampling and the correct orientation of samples selected for laboratory testing is of paramount importance if true ground conditions are to be modelled representatively. Laboratory tests associated with the determination of permeability are:

- *Constant head permeameter*. For non-cohesive materials. Sample is radially confined, therefore lateral strain is zero.
- *Constant head permeability determination in a triaxial cell*. Normally for cohesive materials and where desired mean effective stresses are required. The sample is isotropically consolidated and a pressure differential applied (driving head) to the separate top and base drainage lines causing flow to occur.
- *Permeability determination in a hydraulic cell*. This equipment, mentioned previously, can be used to measure permeability along different vertical or horizontal drainage paths and can be carried out as part of a test which also gives consolidation parameters. For the possible flow directions available see **Figure 49.10**.

For rock samples, intact specimens can be tested (usually constant head), but it should be remembered that the permeability of the *in situ* material may be dominated by the presence of discontinuities and their aperture/orientation, infill, spacing and persistence.

49.9 Non-standard and dynamic tests

By mentioning the broad range of basic (and some more advanced) parameters which can be gained from the test types outlined previously, it would also be prudent to write a little about the more 'advanced' types of test which are also available in a few specialist laboratories. These tests are certainly not routine and require the use of highly specialist equipment and highly experienced staff.

Due to modern construction requirements and the need for design with dynamic loadings (such as wind turbine monopiles, etc.), geotechnical engineers are increasingly asked for the dynamic parameters more associated with those for foundation design in earthquake regions. Other tests are not necessarily 'dynamic' but are equally more towards the research end of testing. As with the anisotropic triaxial tests and their advanced instrumentation, these test types are often more appropriate to advanced numerical analysis designs and studies. They are certainly not routine and often come with a price tag to match; however, you do get what you pay for (as long as open and clear communication prevails throughout).

The following list is not exhaustive but is intended to highlight some of the more common research-level tests available and the apparatus associated with them.

49.9.1 Cyclic triaxial test

As it is named, this is a 'hybrid' triaxial testing frame which is built to 'cycle' the soil/weak rock sample by either stress or strain control around a mean level at rates commonly around 0.3 Hz with data capture of the transducers at many times a second. Cyclic strength depends upon many factors, including density, confining pressure, applied cyclic shear stress, stress history, grain structure, age of soil deposit, specimen preparation procedure, and the frequency, uniformity and shape of the cyclic wave-form (ASTM D5311). In addition it should be noted that non-uniform stress conditions are imposed by the specimen end platens which may cause a redistribution of void ratio within the specimen during the test. These tests are often carried out on non-cohesive soils and, since such materials are unable to withstand tension, the maximum cyclic shear stress that can be applied to the specimen is equal to one half of the initial total axial pressure (ASTM D5311). Care should obviously be taken in the design of such tests, and thought given to the fact that uneven pore pressure distributions throughout the sample may result depending upon the permeability of the soil and the rate at which it is cycled. Young's modulus and soil damping properties can also be evaluated for specialist design using this test type (ASTM D3999).

49.9.2 Simple shear

This can be carried out either as a monotonic test or as a dynamic cyclic test. The shear strength is measured under constant volume conditions that are equivalent to undrained conditions for a saturated specimen; hence, the test is applicable to field conditions where soils have fully consolidated under

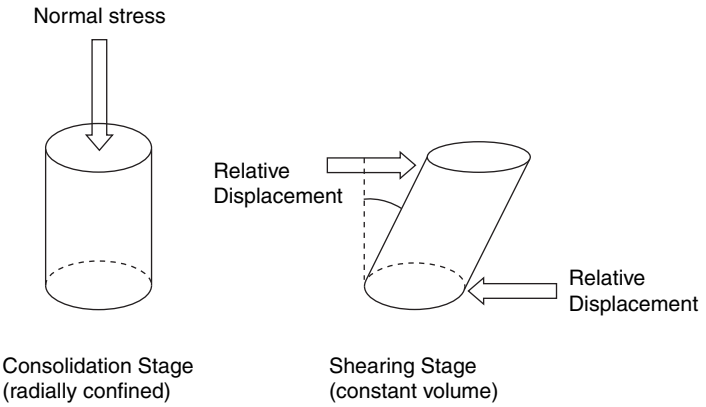


Figure 49.13 Simple shear mechanism

one set of stresses, and are then subjected to changes in stress without time for further drainage to occur (see **Figure 49.13**). The constant volume (undrained strength) is a function of stress conditions (plane strain) and the principal stresses continuously rotate due to the application of shear stress. This simple shear stress condition occurs in many field situations including zones below a long embankment and around axially loaded piles (ASTM D6528).

49.9.3 Resonant column

Figure 49.14 shows a resonant column with the cell top removed. The top part of the equipment is used to induce torsional movement to the top of the sample by an electromagnetic drive system which can run at a range of frequencies to determine the resonant frequency of a sample.

Such tests and test equipment are used to evaluate the shear moduli and damping characteristics of soil at very small strain amplitudes. Although there are two distinctly different equipment types for these tests, both apply torsion/rotation to the top of the sample in order to find the resonant frequency of the material at a controlled stress. These test methods are non-destructive if the strain amplitudes are less than 10^{-4} radians and many measurements may be made on the same sample and with various states of ambient stress (ASTM D4015).

49.9.4 Hollow cylinder test

Certainly the rarest of commercial tests, this equipment allows a rotational displacement to be imposed on a 'hollow' cylindrical specimen where independent control can be maintained for all three principal stresses (unlike triaxial tests which can only independently control two of the three principle stresses (where $\sigma_2 = \sigma_3$)). For this reason studies can be made of the intermediate principal stress (σ_2), sample anisotropy and the effects of principal stress rotation. These are 'research level' tests and the parameters derived are usually only used for the most advanced numerical analysis. Tests are available for both soil and rock and the hollow cylinder is most useful in the definition and determination of anisotropic material properties.

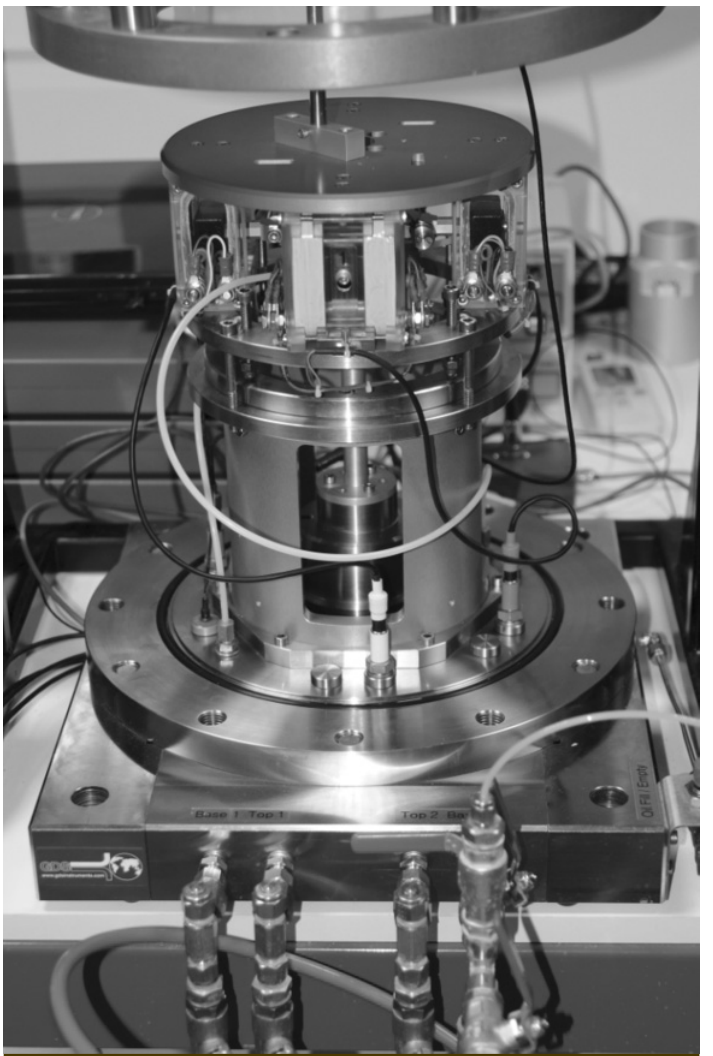


Figure 49.14 Resonant column apparatus
Courtesy of Russell Geotechnical Innovations

Simpler but equally non-routine tests can be used to model specific behaviour of soils and materials. The shearbox can be used to model the frictional behaviour which occurs at the interface between soil and a geotextile/steel/concrete surface. A ringshear can be used to model the soil behaviour at an interface between soil and a steel pile (Jardine *et al.*, 2005). In addition tests can be carried out to determine the dispersion or erodibility of soils and rocks when abraded or exposed to persistent high moisture contents or flowing water. The tests listed in this section are certainly non-exhaustive and complete volumes could be written about laboratory testing and parameter determination. A good general series of volumes to read are those by Head (1986), *Manual of Soil Laboratory Testing*.

49.10 Test certificates and results

Test results are issued as certificates which identify the parameter requirements, data and graphs in accordance with the test methods and standards used. There should be no problem

requesting further detail from the laboratories involved, to understand or verify any particular test conditions or methods used. It is here that open communication is important, and always remember that you are the paying customer. This line of communication should also allow the laboratory to freely communicate any observations or potential sample problems which may lead to unexpected results. It is this knowledge sharing that will improve the resultant parameter and overall design quality.

In order to supplement our ground model and design requirements we can see that careful planning at all stages of the ground investigation process is required and hence, why the natural process order of this chapter has been reversed somewhat. This process of investigation from sampling techniques through to sample storage, transport and laboratory testing requires careful planning and supervision. Any loss in integrity of the sample material properties at any stage in this sequence could have catastrophic effects on the parameters supplied from the testing house. These parameters are largely interlinked, especially moisture content and the sample physical integrity which dominate the effective stress characteristics. Geotechnical engineering is one of the only sciences which starts with a natural material being removed from a stable environment (the ground) and taken through a series of potentially damaging processes, through water addition/loss, exposure to atmosphere, physical handling (including jarring and vibration), temperature cycles and, finally, to a 'stable' environment where the material is tested for specific parameters which are to be representative of the ground from whence it came. How do we do it? This is the difference between a good site investigation (and all of the processes involved therein) and a not so good site investigation.

Fundamental in this way of thinking is the preservation of the sample moisture content. The determination of moisture content in the laboratory is only accurate if the sample retains the same moisture content that it had at that time in the ground. The determination of bulk density, strength, effective stress and stiffness, to name but a few, are all controlled by moisture content. If this is allowed to change between sampling (or altered by the sampling method) and the final test then the design will be based on erroneous values.

To realise this from the planning stage will help to identify possible problems and build them into the specification, and drive the ground investigation in the correct way from the start. If all stakeholders are involved in the initial stages and the project expectations and responsibilities are clear then the majority of these 'integrity loss' components can be minimised. The sample tested in the laboratory is only as good as the sample received there and this also assumes that the correct test was scheduled and that the laboratory was proficient in that particular method.

49.11 Sampling methods

This part of the project will have been conceived at an early stage as the ground investigation contractor will need the relevant equipment and correct experience for the project awarded. They will need to extract the samples from the ground and to deliver them to the laboratory for testing in the best possible condition. Here a chain of custody is formed and the 'smooth' operation of this will depend upon the sharing of information and open communication which will allow some flexibility to be built in for 'on-the-job' improvement. In an ideal world the laboratory will test a soil or rock which is in the same condition (and is therefore entirely representative) of the material *in situ*. This chain begins with the excavation of a trial pit or the drilling of a hole. A sample is then taken in various ways (to be explained later in more detail) and sealed in order to maintain its integrity. This sample is then either stored or immediately transported to a laboratory for testing, where again it may be stored (in a queue) whilst awaiting testing. Sampling for chemical and contamination testing is dealt with in Chapter 48 *Geo-environmental testing* of this volume and should be referenced as necessary. Here we are dealing specifically with sampling of the ground for the physical testing required for design parameters. There are many sampling methods available globally, but may be categorised simply as bulk samples, block samples, tube samples and rotary-cored samples. Each category has associated levels of disturbance and some indication of these is given along with the basic requirements for sample preservation. The following list is certainly not exhaustive but should indicate the main principles for 'good practice'. The reader must also adhere to the provisions of Eurocode 7 or other prevalent standards depending upon the geographical location of the investigation or agreed project requirements. Eurocode 7 is very prescriptive in terms of the sample types which may be used for various types of test as are many of the other standards generally used worldwide.

49.12 Bulk samples

This constitutes probably the simplest but the most disturbed sample type. Samples are often hand- or machine-excavated from a trial pit or spoil heap and placed in bags for logging purposes or index tests only. These samples should be of sufficient size to be representative of the horizon of interest, uncontaminated by material from other horizons, and of sufficient quantity for the testing required.

For bulk samples the material should be sealed in a bag with as much air evacuated as possible in order to prevent the sample from 'sweating' or the production of mould during storage. No samples, even if fully sealed, should be left in sunshine as this will not only cause the sample to 'sweat', but will also cause non-uniform heating leading to expansion/contraction of any fissures or textural fabric or aid the growth of mould, fungus or microbial organisms which again may alter the material properties. Any sample should be kept at a constant temperature and away from any localised heat sources. In the UK such temperatures should be no more than 20°C and no less than 5°C (under which the sample may begin to freeze). This is a range of temperature in which the sample may be kept, but it should not be cycled more than 3°C over a mean temperature if at all possible.

49.13 Block samples

Block samples are undisturbed hand-dug blocks which are usually some 0.5 metres square or diameter and at least 0.3 metres in depth. They are then sealed with an impermeable barrier followed by a rigid supporting container constructed around them. This is followed by careful paring from the substrata and removal for total sealing and support for storage and transport. Bearing in mind the preservation of the sample which is required, site- and environment-specific methods would be required for taking block samples and preserving the *in situ* characteristics of the medium sampled. Such samples are limited by access and depth of interest as space is required for personnel to cut the block safely and extract it from the horizon of study.

49.14 Tube samples

Tube samples are taken in a variety of ways depending upon equipment available and access.

The most common tube sampler used in the UK is the U100 and is often used in conjunction with light percussion drilling techniques (tripod-type rigs) where the tubes are driven into the ground. You will easily visualise the disturbance which the material may undergo by having a tube 'hammered' into it. This is part of the reason why U100 tube samples are not suitable for undisturbed testing parameters (but fine for index tests only and logging purposes). Tubes may also be 'pushed' into the substrate using piston-type (fixed piston) equipment which 'jacks' the tube into the bottom of the borehole. This method is superior and when used in conjunction with thin-wall sample tubes provides acceptable quality undisturbed samples. For thin-wall push samples, make sure that the end of the tube has been sharpened, has no burrs and that it is straight and true (at least before it is used). After sampling, also check that the tube has remained straight and true. If the tube has buckled or deformed during the sampling process then the soil within will have deformed (strained) as well, rendering it unsuitable for high quality parameter determination. For tube sampling the basic stages are outlined in **Figure 49.15** (Hight, 2000). From this we can determine the stresses to which the sample is exposed. Disturbance may be caused at any stage in the process but probably the most destructive is when the sample tube actually penetrates the bottom of the borehole (Hight, 2000).

For sampling to take place, the tube (and cutting shoe for U100s) is required to be pushed into the ground. Due to this additional material (sampler) being introduced into the natural soil the bulk density of the material into which they are inserted changes whilst the sampler is intruded into the bottom of the borehole. With this in mind we should in theory use a sampler which has as thin a wall and as sharp a cutting tip as possible in order to prevent local ground densification and undue strains or fabric disturbance. The sampler should be pushed in smoothly and without side movement or jarring (not percussive techniques). From this it can be understood that the area ratio (%) of a sampler plays an immense role in the disturbance of the sample taken. The area ratio is calculated as the volume of soil displaced by the sampler as a proportion of the sample volume (calculated by measuring the internal and external diameter of the cutting shoe or edge). In theory, the lower the value, the less disturbed the material within. Clayton and Siddique (1999) examined the different effects of tube geometries and used examples from the four main sample tube geometries used in the UK at that time plus a fifth experimental design. The geometries of these tubes are shown in **Figure 49.16**.

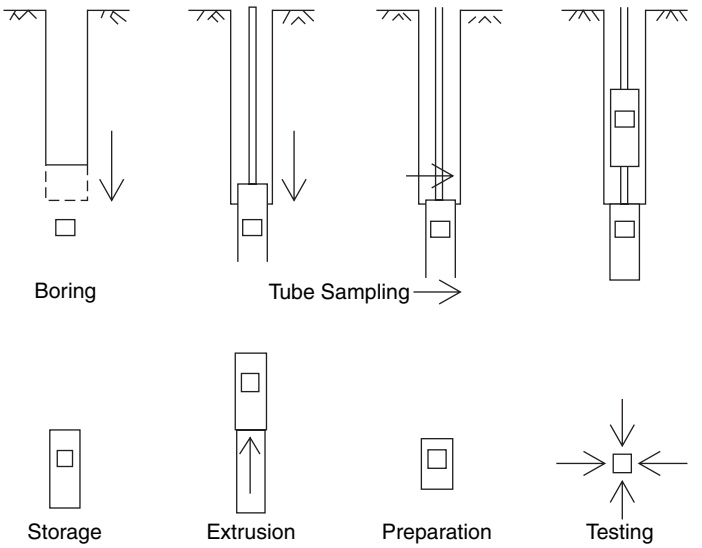


Figure 49.15 U100 Sampling stages
Reproduced from Hight (2000)

Sampler 1 is the geometry of the cutting shoe used on standard 'metal' U100 sample tubes and has an area ratio of 27%. Inside clearance is obtained by a step out from the cutting shoe where it screws onto the sample tube above it.

Sampler 2 is an upgraded version of sampler 1. It has a very similar area ratio, but with an inner step which is replaced with a slight taper and the cutting edge tapers that have been reduced (sharper).

Sampler 3 is the version of the UK cutting tube used for U100 samplers with plastic liners. The area ratio is considerable at 48% and again a small step inside produces the inside clearance between the shoe and the liner.

Sampler 4 is the 'thin-wall' push sampler which has been widely used in many circumstances to produce quite high quality samples. The original tube was described by Harrison (1991) and consists of a tube (normally stainless steel) with a 15° taper on the cutting edge. This tube is 'pushed' into the bottom of the borehole rather than 'hammered' like the previous samplers.

Sampler 5 is an experimental sampler (Hight, 2000) which is similar to sampler 4 but is sharper (5° taper) and has a 0.1 mm flat at the cutting tip.

This is not intended to be an academic publication but it is more than prudent to give some background to the sampler types with regard to sampling and disturbance and link them to the parameters required from the samples. From research

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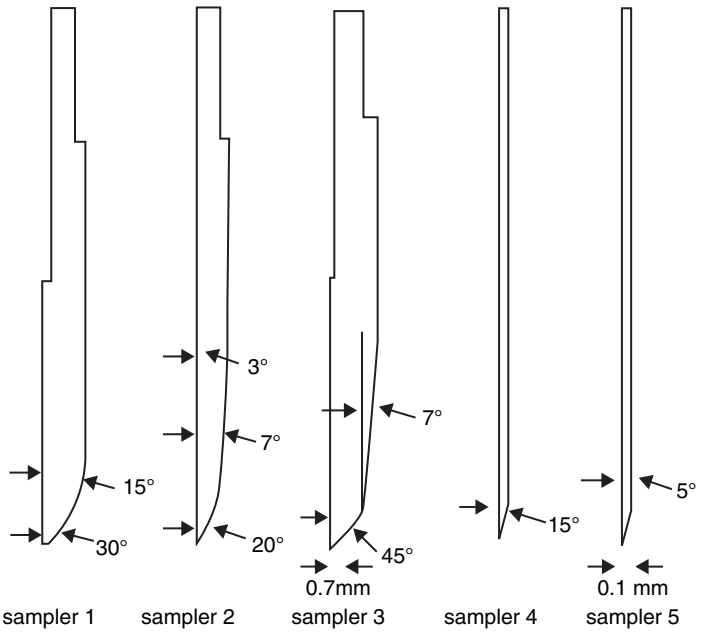


Figure 49.16 Tube sampler geometries
Reproduced from Clayton and Siddique (1999)

testing and some very high grade commercial tests it is known that on natural stiff clays such as London Clay failure occurs at axial strains in the region of 0.75–2.0%. Clayton and Siddique (1999) studied the sampler geometries and made strain predictions along the centre line of the sample as it would be taken using the different geometries (**Figure 49.17**).

From this we can see that samplers 1 and 3 are likely to fail the natural stiff clays during sampling due to the strains imposed on the specimen with sampler 3 being by far the worst offender. The thin-wall push samplers outperform all geometries modelled and sampler 4 should be used as a minimum if stiffness parameters are required. For normally consolidated and lightly overconsolidated clays Hight (2000) notes that the strains at the periphery of the sample, during sampling, causes a zone of re-moulded soil which combines with shear-induced pore water pressures which increase across the sample and are at their highest at the periphery. This leads to an overall reduction in mean effective stress caused by an increased water content in the centre of the sample as the sample re-equilibrates (due to the highly disturbed periphery). The same outcomes are true for overconsolidated clays along with damaged material structure and fabric.

Due to the high permeability of sands, tube sampling will be 'drained' and both volumetric and shear strains will occur (Hight, 2000). Levels of sample disturbance will vary with the *in situ* density of the material and destructuring/density changes will result due to the yield strain at the particle contacts being very low.

For all tube sampling methods it is imperative that the base of the borehole is cleared of debris before the sample is

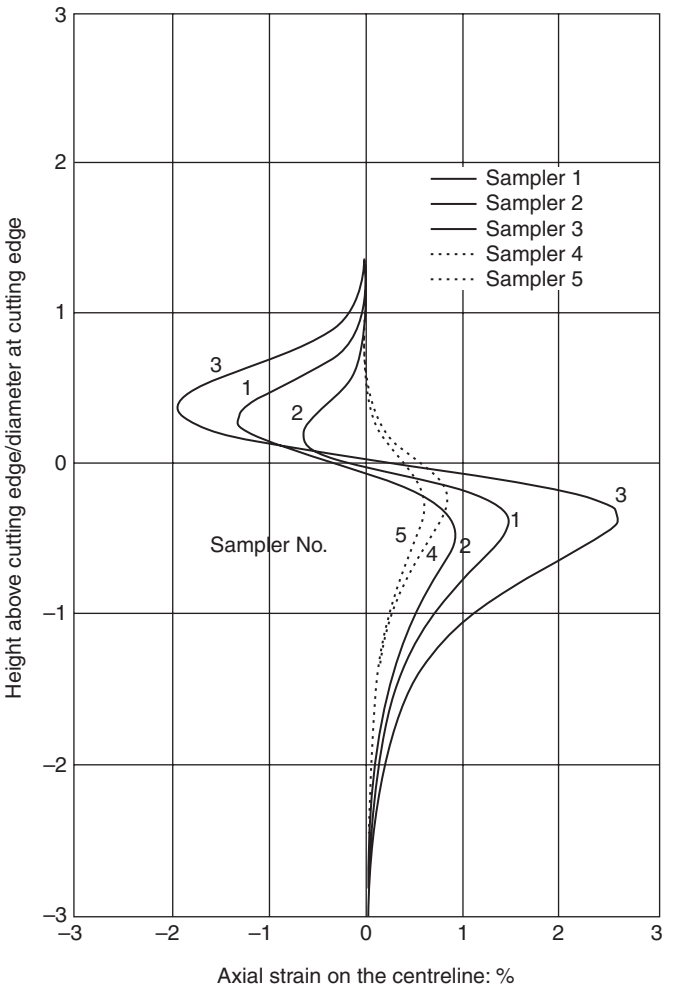


Figure 49.17 Sampler predicted axial strains
Reproduced from Clayton and Siddique (1999)

taken. If this does not occur, the tube will be filled with varying amounts of highly disturbed material which may have very high moisture contents leading to incorrect borehole logs and unrepresentative soil parameters.

On extraction from the borehole the sample tubes should be cleaned and any excess water immediately removed. The intact samples recovered should have their ends painted with low melting point wax and an identification label (indelible ink) placed in the top of the tube and marked as the 'top'. In cases where full recovery has not occurred, an inert non-compressible material should be used to fill the void left in the tube before the sealing caps are put in place. This is carried out to prevent the intact sample sliding around in the tube. The tubes should be stored and transported upright with the top of the sample uppermost. This is most important where soft samples have been taken which may try and 'flow' down the tube if knocked or vibrated during handling or transportation. Even the more competent samples may suffer from fissures opening up if stored on their side when moved

or transported. The end sealing caps should be clearly marked with all the samples' details and way up, again with indelible ink. The tubes should be preferably marked both ends just in case the identification from one end is unreadable or removed (markings on the side of the tube are often rubbed off during handling and transport). A waterproof identification label should also be placed within the top of each tube as a failsafe.

49.15 Rotary core samples

This sampling method is probably the most common high quality method used extensively in the UK and in many circumstances is superior to thin-wall push samples. The advantage here is that the material is removed from the cutting face of the drill bit and so the sampler does not densify the soil as it is inserted but simply 'reams' a stick of intact material from the ground. The drawback is that some lubrication in the form of drill flush is required which means the possible addition of moisture to the sample; however, with sufficient expertise and thought this can largely be overcome.

Rotary drill rigs come in various sizes depending upon access and depth of hole required. A lorry-mounted rotary rig can be seen in **Figure 49.18** whilst a smaller tracked rotary rig (**Figure 49.19**) can be used for slopes or where there is limited access.

These rigs can use a variety of drill bits and drill flush agents depending upon the types of material encountered. They are far too numerous to list here, but if in doubt, drilling trials can be written into the ground investigation plan in order to verify sample quality and recovery before the main exploration phase begins. Typically triple barrel core tubes are used in which the innermost barrel is a semi-rigid plastic liner which enables the sample to be removed without undue stress. Using such equipment correctly and safely requires specialist contractors with knowledge and experience and care should be taken when choosing such contractors. With this method it is possible to sample not only rock, but firm through stiff to hard clays and compact sands. With the correct drill equipment, cutting heads and flush recovery should be very good.

Following is an overview of the field sampling procedure in order to identify the main processes. Variations are allowable depending upon particular requirements and conditions but these must be agreed by or scheduled by the client.

1. Gain a suitable sample from the ground.
2. Clean the sample in such a way that will preserve its strength and fabric retaining the properties of the same material *in situ*.
3. Sub-sample and/or trim the specimen to a size (normally H:D = 3:1) that is suitable for a laboratory test (triaxial or other).
4. Preserve the specimen so that it may be stored until testing is required.
5. Protect the sample from the effects of time and any changes to environmental conditions which may alter the properties or integrity of the sample such as storage temperatures, cycling ambient temperatures, transport shocks and vibrations, ultraviolet light.

The full specification for the preparation and storage of samples from drilling a rotary core ready to be tested in the laboratory may take the form of (but not be limited to the following):

- Remove the core from its liner immediately on extraction from the ground in order to remove drill fluids used and protect the natural moisture content and physical integrity of the core. The core liner should be split diametrically in two halves by using some form of counter-rotating opposing blades set so that they cut the plastic tube (liner) without cutting into or marking the sample within. The use of sharp knives for this operation should not only be prohibited for health and safety reasons but also due to the force required to penetrate and then pull along the tube. This may not only damage the sample within but there is a high risk of operator/spectator injury when slips occur.
- Clean off by wiping with an absorbent cloth any drill fluid or flush/water from the outside of the core.



Figure 49.18 Lorry-mounted rotary drill rig
Courtesy of Soil Engineering (formerly Norwest Holst Soil Engineering Ltd)



Figure 49.19 Track-mounted rotary drill rig
Courtesy of Soil Engineering (formerly Norwest Holst Soil Engineering Ltd)

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- Log the core and sub-sample by carefully cutting for laboratory testing. Samples intended for triaxial testing should have an approximate length to diameter ratio of 3:1 (which will allow for later trimming in the laboratory).
- The sub-samples should then have their extremities prepared for sealing. Rotary-cored samples with very high permeabilities – compact sands for example should have any drill flush removed from their extremity and immediately be sealed. Clays should have the outer 5 mm of the sub-sample carefully trimmed off in a soil lathe in order to expose 'fresh' material which has not been contaminated with drill fluids. This process should be carried out very quickly for both material types but for slightly different reasons. For sandy materials it is important to prevent moisture loss (due to the relatively high void ratio and permeability) and for clay-type samples to prevent moisture ingress softening the sample and reducing its effective stress. Care should be taken that this process is carried out swiftly and in an environment suitable to reduce evaporation and localised heating of the sample.
- The sub-sample should then be sealed in such a way to trap minimal air, not only to maintain its natural moisture content but also its physical integrity for storage and transport. Often samples are wrapped in a layer of aluminium foil for the first layer, which is fine for most sample types as it can be lightly moulded in order to expel air and maintain contact with the sample surface as an impermeable barrier. Some thought should be taken though as salt water and some alkali pore water along with alkali minerals (gypsum) can react with the aluminium causing loss of sealing and reaction with the sample itself. In these instances non-permeable plastic film should be used as the first layer. Traditionally (and because it is commonly available from the local shop) many core sealing operations are carried out using plastic food wrap. The only problem with this is that this stretchy plastic 'food wrap' is by its nature osmotic or semi-permeable, and therefore not ideal. It has been found that the plastic film used to wrap pallets is not only stretchy, strong and seals against itself but is non-osmotic and impermeable. The sample should then have a layer of such material wrapped round two to three times to completely encase it. A label should be enclosed providing the sample identity and orientation. The complete sample should then be coated with low melting-point wax (which is usually a mixture of 50% petrolatum and 50% paraffin wax) which is quite soft and 'sticky'. *Not pure candle wax* (as its melting point is too high). This wax should be heated only to the temperature required for melting and not to the point of boiling. This is not only a health and safety issue but the idea is that as soon as the wax contacts the cooler specimen it will set whilst transferring minimal heat to the sample. The sample should not be dipped in the hot wax pot but should be painted with a brush dipped in the warm wax. These alternating coatings/layers can be repeated in order to protect the sample further, but the sample identity should always be clearly visible. Strong tape such as carpet tape should then be wrapped around the ends of the sample to protect the wax and layered coatings from damage. Finally the sample may be placed in split core liner which is then taped in order to give support to the sample. This core liner (or suitably rigid material) should be cut to the length of the sample so that the sample is retained rigidly and is unable to move about within its support. Caps should be placed over the ends in order to complete the encapsulation. The sample should be relabelled in an indelible fashion with full identification and orientation (top/bottom).
- In essence the sample is to be preserved as near as possible to the condition as it was *in situ*, but isotropically de-stressed. This preservation must be able to withstand handling transportation and storage which in some instances may be for some appreciable time.

49.16 Transport

This is a simple process, but very often overlooked. The samples should be transported in such a way that they are not shocked, dropped or vibrated. If sample integrity is an issue then the samples should be protected accordingly. Personal delivery and transport within the chain is often the only way to maintain high integrity. External transport contractors often do not appreciate the care required when handling a piece of 'soil' and so this should be avoided. Many couriers see such parcels as just 'heavy' rather than highly fragile scientific material. The samples should be transported 'upright' and in a way that they are supported laterally (padded boxes) to prevent toppling, rattling and vibration. Even when transported personally, the samples should not be placed in footwells where they are near heating/cooling vents which may locally heat/cool the samples. This completes the chain of custody to the testing laboratory who will then test the samples for the parameters required.

49.17 The testing laboratory

There are many testing houses available throughout the world, and offer testing to many levels and standards. Care should be taken in choosing the testing laboratory as it is relatively simple for them to buy the equipment to carry out tests but this does not mean that they are proficient in that particular test. Make sure that you are clear about the parameters you require from your tests and the identity of the samples to be tested. Bear in mind that you will need sufficient soil sample of the correct quality in order for the tests to be representative. It is often a good idea to visit prospective laboratories, if possible, and again this promotes the sharing of information and keeping up-to-date with the latest testing and contractual developments. You will also be able to assess the level of expertise, the equipment and processes involved with your proposed testing. Some tests may be beyond the scope of external accreditation bodies (and the accreditors!) and in this instance the reputation and experience of the staff carrying out the tests will prevail.

Over the years the writer has seen some 'very interesting' practices carried out in a range of laboratories; some with high levels of external accreditation. These accreditations are not a guarantee of high quality but proof of a level of proficiency and management system on the day of the audit. Procedures, systems and tests are audited (often by external auditors) so that a certificate can be issued providing evidence of compliance and repeatability. However, this does not necessarily mean that the result issued will be correct. It is very possible to follow an incorrect method repeatedly and gain the incorrect results repeatedly and still be accredited for this! There are many good laboratories out there. Find them and use them.

The new Eurocodes are addressing the standardisation of modern drilling and sampling practices and their relative merits with regard to sample quality for laboratory testing. Hopefully

the demise of the U100 plastic liner sampling system has arrived as samples from such tubes are 'highly disturbed' and are therefore of little value for laboratory testing apart from index properties. Strength, compressibility and permeability determinations will be in varying degrees of error to the material *in situ*. Where the intention is to study the variation of these parameters with depth, U100 tube samples are not good and often contribute to the 'scatter' we see in plots with depth. This, along with other 'mishaps' and lapses in attention or detail in the chain from sampling to testing, all adds to the error band or 'scatter'. Much of this can be avoided very simply through care and attention to detail. Unfortunately this is sometimes lost in the pressure of 'getting the job done'. The parameters we require from these samples for our design should be representative of the material *in situ* and not to the environments which the material has been exposed to on its journey from the ground to the laboratory (and subsequent test methods).

This completes the physical sampling/testing loop of ground investigation. You should have spare material if possible for alternative or repeat tests should you find the need for additional testing. It is very expensive to re-drill or have further boreholes sampled at a later date. All data from testing should be available if required for further analysis even down to the weights for moisture contents or the cone values and moisture contents for Atterberg determinations.

It is understood that in some instances and in some areas around the world some deviations from these guidelines will be required; however, the same principles and ideals should be followed. Notes should be kept as to the methods used and the environment in which the samples were taken and sealed including dates, times and personnel involved. Everything that we do as professionals should be able to withstand scrutiny but also provide sufficient data to repeat or improve our activities in the interests of a forward-looking science. The attention to detail in the complete chain of custody will pay dividends in the quality of the subsequent tests and parameters derived.

49.18 References

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49.18.2 Useful websites

ASTM International (formally known as the American Society for Testing and Materials), contains many internationally recognised references for soil and rock testing; www.astm.org

Home of the British Geotechnical Association, contains information for updates of many relevant technical Standards and links to many other sites of interest; http://bga.city.ac.uk

British Standards Institution, references for UK and European Standards including training and accreditation; www.bsigroup.com

Engineering Group of the Geological Society (EGGS), many useful references for rock behaviour and categorisation; www.geolsoc.org.uk

International Society for Rock Mechanics, contains the European suggested methods for various rock tests (the 'Blue Book'); www.isrm.net

It is recommended this chapter is read in conjunction with

- Chapter 17 *Strength and deformation behaviour of soils*
- Chapter 18 *Rock behaviour*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Appendix A
Standard soil and rock tests

The standards named here are generally applicable to the UK and should be used in association with the present guidelines required to apply to *Eurocode 7: Geotechnical Design – Part 2: Ground Investigation and Testing* (BS EN1997-2: 2007). Eurocode 7 is applicable to European construction and may be accepted in other parts of the world. It should be noted that different standards may apply depending upon the various locations around the world where the construction is to occur. The other major standards applied worldwide are the ASTM standards and these should be applied or used where applicable.

The list here is not exhaustive and is only an indication of some of the more common soil and rock tests available. It should also be noted here that the European standards for the identification and classification of soils and rocks (BS EN ISO 154688-1 (2002), BS EN ISO 154688-2 (2004) and BS EN ISO 14689-1 (2003) implemented into UK practice in 2007 have all been incorporated into BS 5930:1999 Amendment 1 which incorporates a revised section 6 (published in 2007). Earlier versions of BS 5930:1990 do not meet the requirements of the new Eurocodes and would not comply with the recent code changes.

Soil test	Reference
CLASSIFICATION/INDEX TESTS	
Determination of moisture content (MC)	BS1377:Part 2:1990, **3**
Determination of Atterberg limits (liquid and plastic limit, usually four-point cone method)	BS1377:Part 2:1990, **4, 5**
Determination of density	BS1377:Part 2:1990, **7**
Determination of particle density	BS1377:Part 2:1990, **8**
Determination of particle size distribution (PSD)	BS1377:Part 2:1990, **9**
COMPACTION-RELATED TESTS	
Determination of dry density/moisture content relationship (compaction test)	BS1377:Part 4:1990, **3**
Determination of maximum and minimum dry densities for granular soils	BS1377:Part 4:1990, **4**
COMPRESSIBILITY TESTS	
Determination of one-dimensional consolidation properties using a hydraulic cell	BS1377:Part 5:1990, **3**
CONSOLIDATION AND PERMEABILITY EFFECTIVE STRESS TESTS	
Determination of permeability in a hydraulic cell	BS1377:Part 6:1990, **4**
Determination of isotropic consolidation in a triaxial cell	BS1377:Part 6:1990, **5**
Determination of permeability in a triaxial cell	BS1377:Part 6:1990, **6**
SHEAR STRENGTH TESTS (TOTAL STRESS)	
Determination of shear strength by direct shearbox	BS1377:Part 7:1990, **4, 5**
Determination of residual strength using the small ringshear apparatus	BS1377:Part 7:1990, **6**
Determination of undrained shear strength in a triaxial specimen WITHOUT measurement of pore pressure (QUU)	BS1377:Part 7:1990, **8**
Determination of undrained shear strength in a triaxial specimen with multi-stage loading and WITHOUT measurement of pore pressure (QUU multi)	BS1377:Part 7:1990, **9**
SHEAR STRENGTH TESTS (EFFECTIVE STRESS)	
Consolidated-undrained triaxial compression test with measurement of pore pressure (CIU)	BS1377:Part 8:1990, **7**
Consolidated-undrained triaxial compression test with measurement of pore pressure (CID)	BS1377:Part 8:1990, **8**

Rock test	Reference
Preparation of rock core specimens and determination of dimensional and shape tolerances	ASTM D4543-08 or ISRM suggested methods (2007)
Determination of water content	ASTM D2216-10 or ISRM suggested methods (2007)
Determination of porosity/density using buoyancy technique (for both regular and irregular shapes)	ISRM suggested method (2007)
Determination of slake durability index	ASTM D4644-08 or ISRM suggested methods (2007)
Determination of point load strength for diametral and axial tests	ASTM D5731-08
Determination of splitting (Brazillian) tensile strength of intact rock core specimens	ASTM D2936-08
Determination of compressive strength and elastic moduli of intact rock core specimens under varying states of stress and temperatures	ASTM D7012-10

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Chapter 50
Geotechnical reporting

Helen Scholes Geotechnical Consulting Group, London, UK
Phil Smith Geotechnical Consulting Group, London, UK

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Geotechnical reports exist in a number of different forms, each with its own distinct purpose. However, all geotechnical reports should be written with an understanding of the project that they have been prepared for and the use to which the report will be put. Geotechnical reports of all forms must be clear and concise, such that the information contained within them is accessible and unambiguous. If the project programme is such that some information to be contained within the report is required urgently, it may be appropriate to produce multiple reports, or addenda to a main report, rather than delay issuing the time-critical information. Since a report can provide useful information long after it was first commissioned it is important to ensure that all reporting is properly archived, and thought is given to how the information contained within the report will remain accessible.

50.1 Factual reporting

50.1.1 Introduction

The primary purpose of a geotechnical factual report is to accurately and precisely describe the ground 'as found': as it occurs *in situ*. While this description should be made with the knowledge that the end-user of the report is likely to make use of the report to design and construct some form of construction, a factual report is not interpretative: the report should be factually accurate and as complete as possible, and not written on the assumption that construction will be of a certain form or use certain techniques. Factual reports are mostly typically the method of reporting site or ground investigations. Such investigation works are often undertaken at an early stage in a project, and the information from the site and ground investigations may result in changes to the layout, nature or methodology of construction of the proposed works. Hence a factual report should always be a complete record of all factual information available, and no attempt should be made to interpret data, or assign design parameters or make design recommendations. If interpretation and recommended parameters are required, this requires completion of an interpretative report (see below).

Thus a factual report should be a clear, concise, complete and accurate description of the ground 'as found'. The nature of this description is dependent on the methods and techniques which have been used to obtain the information which is being reported.

It is important to realise that the true behaviour of the ground *in situ*, and its response to any construction work that may be undertaken which affects it, can rarely if ever be precisely determined through investigation techniques. All methods of inspection, sampling and testing of the soil that are available by their nature are limited to testing a limited volume of soil: this may amount to many tens of cubic metres of soil for *in situ* testing, or as little as a few grams of soil in a laboratory test. However, no test truly determines the response of the entire *in situ* soil mass for the full range of conditions. The difference in volume of soil testing can result in apparently factual data that are inconsistent: for example, it is well known that the factual result of laboratory permeability testing gives a different indication for a soil's permeability than a variable head test undertaken in a borehole. This is a reflection of the scale effects that apply: the structure of a small intact laboratory sample is much less likely to contain large fissures than the volume of soil affected by an *in situ* variable head test, and as a result often gives permeability results two orders of magnitude lower than the field test. Both tests are 'correct' and give factual information; neither necessarily gives an entirely true and accurate value for the soil permeability. Clearly, any report giving the measured permeability of the soil would need to give complete information on the nature of the test used to determine permeability, so that the end-user would have some indication of whether the test was obtained from a small sample or larger soil mass.

Factual reporting also needs to provide complete information about the nature of the techniques used to recover and test samples, since it is necessary to allow for how such techniques may have affected the information obtained. For example, soil and rock core recovered from rotary-drilled boreholes may have drilling-induced fractures which are not indicative of the state of the material *in situ*; methods of sampling can induce stresses and/or deformations in a sample, affect moisture contents and soil suctions, etc. Thus reporting a laboratory test without reporting the method by which the soil sample was obtained may render the entire test worthless, if the result may have been significantly affected by sampling disturbance.

In undertaking factual reporting, there is already much guidance. The majority of ground investigation techniques used on site, and laboratory test methods applied to determine soil parameters, are defined through British standards. These have relatively recently been supplemented or superseded by European standards, which while not identical in their

requirements, are equivalent in defining methods of undertaking and reporting work. Beyond this, there are national accreditation schemes, applicable to laboratory testing in particular (both geotechnical and environmental), which provide further guidance on the format and content of test reports. Other publications, for example the Specification for Ground Investigation (Site Investigation Steering Group, 1993), also include guidance as to what data should be reported as part of a factual report. However, there are still many techniques or tests where the available guidance is incomplete or absent, and particular project/client requirements or new research leading to changes in the understanding of how a test or technique works may result in the requirement to report factual information in a non-standard manner.

50.1.2 Methods of obtaining details

The information provided in a factual report obviously depends on the nature of the works being reported.

50.1.2.1 Non-intrusive

There are a variety of non-intrusive works that may be undertaken for engineering or geotechnical purposes that require reporting. Most typical would be a simple site walkover. This may be of a relatively small site, but could equally extend to very large worksites, particularly for larger infrastructure projects, where the site may be very linear in nature. Where sites are extensive in nature, the simple walkover may be supported or even replaced by aerial photographic survey. There is also the potential for geophysical surveys to be undertaken, which may be by surface (using hand-held or vehicle-mounted equipment) or airborne. Geophysical surveying may also be undertaken in a marine/overwater environment from boats.

Both the site walkover and the aerial photographic survey are fundamentally visual surveys, aiming to identify and report the same types of features. The intention is to identify and report those features of the ground that constitute potential hazards to the proposed project. Most commonly, this is areas of unstable land: active or relic landslips. Reporting such surveys therefore requires that all indications of current or previous ground movement be identified: distorted fences, tilted walls or trees and slip-scars are all obvious features. Aspects of site drainage are also important to identify, and this can be done through reporting of obvious drains, streams or larger watercourses, including those dry at the time of the survey. However, general geomorphology and land use (particularly the nature of the vegetation growing in a location) may also provide useful indication of the presence or absence of groundwater.

A specific form of visual survey is geological mapping. In many cases, the nature of the underlying geology can be adequately determined by a desk study, but there may be instances where accurate identification of a geological boundary is required.

Thus it can be seen that there is no convenient checklist in completing and reporting a visual site inspection: on one site, the nature of the vegetation may be irrelevant, on another it may give critical guidance to the presence of groundwater. A visual site inspection report needs to be as complete and accurate as possible, and to be undertaken with knowledge and understanding of the proposed project; each site inspection report must by its nature be as individual as the site to which it applies.

Where geophysics methods have been undertaken on a site, the information to be reported clearly depends on the particular method or methods employed. However, geophysical investigations are frequently undertaken and reported by specialist sub-contractors, and the reports normally include some degree of data processing or interpretation. This is acceptable within a factual report; however, the geophysical report should contain all relevant details of calibration applicable to the test, and the full test data from the tests should be included within the report (possibly as an appendix; due to the volume of data, an electronic format will likely be most suitable), such that further analysis/re-analysis of the test data can be undertaken.

50.1.2.2 Intrusive, large-face excavations

'Large-face excavations' refers to any situation where it is possible to view a section through the ground. Most commonly in ground investigations, this is obtained through the excavation of trial pits and trial trenches, for which there are codes and standards defining what should be reported. However, good information on the soil stratigraphy can also be obtained from natural exposures (for example sea-cliffs), and from large-scale man-made excavations. Such excavations may be connected to construction, possibly even the project for which data are being reported, and take the form of large box excavations or vertical shafts, giving a section through the ground from surface. There may also be more linear features at ground surface (cuttings, canals) or at depth (tunnels). Quarries and sand pits (whether operational or closed) also frequently provide potential for inspection of sections through the ground.

Whatever form the excavation takes, certain basic information should always be reported. The codes for reporting of trial pits should always be reviewed for any such excavation, since regardless of how the excavation was formed, it is, in effect, a trial pit. Precise location (in plan and elevation) must be given, along with details of date and weather. The excavation faces should be sketched (with scale/dimensions shown), and it is good practice to include photographs as well. Where samples are taken, it is important to accurately locate where these were taken, showing face of excavation, depth and orientation: again, photography can be useful here, with 'before' and 'after' images of where the sample was taken. Similarly, any *in situ* testing needs to be accurately located. Reporting of large-face excavations should also include details of how the face was created, when and by whom (as applicable). The absence of something within a large-face excavation can often be as important as its presence: for example, any trial pit or trench should generally indicate where groundwater was encountered, and if not encountered then this should be

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expressly stated. In other situations, it may be significant to report the absence of fissuring or a particular stratum, or of evidence of contamination.

50.1.2.3 Intrusive, small-sample size investigations

Intrusive small-sample size investigation invariably means boreholes of some form. There are a variety of techniques for forming boreholes, but any borehole formed for construction purposes should be undertaken broadly in accordance with the British/European standard, and will therefore be reported accordingly. If reporting is to deviate from the appropriate standard, it needs to be specifically instructed. It is important therefore to know what standard the work is being undertaken to and to be familiar with the reporting requirements of that standard, before the work commences on site, so that if variation from the standard is required, it can be instructed in good time.

50.1.2.4 Intrusive, non-sampling: *in situ* testing

In addition to boreholes, which are an intrusive technique that provides for recovery of soil samples, there are numerous methods of undertaking field testing of the soil. These vary greatly in the scale of the sample they test: pocket penetrometers and hand vane tests test a small volume of soil at shallow depth (typically within the wall of a trial pit); cone penetration testing tests a relatively small area of soil, but can test a continuous column of soil tens of metres deep; a plate loading test may be carried out at ground surface or in a shallow excavation, but affects a volume of soil to some depth below the level of the test plate. Hence, the reporting of such testing is specific to the method being applied. Most such field testing is fully defined by British/European standards, which accordingly define the reporting requirements.

While in some cases, the data from this type of test are relatively straightforward to report (for example, a pocket penetrometer), in other cases, considerable volumes of data may be created by the test – for example, pressuremeter tests. These tests require consideration of how all the data can be reported, and often require some degree of electronic/digital reporting as well as any printed report (see section 50.2).

Calibration records, where applicable, should always be included in any field test report. Where the report output is generated by processing of field readings, the actual instrument data should always also be made available as part of the factual reporting, to allow for re-analysis of the data if some discrepancy is detected at a later date. Additionally, any problems encountered during the testing or unexpected results from the tests should be highlighted when reporting *in situ* testing.

It is common for the more complex *in situ* tests (e.g. pressuremeter testing) to be reported in a sub-report appended to the main factual report. The specialist test report is often a combined factual and interpretative report even when submitted as part of the factual report, since the direct output of the tests is rarely in a format that is directly useful to an engineer: the tests may give pressures within the instrument or displacements of parts of the instrument, which are interpreted through appropriate calibration factors to give soil material properties.

50.1.2.5 Specialist down-hole tests

In addition to the range of standard tests that may be carried out during drilling, there are numerous tests that are undertaken after completion of a borehole; typically, these involve some form of groundwater monitoring, including pump tests, variable head permeability tests and packer tests. All these type of tests are well covered by appropriate standards and codes of practice, which include reporting requirements.

Another form of testing that may be done during boring but is often undertaken in completed boreholes is geophysics. These tests generate a variety of data, and therefore the reporting requirement is specific to each test. Typically, however, down-hole geophysics gives data that vary with depth. The data reported should always include sufficient information on the equipment used and details of installation so that the test could be repeated from the reported information. As with *in situ* tests and non-intrusive geophysics, the specialist nature of intrusive geophysical investigations means it is common for the report of the fieldwork to combine the factual information with an interpretation of this information, but for this combined factual/interpretative geophysics report to be included as a discrete sub-section within a factual report. Again, calibration information and raw data from the tests should be included within the report.

50.1.3 Reporting of field techniques

As detailed above, practically all field investigation techniques that may be used and which will hence need reporting are covered by a British and/or European standard, and thus the reporting requirements are fully defined. However, while such standards provide guidance to best practice, variation from them is possible. Regardless of whether a standard is being strictly followed or not, certain data must always be reported.

Any factual report of a field investigation technique needs to clearly state what method was used to obtain the data. Moreover, the actual type of equipment used should also be reported. Different sizes or makes of plant may perform differently, providing different results: for example, it has long been recognised that the type of SPT rod used during a test can affect the result. However, accurate recording of the type of plant and where appropriate the individual rig provides useful quality control on the works, and can be vital in interpreting the data recovered. As a further example, a particular rotary drilling rig repeatedly failed to recover soil core from a particular stratum, while other rigs had no problem. Knowing that it was the same rig that failed to obtain core recovery meant that the areas of 'no-recovery' shown on the log could be attributed to a problem with the drilling method, and were not indicative of a true soil condition.

Similarly, the actual operatives of any plant should be recorded: in the above case, different drillers operated the rig,

so it was known that the problem was not operator-dependent. However, it is possible to see apparent operator-dependent results in some cases: for example, using the same type of plant across the same site, a case has been observed by the authors where SPT results from one driller were consistently and significantly higher than those obtained by a second driller, despite boring in the same ground conditions. The project concerned was undertaken before the routine measurement of SPT hammer energy ratio, and had this form of calibration been available at the time of the works, this may have explained the variation in the recorded data. However, by knowing which driller worked on which borehole, it was possible to account for the apparently inconsistent test data from across the site.

All investigation locations need to be accurately located and orientated (if applicable). Someone picking up the report of the work at any point in the future, be it a year's time or a hundred years' time, should be able to work out exactly where the work was undertaken. For this reason, simple sketches of the worksite on their own are not adequate: the road layouts, buildings, trees, etc. shown on such sketches may completely change if the site is redeveloped, making locating the works impossible; cut or fill operations can change ground elevations by many metres, making it impossible to determine the level of the works.

It is therefore good practice to ensure that all works are surveyed to a recognised datum. Within the UK, the best and most obvious choice is the Ordnance Survey National Grid, since this is well established, widely accessible and unlikely to become redundant within the foreseeable future. Specific project grid/data may be used, but care should be exercised in selecting this option, since details of the project grid may become unavailable in the future (through disposal or other loss of records), leading to the reported information becoming unusable. Where project-specific grids/data are used, it should be clearly stated that the information on location and elevation is to a project grid, and the report should include details of the project grid.

For larger-scale projects, or those extending beyond the boundaries of the O/S (or other) national grid, altitude and longitude information may be provided, along with height above mean sea level, typically determined through use of a global positioning system (GPS). However, even here, there are different systems of mapping and determination of coordinates, so the equipment and methodology used in determining the site position should be explicitly stated.

While sketch plans alone are not adequate, such a sketch, or even better a scalable drawing or site plan, should be included in any report. Such drawings can be particularly valuable where works have some degree of orientation: trial trenches or inclined boreholes, for example, will have a definite orientation, while geological mapping of strata is likely to require reporting of stratigraphic dip direction.

The date of all works should always be given, with the text of any factual report stating the date of the full work period, and individual field test or borehole logs showing the date for that particular operation. Where applicable, times should also be given: an example of where this would be appropriate is where groundwater levels are to be monitored close to a tidal waterway and so may be showing tidal variation.

Weather conditions are also an external factor that should be reported: results of field testing or monitoring may be influenced by temperature, moisture or atmospheric pressure.

Any test or method of investigation reported should always include full records of any appropriate calibration factor that has been applied to the test, and also full records of cleaning, maintenance or calibration of any instrumentation used, if applicable: some instrumentation is capable of showing significant drift from true results if not regularly and correctly maintained, so without the calibration records, the data from such instrumentation cannot be treated as being reliable.

Zero readings and the absence of some material or behaviour should be reported where appropriate. By way of example, any intrusive investigation into the ground capable of identifying groundwater should always state where it occurred, or that it was not encountered. If gas monitoring is undertaken, readings indicating zero concentrations of particular gases should be explicitly reported. Knowing that there is the absence of something may be more valuable than knowing its concentration where it is present.

Individual features of the report by nature must always be tailored to the information being reported. While data such as borehole logs are typically reported to standard scales (often either 5 m a page or 10 m a page), the important thing is that the information is reported clearly and unambiguously. If, for example, the concentration of data obtained from a borehole to be reported makes a 4 m per page scale most suitable, this is the best scale to use, and the scale used should then be clearly stated.

50.1.4 Reporting of laboratory tests

Since the vast majority of laboratory tests are undertaken in accordance with an established code or standard, the reporting requirements are generally also prescribed. However, variations from the standard reporting format may be undertaken where specific project requirements demand it. In such cases, care needs to be taken to ensure that such non-standard reporting is undertaken consistently throughout. It should also be noted that any variation from the established codes/standards reporting requirements will prevent the test being reported as to the standard, even though the actual testing phase was entirely in accordance with the standard. If data are being reported to be supplied to a third party, there may be a requirement from that party that all tests are to a particular standard, so non-standard reporting may render the entire test unacceptable.

Non-standard reporting may be required because of a perceived weakness in the standard for reporting, perhaps because the precision of the standard reporting fails to provide sufficient discretion for the project underway. Alternatively, it may

represent improvements in the theoretical understanding of a particular test or soil behaviour in general, which has yet to be incorporated into the standards.

As with field monitoring techniques, laboratory tests may have applicable calibration factors that apply to the output data, calibration requirements for the equipment, or method detection limits indicating the smallest quantity that can be detected. Such information should be reported. Where a quantity is less than the detection limit, this should be explicitly reported: it is not correct to report zero concentration, where the test being used is incapable of discriminating between zero concentrations and very low non-zero concentrations.

50.1.5 Reporting of down-hole tests

In addition to the various field techniques and laboratory tests that require reporting, there are a variety of tests which may be undertaken in the field after the main period of fieldwork has been completed. These most typically relate to groundwater, involving tests to determine permeability, and as such the implementation and reporting of these tests is covered by published codes and standards, in the same way as intrusive field investigations and laboratory tests are.

However, such testing may also include geophysical investigations, or geo-environmental sampling and field testing of groundwater or ground-gas. The reporting of such tests may not be covered by established guidance, but may be treated as if it were a field test undertaken during on-site works, with the same requirements to report equipment used, calibration factors, location and orientation, date and weather, etc.

Reporting of field tests undertaken after the main fieldwork period does lead to potential issues over timing of reporting, as discussed in sections 50.4 and 50.5.

50.2 Electronic data

The traditional method of delivering a report, whether factual or interpretative, has been in the form of a bound hard copy. Smaller reports may constitute only a few pages, and be issued as 'letter reports'; large investigations may require reports consisting of many volumes. However, hard copy reports of this form alone are not necessarily the most convenient form for a factual report, since it is not easy to extract and manipulate data. Additionally, producing several copies of multi-volume reports typically requires a significant quantity of paper to be used, which may be at odds with modern standards of good environmental practice. For these reasons, there is an increasing acceptance of the need for electronic data transfer and reporting using electronic media.

The most common form of electronic data transfer in the UK is AGS: that is, data transfer according to the Association of Geotechnical and Geoenvironmental Specialists' Electronic Transfer of Geotechnical and Geoenvironmental Data format (AGS, 1999). This provides a common framework for transfer of geotechnical and geo-environmental data, enabling the data to be readily transferred and manipulated, or input into a database. While in theory, the use of this format means that data can be transferred seamlessly, the reality is that considerable work may still be required in manipulating data before they can be used.

While there is an increasing tendency for laboratories and ground investigation contractors to utilise systems that automatically generate AGS data during the reporting process, this is not universal. Some organisations generate AGS by manually inputting data. As a result, there is the potential for data to be mis-entered, leading to factual inaccuracies and inconsistencies to develop within the data file. There is software available that can check for the latter, but it is very difficult to confirm that factual inaccuracies are not present in the data. Knowledge of how the data are generated is therefore vital, as is information regarding the standard of the quality management system employed by the company or individual generating the data. It is good practice to spot-check any electronic data received against the printed master copy, but if there is any doubt as to the quality control of the data generating process, extensive checking is vital. Such checking is laborious and time-consuming, and unfortunately often not carried out to a sufficient standard. However, no electronic data should be used if there is any doubt as to their accuracy.

While the AGS format is very comprehensive, there are still tests or aspects of tests that cannot be adequately described using it. Geophysics results in particular are not readily reported through AGS. In such situations, it is normally appropriate to use a spreadsheet format for the data. The need to thoroughly check all such data remains.

While electronic data formats allow for rapid and easy data transfer, the real value is in the ability to create databases of project information that can be easily interrogated to enable specific data to be accessed in their entirety and with minimal delay. However, the ease with which data can be extracted means that where a database exists, it may be relied upon to such an extent that it becomes the sole source of data, and the data may not be referenced against the printed master copy. It thus is imperative that where a database exists, it is strictly controlled. While the ability to extract data needs to be limited only to the normal operational procedures of the company involved, subject to any appropriate commercial confidentiality, the ability to input or change data within a database needs to be controlled, and any data to be entered into the database need to have been checked fully beforehand.

The above points refer primarily to use of electronic data transfer/storage of traditional geotechnical factual data. The development of digital technology enables a range of new information to be obtained. For example, it is now possible to obtain digital data from monitoring instruments on rotary borehole rigs, which show factors such as drilling advance rates, rotational speeds and various pressures. Such information appears so far to be principally of value to the drilling contractor while doing the work on site, but may prove to have value in interpreting the ground conditions. Since the use of

this form of instrumented plant has not yet become widespread, such data are not, at the time of writing, routinely available. However, it illustrates how the range of data that may be available for reporting is not a constant, but will change as technology and methods of working change.

A form of digital reporting that is now routinely encountered is that of the Adobe Acrobat format (PDF files); it is now increasingly common practice for the master hard copy report to be provided in this format also, often directly from the original word-processing software used to create the report. Such electronic reports are very valuable, since they can generally be transferred easily, by CD, datastick or email, and they enable multiple copies of a report to be created and issued without the expense or environmental impact of multiple printed copies. However, a good quality electronic copy of a report requires some degree of processing: this file format allows pages to be bookmarked, such that for example, the first page of each chapter can be found through selecting the appropriate on-screen button. Large reports where this has not been done are considerably less useful than where the report is fully and sensibly bookmarked, since much of the time-saving that can be gained from using an electronic report is lost. While a PDF file provides a convenient form for storage and transfer of a report, the information cannot generally be readily extracted and manipulated, so the provision of a PDF format report can supplement, but does not replace, AGS format data transfer.

Another form of digital data that represents a new form for data to be presented is that of digital photography and digital imaging. Provision of core photographs has long been a standard requirement of any ground investigation involving rotary drilling for core; the widespread introduction of good quality digital cameras has resulted in the majority of such core photographs being provided not just as printed copies, but also as digital image files, which, as with electronic format reports, offers considerable advantages in copying or transferring the images. Care needs to be taken in the use of digital photography that any printed image is a true likeness of the actual soil, since the print is often made using a standard office printer, where the quality of colour reproduction may fluctuate. However, the same issue can occur in developing and printing of conventional film, so is not an issue that prohibits use of digital imaging. Digital photographs also offer the opportunity to manipulate colour or contrast of the image, which can be of great assistance in examining and identifying details of soil structure. Similarly, sample photography is now most likely undertaken with digital cameras, offering the same advantages and limitations as for core photography.

Where digital images are provided, the image file should be named in a sensible and consistent manner, such that it is obvious from the file name what the image shows.

A relatively new variant of core photography is the use of high-resolution core scanning. This generates a high quality image of the entire core, which with the appropriate software can be readily manipulated. The quality and resolution of the image is such that small details of the soil structure can be identified, and the images are far superior to traditional core photography. However, the availability of the scanners is currently limited, the scanning process is more involved than photographs, and requires more resources on site, and the digital size of the images is very large, leading to some problems storing or transferring the images (images cannot be emailed routinely due to their size).

In any form of electronic or digital data storage or transfer, there are a number of issues that must be addressed. AGS is a standard format across the industry, and the widespread use of Microsoft Office software makes it likely that any spreadsheet-based data will also be readily accessible. However, other electronic data formats may require specialist software to enable the data to be viewed/extracted. If the software is hard and/or expensive to acquire, or difficult to use, or consumes a lot of digital storage space on the computer, it may not be possible for all would-be users of the data to run the software. Even where the software is common, care must be taken to allow for different versions of the software. While newer versions of software are commonly written to be compatible with earlier versions, if the data are produced using the latest version of a piece of common software, a user operating an older version may be unable to fully access the data.

This leads to two related issues that need to be considered when considering electronic reporting. If a project is expected to be running over a prolonged period, it is possible that commercially available software used to report data at the start of the project will be upgraded during the course of the works. A decision will then need to be taken as to whether to upgrade the software, and accept that there will be some degree of inconsistency in the project data set, or continue to operate using the older software, which may lead to problems if companies generating the data have updated their systems, or if the software version ceases to be supported by the manufacturer.

Thought must also be given to the long-term availability of any software. Factual reporting from ground investigations is typically used relatively soon after it is generated, but often continues to be of use for many years after. If software is not common, then the problems of having the appropriate software available to access the data are likely to become more pronounced with time. It is possible that if the software operation requires a licence to function and the supplying company has ceased trading, the software will be completely unusable, and hence the digital data will be lost. Even if this does not occur, if data are supplied in an obscure format, it is important to record (non-digitally) what the format and appropriate software to access the data is, such that at any future time, it is possible to identify this and access the data.

Having focused on the potential difficulties of long-term data access due to software, it is appropriate to mention long-term data storage as a further issue affecting reporting. The lifespan of the printed page is well proven: the lifespan of a CD-ROM, or datastick, or magnetic storage tape, is less well demonstrated. If

digital data are to be archived, thought needs to be given to the environmental conditions in which it will be stored, such that the lifespan of the storage medium is maximised. Consideration also needs to be given to security and back-up copies. Archived data need to be accessible only to those who have authorisation to access the data, but depending on the value of the data concerned, thought should be given to having back-up copies stored separately from the main archive.

One issue applicable, but not unique, to electronic reporting is the need for personnel using software to be adequately trained. Knowing how a piece of software functions makes it less likely that errors will be made in generating/inputting data, and gives the operator a better idea of where problems are most likely to occur. It is important therefore that personnel involved in reporting are familiar with any software being used.

If the software is being used for any sort of analytical or interpretative function, then there is also a requirement to ensure that its operation is adequately validated. Commercial software does not always provide sufficient details or validation of its operation that its use would meet a reasonable quality system without further proof of its reliability.

50.3 Interpretative reporting

Interpretative reporting follows on from the factual report, and provides the manner in which the strictly factual data can be related to the specific project for which they have been obtained. Thus while it can useful to know what the proposed development of a site is to be when preparing a factual report, it is vital for an interpretative report.

The exact content of an interpretative report is, however, still open to some variation: the report may be being produced for a fully scoped and planned proposed development, or the proposals may still be quite vague. In the first instance, the proposals may already have determined that the foundations are to be piled, and the interpretative report is thus required to give guidance on the details of the piles likely to be required; in the latter case, part of the function of the interpretative report may be to provide recommendations as to the basic nature of the foundations (raft, piles, etc.). It is thus important that the requirements of the client in respect of the content and use of the interpretative report are well established and understood.

Interpretative reports are commonly also used to give guidance on design parameters for the soil. Again, where this is to be done, the client's requirements need to be fully understood. Design parameters can be presented as recommended values, design lines (possibly showing variation of a parameter with depth), upper bound and lower bound lines for maximum/minimum credible values, upper bound and lower bound lines for maximum/minimum possible values, etc. The nature of the proposed development and the expected method of design will affect which of these formats for reporting data is required. It is also possible that specific design parameters are not required, only combined plots showing the actual field and laboratory test data, enabling a design consultant to review the data and select their own design values. Alternatively, particular design codes, standards or methodologies may specify how geotechnical design parameters should be selected. The data available may also affect the method of selecting design parameters: many field tests tend to give results exhibiting significant scatter in value, where boundary lines may be more appropriate than a single design line. Conversely, if the available data are very limited in quantity, a conservatively selected single value may be more appropriate.

Interpretative reports also enable details of the factual report to be reviewed and possibly explained. A factual report may contain borehole logs, and such logs may be present on a cross-section, but it is incorrect to interpret the geology between boreholes in a factual report. In an interpretative report, interpreting the geological stratigraphy between boreholes is generally a fundamental part of producing the report, since it provides the understanding of the form and nature of the soil mass. Through such a process, various geohazards may be identified, for example faults identified from vertically displaced stratigraphic boundaries, or buried river channels with possible high volume groundwater flow from unexpected soil types encountered in a borehole. Thus while it is relatively simple to complete a factual report, since it merely requires complete and accurate reporting of all information, the interpretative report is more difficult to complete to a useful standard. It is not an exercise in repeating information from the factual report, but requires an intelligent and informed assessment of that factual information relative to the proposed development for which the information has been gathered. Moreover, the briefing to produce the report may require that it allows for a variety of design methods, construction techniques, building layouts, etc., since all aspects of site investigation and reporting tend to occur early in a project's life when quite fundamental changes to the project may still occur.

Interpretative reports often include recommendations for further investigations, where the assessment of the available factual information reveals deficiencies in the quality or quantity of the available data.

It should be noted that an interpretative report may be combined with a factual report, where the requirement of the report is both to provide full and complete factual information and to include an assessment of the data meeting the standards of an interpretative report.

50.4 Other geotechnical reports

The most common form of geotechnical reports are the factual and interpretative reports that result from some form of site or ground investigation. However, there are a number of other geotechnical reports which may be encountered.

Generally, the discussion of reporting given here is focused on geotechnical reporting only, with some consideration of geo-environmental issues. However, geotechnical issues are not the only concern at many sites, and it is often most efficient to combine all investigation and reporting of a site into one work package. Thus both factual and interpretative reporting may be required to consider factors such as geology and geohazards,

environmental contaminants (in soil, water and air), broader environmental issues (e.g. flood risk, naturally occurring radon gas) and heritage and archaeological issues (including possible issues of unexploded ordnance). Such issues should typically have been identified in the desk study, and may have been adequately addressed at that stage, but there is the potential for specific site works related to these aspects which will then need to be reported and the implications of what was found will need to be discussed.

The introduction of Eurocode 7 has led to the production of ground investigation reports (GIR) and geotechnical design reports (GDR). The GIR broadly combines the factual report with interpretation to generate design parameters which are then fed into the GDR; the GIR in fact forms part of the GDR. Eurocode 7 defines the format for presenting information, and requires known limitations of test results to be stated, so that users of the data have an indication of their reliability. The GDR provides the foundation design and recommendations that may formerly have been encountered in an interpretative report. It must include the assumptions and data that feed into the design, the methods used in the design, and verification of safety and serviceability. It also requires that supervision, monitoring and maintenance requirements of the completed structure be reported and provided to the owner/client. The specific requirements of the GIR and GDR are stated more fully in EC7.

Geotechnical baseline reports (GBR) are a specialist form of geotechnical report which are produced for commercial rather than technical purposes. They may also be known as ground reference conditions. They draw on and interpret the available data to define baseline conditions relevant to the proposed construction. These baseline conditions are applicable to a specific contract and establish what conditions a contractor should expect to encounter in the ground when undertaking works under that contract. If conditions are worse than these, and the contractor can demonstrate a resulting loss or delay, then a compensation event may be triggered. A GBR must contain statements that are concise, measurable and clearly defined, with no ambiguity or uncertainty, and which are based on a reasonable and realistic assessment of what will be encountered. They define what ground conditions are foreseeable, and hence what ground conditions are unforeseeable, relative to the works to which the report relates. A GBR is not an interpretative report, nor is it a basis for design; it is a means by which the allocation of ground risk is assigned between contractor and client; the ground conditions should normally be stated as accurately as possible: if the conditions are stated to be better than they actually are, the client will be liable to increased claims for compensation; if a worst-case attitude is taken and the ground conditions are described as worse than they actually are, the contractor will assume that there is an increased risk, and will price for the works accordingly, again leading to financial loss to the client. However, in practice, the commercial nature of a GBR does sometimes result in an unrealistic assessment of the ground conditions, reflecting the client's attitude to financial risk.

Another specialist form of geotechnical report is a risk register. In reality, a risk register is not specifically a geotechnical report, since it should apply to all aspects of a project, and list all risks to the project. The risk register is, as its name indicates, a means of identifying and tracking all risks to the project. Typically, the register would detail the nature of the risk, the likelihood of it being encountered and its potential impact on the project, to give an overall risk status, using standard risk assessment procedures. Recommendations as to how to mitigate the risk may then be given, with the party responsible for undertaking the mitigation identified, and the residual risk after mitigation being stated, along with where that residual risk lies. While a risk register is not specifically a geotechnical report, geotechnically related risks are often some of the more significant to a project, due to uncertainty about the ground; hence risk registers routinely require a geotechnical input.

Where field monitoring or instrumentation is installed, there will generally be a requirement for ongoing monitoring. Such monitoring will require to be reported. The frequency and manner in which these reports are issued will depend on the frequency of the monitoring, and the requirement of the project. Typically, following ground investigation works, groundwater monitoring instruments are placed in the ground. Monitoring of these tends to be daily while site works are ongoing, but then becomes less frequent post-site work. Monitoring at monthly or three-monthly periods is not untypical, and while the results of each site visit should generally be provided to the engineer within a day or two of the visit, the contractor would normally only be expected to produce a factual report on the monitoring at the end of the monitoring period. However, monitoring of instrumentation can continue into the construction and post-construction period, in which circumstances, reporting may need to be more formal and frequent. This may require, for example, formal issue of daily groundwater data, or real-time remote monitoring of displacements of a retaining wall. In such cases, reporting is most likely to be electronic, perhaps with a summary printed report periodically. Where this type of data is being generated and issued, it is important to ensure that the critical data are prominent, and that the ability to generate and issue huge quantities of data is not allowed to swamp the recipient of the report, potentially resulting in significant information not being identified and acted upon. The requirements for frequency and format of monitoring reporting will generally be identified in the specification for the broader works.

50.5 Reporting production and timescale

Having considered the nature of what may be reported in a geotechnical report, and how that information may be reported, consideration needs to be given to the practicalities of the report.

A geotechnical report from a ground investigation is often required early in a project's life, and may be a requirement before the proposed design can be progressed. Therefore, the timing of the report is of importance. Where ground investigation works

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are very large, the time required to complete and issue a report may be considerable, as potentially will be the time needed to fully check and correct the report. Allowance needs to be made in any project programme for this time period. In some circumstances, it may be appropriate to issue reports in stages: long-term monitoring by definition is completed many months after the fieldwork stage of a ground investigation is completed, so it would be inappropriate to delay issuing the factual report from the fieldwork until the monitoring is complete. Similarly but less obviously, some laboratory testing (particularly drained tests on clay soil specimens) can take prolonged periods to complete. Where the available laboratory resource to complete these tests is limited and multiple tests are required, it may take several months before this part of the laboratory test programme is completed. In such cases, the demand for basic stratigraphic information may require that these laboratory tests are reported as a later addendum to the factual report.

The programming of production and issue of the interpretative report can also be a significant factor. It is not uncommon for the interpretative report to draw on data from more than one report: there may be a desk study, several phases of ground investigations for the project concerned, and a variety of historical data. As previously noted, the interpretative report is in some ways harder to produce than the factual report, since it is not a simple statement of fact, and therefore may take considerably longer to complete than the factual reports upon which it is based.

When generating a report, it is necessary to consider how many copies of the report are required. It will rarely be sufficient to generate just a single copy for the client: additional copies may be needed by one or more design engineers, and other interested parties (architects, insurers, etc.). However, efforts should be made to ensure only the number of reports actually required are produced, both for commercial and environmental reasons. Production of electronic copies of the report are valuable in this respect, since they allow the report to be readily issued as required, in whole or in part.

In some circumstances, the issue of payment for the report also needs to be carefully considered. The typical factual report from a small ground investigation will be relatively straightforward, and a simple lump sum for its production may be appropriate. However, if the works are larger in scope or undertaken under a term contract and may vary considerably in nature and extent, such a mechanism may be inequable. Allowance for reporting costs to be based on the value of the fieldwork undertaken is in some circumstances appropriate.

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McDowell, P. W., Barker, R. D., Butcher, A. P., Culshaw, M. G., Jackson, P. D., McCann, D. M. *et al.* (2002). *Geophysics in engineering investigations*. CIRIA Report C562. London: Construction Industry Research and Information Association.

Site Investigation Steering Group (1993) *Site Investigation in Construction 2: Planning, Procurement and Quality Management*. Site Investigation in Construction. London: Thomas Telford.

50.6.2 Useful websites

Association of Geotechnical and Geoenvironmental Specialists (AGS); www.ags.org.uk/site/home/index.cfm

It is recommended this chapter is read in conjunction with

- Chapter 9 *Foundation design decisions*
- Chapter 44 *Planning, procurement and management*
- Chapter 52 *Foundation types and conceptual design principles*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Section 5: Design of foundations

Section editor: **Anthony S. O'Brien**

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Chapter 51

Introduction to Section 5

Anthony S. O'Brien Mott MacDonald, Croydon, UK

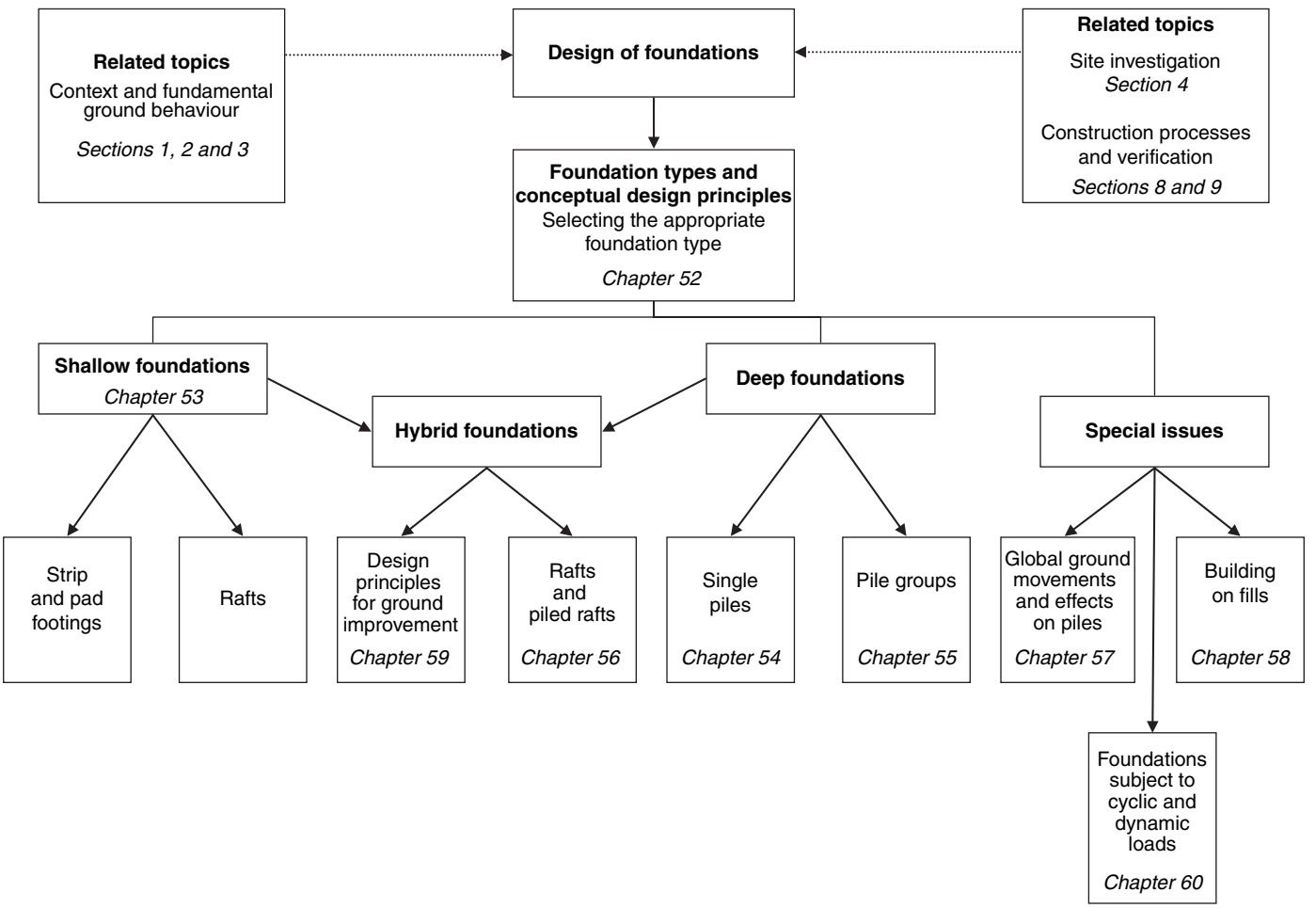


Figure 51.1 Layout of chapters in Section 5

Figure 51.1 outlines the layout and contents of Section 5 *Design of foundations*.

Foundation design is usually divided into two broad categories: shallow and deep, and these categories are also used here. In modern foundation engineering it is helpful to consider a third category: hybrid. Hybrid foundations have their own unique features, although their design usually requires an assessment of both shallow and deep foundation behaviour. Examples of hybrid foundations include deep ground improvement and piled rafts.

The final set of topics, special issues, include building on fills, the influence of global ground movements on deep foundations, and foundations subject to cyclic and dynamic loading, including earthquakes.

In common with other sections of this manual, this section is intended to provide guidance to practising engineers. Given the enormous breadth of the subject and the vast range of ground conditions and structures which a foundation designer may encounter, this section cannot, within the available space, be completely comprehensive. The intent is to outline:

- the fundamental principles of good design practice;
- the key mechanisms of ground and ground–structure interaction behaviour which need to be considered;

- commonly used design methods;
- the need to integrate a range of specialist disciplines at different stages of the design process;

and to provide:

- references for detailed study;
- some brief case histories which highlight key issues.

A quote from Terzaghi (1936) is particularly pertinent:

> Whoever expects from soil mechanics a set of simple, hard and fast rules for settlement computations will be deeply disappointed…The nature of the problem strictly precludes such rules.

Hence, it is essential for the foundation designer to think carefully about the likely deformation and failure mechanisms which may occur both during and after construction; then compile a checklist of questions which need to be considered and answered.

There have been enormous developments in foundation engineering during the last few decades. Despite these developments, failures still regularly occur and there are also numerous examples of grossly overconservative design and poor construction practice. There are several commercial and technical factors that can cause these problems. Technically, it is important to:

(i) Recognise that foundation engineering is a 'process', and that success depends on a series of interlinked activities during *both* design and construction.

(ii) Have a coherent approach to ground risk management; the 'geotechnical triangle' is a valuable framework in this context. An understanding of the site's history and its groundwater regime are critically important.

(iii) Have good communication across different design/construction teams.

(iv) Develop a good understanding of ground–structure interaction. It is vitally important for geotechnical and structural engineers to have early two-way discussions, so that the overall behaviour of the proposed works are understood. In particular, the best opportunity for economic foundation design is to set realistic (rather than arbitrary and usually overconservative) limits on foundation movement.

(v) Have a good awareness of relevant case histories of past foundation performance, and to carefully assess the relevant parameters for assessing stability – and in particular, foundation deformation. Sophisticated analysis is not a substitute for a proper selection of design parameters, based on well designed and supervised ground investigations.

Chapter 9 *Foundation design decisions* provides an introduction to these themes, and these are developed in more detail throughout Section 5.

Although it is a simplification, it is fair to state that routine ground investigation and analysis methods can lead to:

(i) overconservative design of foundations in heavily overconsolidated soils, such as stiff clays;

(ii) unsafe foundation design in normally and lightly overconsolidated soils, such as soft clays.

Modern developments in ground investigation and geotechnical analysis can avoid these problems, although these modern techniques are still under-used across the civil engineering industry. The chapters in Section 5 highlight some of the developments which are mature enough to be used more routinely. It is hoped that Section 5 will stimulate an improvement in foundation design practice.

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Chapter 52

Foundation types and conceptual design principles

Anthony S. O'Brien Mott MacDonald, Croydon, UK

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The main types of foundation include: shallow foundations (pad, strip, raft); deep foundations (piles, caissons, barettes) and hybrid foundations (deep ground improvement, piled rafts). Foundation engineering requires a broad range of skills, from an appreciation of geology and hydrogeology to structural engineering. The performance of foundations is dependent not only on how they are designed, but also on how they are constructed. The overall design process needs to be well managed, to ensure that there are good communications between different design teams and between design and construction. To provide a framework for selecting the most appropriate type of foundation, a useful mnemonic is 'the 5 S's':

S – Soil; S – Structure; S – Site; S – Safety; S – Sustainability

The magnitude of allowable foundation movement is a key factor in determining the type, size and cost of foundations, and this chapter provides guidance on routine limits. Parameter selection is a common pitfall, and the critical information requirements are described.

52.1 Introduction

This chapter outlines the types of foundation that are commonly used, together with the key factors which need to be considered before selecting a particular foundation type.

Foundation engineering requires a working knowledge of:

- geotechnical engineering;
- construction methods;
- ground-structure interaction.

Professor Peck (1962) outlined the three areas of knowledge that are needed in geotechnical engineering:

(i) a knowledge of precedents, i.e. a knowledge of case histories;
(ii) a working knowledge of geology;
(iii) familiarity with soil mechanics.

In the vast majority of cases, a foundation design is developed on the basis of an appreciation of the site geology, the nature of the proposed structure and any particular site constraints. The role of analysis is then to check that a proposed foundation design will be acceptable. Although analysis is important it is just one part of the overall design process.

Many young civil engineers will have some knowledge of (iii), but it is important that they endeavour to develop their knowledge of (i) and (ii) above. A knowledge of geology (and hydrogeology) is critically important, since it enables many of the risks associated with foundation engineering at a particular site to be assessed. Simplifying assumptions have to be made before any analysis can be carried out (this includes sophisticated computer modelling). The site geology and hydrogeology have to be considered and understood, as early as possible in the project. Discussions should be held with an experienced geologist. Relevant technical literature and, particularly, local case histories should be reviewed. Once this has been done then the appropriateness, or not, of certain assumptions inherent in a particular method of calculation can be judged, and the likely errors associated with the predictions from calculations can be assessed. A knowledge of case histories is probably the most important of the three attributes, since this enables the engineer to have an understanding of:

(i) what has worked, or not, in the past;
(ii) the consequences of particular construction activities on subsequent performance;
(iii) past performance, against which the reliability of different methods of analysis can be compared;
(iv) when a proposed activity is going beyond what has been attempted before – this will then require a special effort and expert advice in order to safely develop the design.

A foundation is a structural member and supports a superstructure. So an awareness of the sources and nature of structural loads, the structure's tolerance of foundation movements, and an understanding of ground–structure interaction is also needed. Finally, the foundations must be built economically and safely. Hence, a designer needs to have an appreciation of construction methods and equipment, in order to develop a design that is practical to build.

This list of attributes, which a foundation designer needs to possess, is rather intimidating. The author has not yet come across the god-like creature who possesses a perfect knowledge of all these topics. Therefore, first and foremost, the foundation designer must be prepared to ask questions and discuss these with other design and construction specialists (e.g. geologists, structural and material engineers, specialist sub-contractors, etc.). Good foundation design is usually the result of a multidisciplinary team effort, where two-way communications across the team occur regularly and frequently.

52.2 Foundation types

There are two generic types of foundation: shallow and deep. Foundations developed on improved ground can be considered to be a hybrid of both shallow and deep foundations, although ground improvement design requires additional considerations. The advantages and disadvantages of different foundation types are outlined in **Table 52.1**.

(1) *Shallow foundations*: these typically comprise pad, strip or raft foundations; refer to **Figure 52.1**. Pad foundations, **Figure 52.1(a)**, may be square, circular or rectangular. A pad foundation usually only supports one or two columns. Strip foundations (**Figure 52.1(b)**) are commonly used to provide support to a load-bearing wall or for several closely spaced columns. The length of a strip foundation is much greater than its width. A raft foundation (**Figure 52.1(c)**) could support the entire structure or a substantial part of it. A 'compensated' raft (also known as a 'buoyant' raft) is a special type of raft. It includes a void, thereby reducing the 'net' increase in bearing pressure (section 52.5) on the ground below the foundation. This will increase the factor of safety against bearing capacity failure and reduce foundation settlement, compared with a conventional raft. The structural form of shallow foundations can vary, but pad and strip foundations could comprise either mass concrete or reinforced concrete, whereas raft foundations usually comprise reinforced concrete. The soil resistance to the loads applied by the structure are predominantly developed by the near-surface soil below the base of the foundation. Shallow foundations are usually constructed using simple excavation plant and general labour. Tomlinson *et al*. (1987) gives useful advice on routine foundation design for low-rise buildings.

(2) *Deep foundations*: for the majority of routine structures, deep foundations comprise piles; refer to **Figure 52.2**. There are numerous different types of pile, discussed in Chapter 54 *Single piles* although piles are usually classified as either driven or bored. Pile construction is a specialist activity and should only be carried out by contractors who have the appropriate equipment and experience.

The layout of pile foundations can vary substantially from a single pile below a load-bearing column, to a large number of piles in a group supporting the entire structure, via a pile cap (which transfers load from the structure directly into the piles). For a conventionally designed pile group, it is assumed that the piles carry the entire load.

Deep foundations also include barettes, shafts or caissons, which tend to be bespoke foundations for special circumstances. Caissons, barettes and shafts typically have a large diameter or cross-sectional area relative to their depth in comparison with conventional piles. Barettes are rectangular in plan, and are installed using diaphragm-wall construction techniques. Both shafts and caissons are circular in plan; however, they are usually constructed by different methods. Shafts could be hand-dug or excavated using back-actors, with excavation support installed top-down, using pre-cast concrete rings or cast-*in situ* concrete lining. Caissons are usually jacked or sunk into place using combinations of kentledge, excavation and perimeter lubrication (using bentonite). For both piled foundations and deep shafts the load transfer to the ground is via shear along the vertical sides of the shaft or pile and by end-bearing resistance.

Piled rafts are a hybrid type of foundation with the structural load being transferred to the soil via the raft (as for a shallow foundation) and the piles (as for a deep foundation). The principal difference between a piled raft and a conventional pile group is that the piles are designed to mobilise most of their ultimate load-bearing resistance and act as settlement-reducing elements whilst the raft provides the appropriate overall factor of safety against bearing capacity failure. Compared with a conventional pile group, a piled raft has a relatively small number of widely spaced piles (**Figure 52.3**). Piled-raft design requires specialist input.

(3) *Foundations on improved ground*: in general, the purpose of ground improvement is to increase the strength and stiffness of the ground below a proposed structure, and then shallow structural foundations are constructed on the surface of the improved ground (**Figure 52.4**). Below the structural foundations, a granular layer (sometimes reinforced with geogrids) is often used to transfer loads into the reinforcing elements (**Figure 52.4**). The mechanism of ground improvement is usually either:

- **(i)** Mass reinforcement – usually for clays, silts or made ground. A large number of stiff elements are introduced into the compressible layer.
- **(ii)** Densification – usually for sands or gravels. A compaction process forces a rearrangement of the soil particles into a denser state.

Reinforcement: techniques such as vibro stone columns or deep soil mixing facilitate the replacement of soft compressible materials by stronger materials introduced into the ground via specialist equipment. These stronger elements carry most of the foundation load. However, the remaining soft compressible materials will also carry some of the foundation loads, hence the term 'reinforcement'. It is important to note that the reinforcing elements usually have negligible tensile and bending resistance. Sometimes reinforcing elements such as stone columns are called 'stone piles'; however, this is quite incorrect and misleading. Foundations built upon improved ground will behave in a quite different manner to piled foundations (**Figure 52.5**).

Densification: soils such as loose sand with a low silt or clay content can be compacted into a denser state. Once densified the sand will be a more competent stratum and be able to carry the foundation loads directly. Hence, conventional shallow foundations can be placed on the densified layer.

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Densification may be carried out via a wide range of specialist equipment. The key issue for densification projects is usually the relative density that can be achieved compared with the value which is needed to ensure that foundation settlement is acceptable.

The success of ground improvement (both by reinforcement or densification) is sensitive to the nature of the soil profile and the soil macrofabric. For example, densification by vibrocompaction may be straightforward if the sand is homogeneous and has a low silt or clay content. Compaction will be much more difficult and often less effective if there are numerous bands of silt or clay in the sand. These soil features can be easily missed by conventional boreholes and sampling at discrete depth intervals. Continuous sampling or profiling techniques (such as static cone penetration testing, CPTs) are usually very beneficial when designing ground improvement. Similar considerations can apply to reinforcement techniques. Quality control and field testing in order to verify the effectiveness of ground improvement are a significant component of any ground-improvement project.

52.3 Foundation selection – conceptual design principles

52.3.1 Information requirements and the foundation design process

Figure 52.6 outlines the typical information requirements, the design deliverables (reports, etc.) and the project phases that are followed to design and construct a foundation. During the early stages the key questions are:

(1) *Site geology, hydrogeology and history*: what are the geological age and the depositional environment of the main deposits under the site? What subsequent changes may have occurred (e.g. landslides, man's influence)?
(2) *What will be built?*: form of structure, likely construction methods, main site constraints, applied loads, principal constraints (e.g. allowable structural movements)?
(3) *Engineering knowledge*: what existing knowledge do we have? For example, relevant technical literature, good case histories, local customs and experience or expertise.

This collation of available data into a desk study report is an important and cost-effective means of managing ground risks on a project. Sole reliance on boreholes in the absence of a proper desk study is potentially very dangerous. The main ground hazards can be identified from the desk study and the key features of ground behaviour can be assessed. This will inform the design of intrusive ground investigation (refer to Chapters 40 *The ground as a hazard*, 41 *Man-made hazards and obstructions*, 43 *Preliminary studies* and 44 *Planning, procurement and management*). It is often the case that designers 'inherit' ground investigation reports from previous or adjacent projects and studies. It is very important that the scope, adequacy and reliability of this information are assessed in the context of the current project requirements. If the structure has been relocated, or more onerous loading or allowable movement criteria are required, then an additional ground investigation may be needed (perhaps using more sophisticated investigation techniques).

Foundation design is an iterative process. Once relevant information has been obtained for the site, the key data should

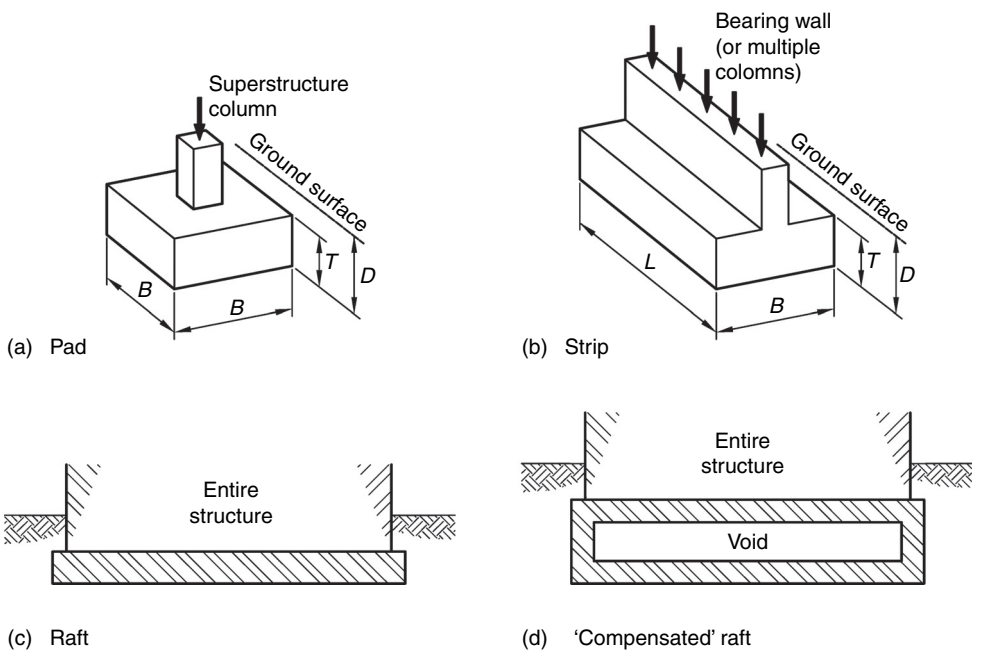


Figure 52.1 Types of shallow foundations

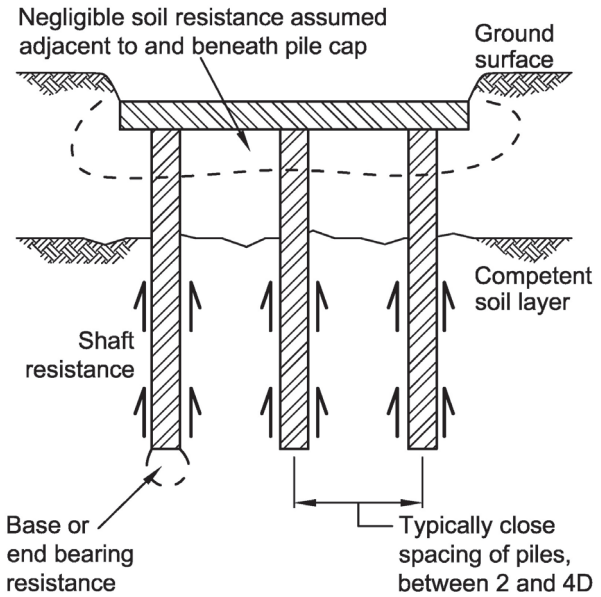


Figure 52.2 Deep foundation, conventional pile group

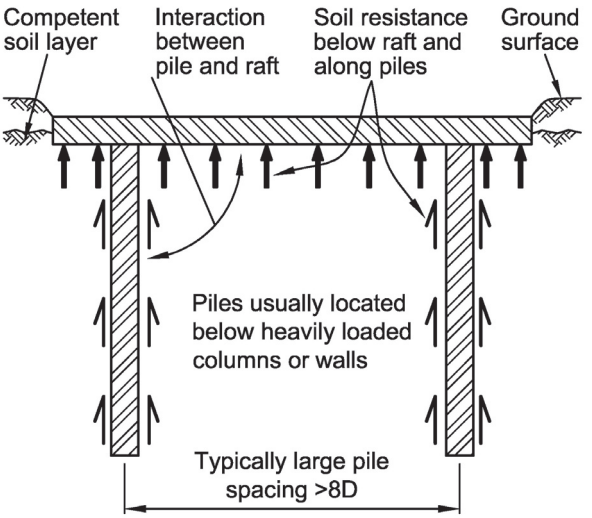
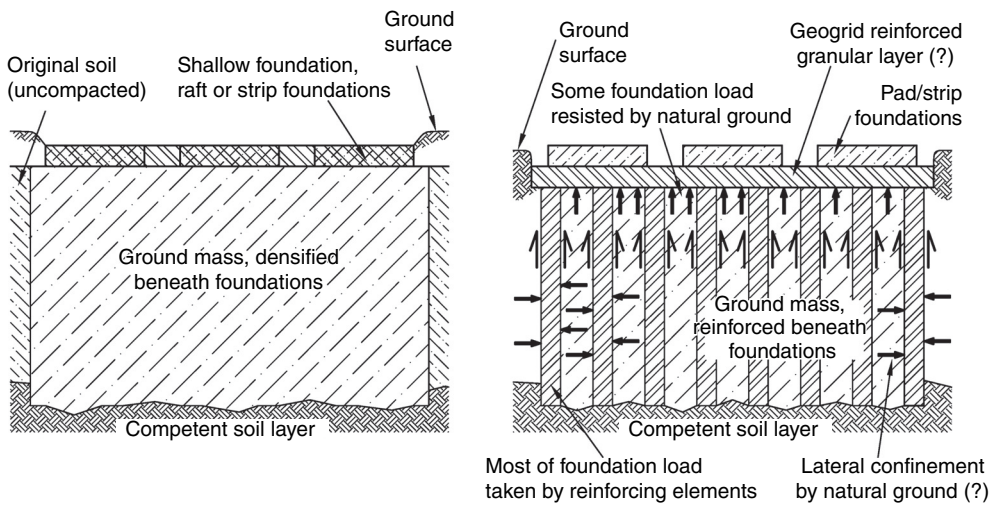


Figure 52.3 Hybrid foundation, piled raft



(a) Ground improvement by densification

(b) Ground improvement by reinforcement

Figure 52.4 Hybrid foundation – deep ground improvement

be summarised on a plan and vertical cross-sections through *and* beyond the site boundaries. Drawn to scale, with levels referenced to an appropriate survey datum, as a minimum, the following features should be drawn:

- **(i)** the previous and proposed ground surfaces;
- **(ii)** any previous and neighbouring structures and underground services;
- **(iii)** ground profile, strata boundaries (including selected ground properties from testing);
- **(iv)** groundwater levels;
- **(v)** outline of the proposed structure and proposed locations for foundations;
- **(vi)** list as bullet points, the key site constraints and hazards.

Based on a knowledge of the ground profile, the type of structure and key site constraints, an experienced engineer should be able to identify appropriate conceptual designs for the foundations. Analyses would then be carried out to check that this concept had an acceptable level of stability and that foundation movements were acceptable. If these are not acceptable, then the concept would be modified (say, pile foundations would

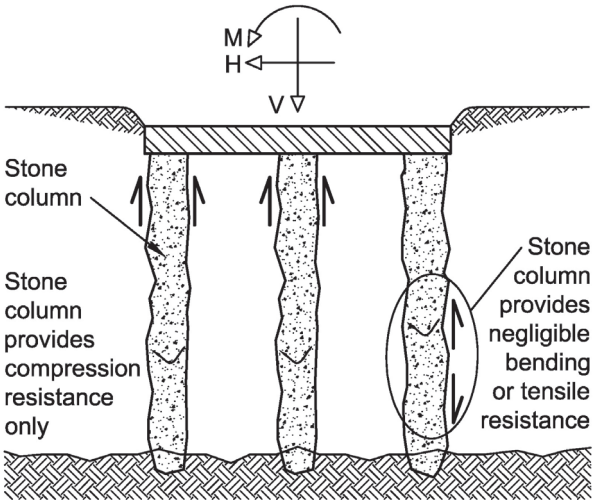


Figure 52.5 Ground improvement as a reinforcement method has limited bending and tensile resistance

be deepened, or shallow foundations would be widened) and rechecked. It is common to consider a few different options. Once technically acceptable foundations are identified, then the costs are assessed. Throughout this conceptual design process, the way in which uncertainties can be best managed in order to minimise risks needs to be considered, together with the buildability of the concept (in essence, 'buildability' is a consideration of how simple, quick and safe the proposed design will be to construct). Once an acceptable foundation design has been identified, more detailed analysis is carried out to finalise the design and prepare drawings and specifications for construction. The nature of the ground risks will usually inform the development of site-specific specification requirements, which may involve observations by experienced engineers or geologists at critical construction stages or specific field tests.

The foundation design process does not end when design drawings are issued. Given the intrinsic uncertainties associated with variations in ground conditions, and the way in which different construction processes may affect subsequent ground behaviour, it is always important that the designer has the opportunity to verify critical design assumptions. However, with some forms of modern contracts and procurement practices (e.g. Design and Build), it can be challenging for a designer to get adequate site supervision opportunities. Nevertheless, it is important that this happens.

52.3.2 General considerations

Any foundation design has to include a consideration of the following:

(i) acceptable stability and deformation;
(ii) risk management;
(iii) costs and programme for construction.

Foundation type	Advantages	Disadvantages
Pad/strip footing	(1) Simple to construct (2) Cheap and quick	(1) Competent soils (note 1), need to be reasonably close to ground surface (typically less than 3 m or 4 m) (2) If the groundwater table is above the base of the footing and there are permeable soils then barriers or dewatering are needed during construction (3) Limited application if applied loads include large horizontal forces or overturning moments (4) Vulnerable to large differential settlements if applied loads vary significantly across structure or if ground conditions are heterogeneous (5) Interaction between adjacent footings, if closely spaced?
Raft foundations	(1) Simple to construct (2) Specialist equipment and sub-contractors not required (3) If sufficient rigidity, then potential for differential settlement is significantly reduced (4) Can 'bridge' across local soft pockets of soil	(1) As noted above (2) As noted above (3) Large area of excavation required, hence, large amounts of spoil generated (particular problem if excavated soils are contaminated) (4) Depth of influence of significant increases in stress will be considerable, will it affect underground infrastructure? Are weak layers present at depth? (5) Raft deformation needs to be carefully checked, is movement acceptably small?
Shallow foundations generally	(1) Simplicity (2) Cheap if appropriate conditions	(1) Inappropriate if site affected by 'global' ground movements (note 2) (2) Excessive depth may be needed to locate foundation below seasonal changes in moisture content or frost penetration (note 3), or, if close to rivers, potential scour depth (3) Inappropriate if desired footing depth would lead to undermining of existing foundations, utilities, roads, canals, railways, etc.

(Continued)

Foundation type	Advantages	Disadvantages
Piled foundations	(1) Large variety of pile types, diameters, depths are possible, so very large applied loads can be resisted (2) If near-surface soils are weak or of variable thickness then pile lengths can be readily modified to suit (3) If near-surface soils are contaminated then exposure can be minimised by appropriate choice of pile type (4) If site boundaries limit width of shallow foundations then piled foundations may be more appropriate (5) Horizontal and moment loads can be resisted (although a pile group may be more appropriate, rather than a single pile)	(1) Can be relatively expensive (2) Construction will require specialist contractor input, hence, construction sequence may become complex (3) Piling construction can be adversely affected by local variations in ground and groundwater conditions; technical and commercial implications? (4) Pile capacity can be significantly affected by construction method and equipment and timing of key activities (5) Costs and time to carry out pile testing (6) Pile construction may adversely impact adjacent structures, utilities, etc.
Caissons/shafts/ barrettes	(1) Can provide extremely high capacity foundations for 'special' structures (2) Can provide relatively high stiffness/capacity to resist large horizontal loads or overturning moments (3) Shafts, in appropriate circumstances, can provide high capacity foundations in sites with limited plan area or headroom	(1) May be complex or expensive to construct (2) Design and construction requires expert input, these skills are usually in short supply (3) Verification of design assumptions may be challenging (4) Deformation behaviour requires careful assessment, may not be amenable to routine analysis methods
Deep ground improvement	(1) Can create cost-effective foundations in poor ground conditions (2) Wide variety of techniques are available, which can allow a broad range of ground conditions to be improved (3) Techniques can optimise the behaviour of the *in situ* ground, hence is a 'sustainable' solution, with a relatively low carbon footprint	(1) The wide range of techniques can be difficult for non-specialists to understand, hence, selection of appropriate method can be challenging (2) Effectiveness of ground improvement can be very sensitive to local changes in ground conditions and to changes in 'workmanship', installation method and equipment (3) A significant effort is needed to verify the effectiveness of ground improvement, this can often be underestimated (in terms of cost/time and complexity of interpretation)
Piled rafts	(1) More economic than conventionally designed pile groups (2) Since fewer piles to install, quicker to construct (3) Appropriately located piles, below large column loads, can significantly reduce structural forces in raft, hence, raft can be thinner, with less steel reinforcement	(1) Complex ground-structure interaction analysis is required, expert input will be required (2) Due to (1) above, more sophisticated ground investigation and interpretation of ground conditions may be required (3) Raft behaviour is important. Hence, as for conventional rafts, competent soils needed at reasonable depth below ground surface (4) Due to (3) above, may not be appropriate if adjacent construction activities lead to 'undermining' of raft

Notes

1. The definition of 'competent' soils will depend upon the foundation loads and allowable movements. Typically, competent soils include firm to stiff clays, becoming very stiff at depth, or medium dense to very dense sands.
2. Global ground movements may develop due to a wide range of causes which are independent of the proposed foundation loads, refer to Chapter 53 *Shallow foundations*. For example, the presence of old mine workings, solution features, adjacent slope movements, etc. may lend to 'global' ground movements.
3. Minimum depths for frost protection and seasonal moisture content (pore water pressure) change are given in local building regulations and codes, also refer to Chapter 53 *Shallow foundations*. Seasonal moisture content changes are mainly affected by the type of vegetation present on site; if there are high water demand trees on high plasticity clays then the effects can be significant and extensive (refer to NHBC Standards, 2003 and Chapter 53).

Table 52.1 Advantages and disadvantages of different foundation types

52.3.2.1 Acceptable stability and deformation

The foundations need to carry the foundation loads safely; hence, adequate factors of safety against collapse are required and excessive foundation movement must be prevented. There can be a wide range of different collapse or deformation mechanisms that may need to be considered (**Figure 52.7**). One pitfall can be to focus only on considering collapse or deformation mechanisms for the foundation operating in isolation, solely due to the structural loads imposed by the superstructure. Additional collapse and deformation modes may exist due to the nature of the soil and groundwater regime; the site's location (e.g. if the site is adjacent to an unstable slope); the site's history (e.g. if it has been affected by mining or quarrying); or due to the nature of the regional hazards (e.g. earthquakes, flooding, etc.). In many cases detailed analysis is inappropriate, and eliminating the hazard (e.g. relocating foundations, stabilising the ground mass, preventing erosion, etc.) is the best approach. 'Global' ground movements across a site need to be considered. **Figure 52.7(g)** illustrates a common situation where a sloping site requires cut and fill earthworks to create a level platform in order to build the

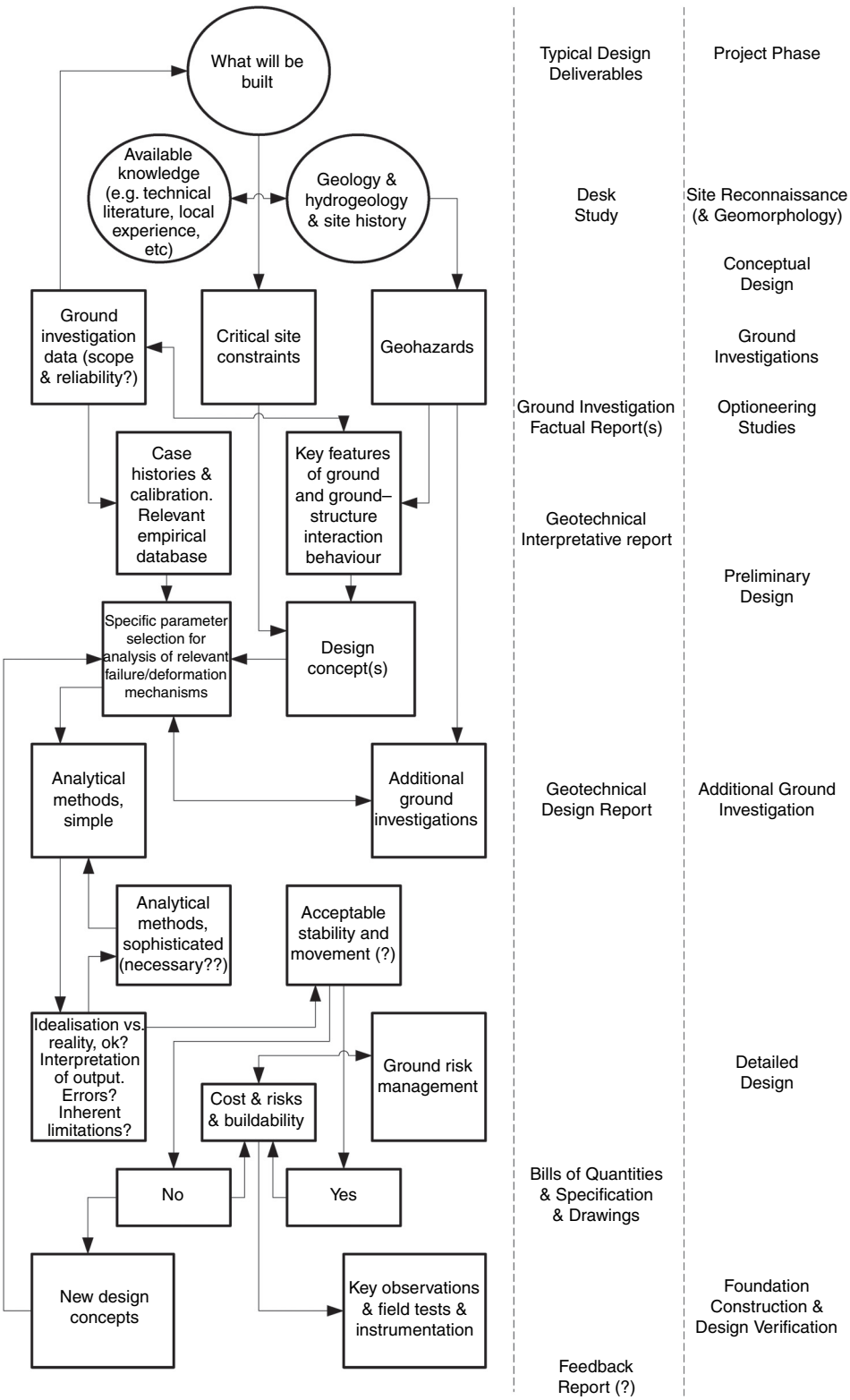
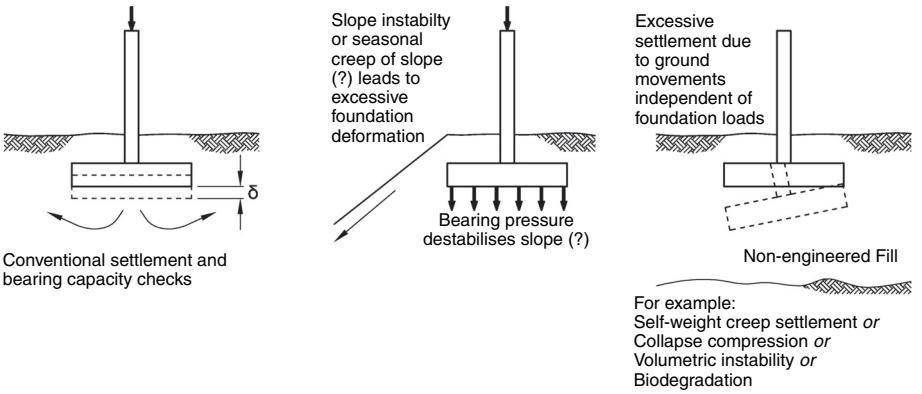


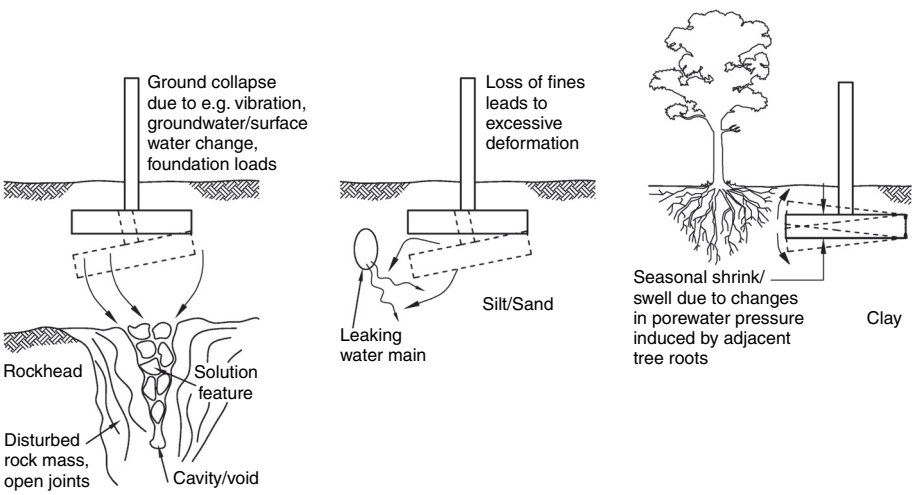
Figure 52.6 Information requirements and the foundation design process



(a) Acceptable stability and deformation under applied superstructure loads

(b) Potential interactions with adjacent slopes (or adjacent retaining walls, foundations, etc.)

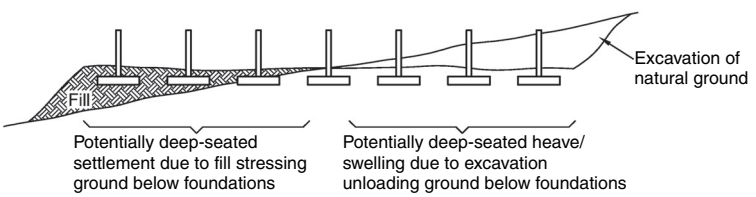
(c) Non-engineered Fill Excessive Deformation independent of foundation loads



(d) Excessive deformation due to void/cavity collapse in solution feature (or mineworkings)

(e) Seepage erosion and loss of fines, causing excessive deformation

(f) Seasonal shrink/swell due to trees



(g) 'Global' ground movements due to cut/fill to form level site

Note:
This is not intended to be a comprehensive summary of all potential deformation and collapse modes. Important to recognise that excessive deformation/collapse may occur due to a variety of causes, some of which are independent of applied foundation loading.

Figure 52.7 Examples of various deformation and collapse modes

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new structure. Strip foundations form the foundations for the new structure. In this situation, even if the earthworks are well engineered and properly constructed, significant global ground movements may develop since:

(i) Beneath the zone of fill there will be deep-seated increases in the vertical stress, due to the weight of the fill, in the underlying natural ground; this will cause settlement (in addition to any settlement due to the bearing pressure on the strip foundation).

(ii) Similar to (i), beneath the zone of excavation there will be deep-seated decreases in the vertical stress in the underlying natural ground, this will cause heave and swelling. This may be larger than any settlement induced by the foundation bearing pressure below the strip foundations, and may lead to a *net upward* movement of the foundations.

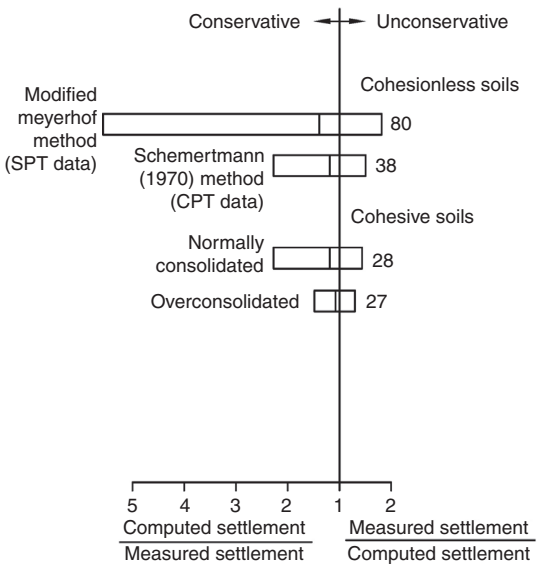


Figure 52.8 Reliability of deformation calculations. The figure gives a comparison between computed and measured settlements of spread footings. Each bar represents the 90% confidence interval (i.e. 90% of the settlement predictions will be within this range). The line in the middle of each bar represents the average prediction and the number to the right indicates the number of data points used to evaluate each method

Based on data from Burland and Burbridge (1984), Schmertmann (1970), Wahls (1985), Butler (1975)

Failure mode	Foundation type	Historic practice: typical overall factor of safety	Limit state codes, e.g. EC7: partial factor on soil strength		
Base sliding	Shallow	1.5 to 2.0	tan φ	c′	S_u
Bearing capacity	Shallow	2.5 to 3.5	1.25	1.25	1.4
			1.25	1.25	1.4
Overall stability	Shallow	1.3 to 1.5	1.25	1.25	1.4
Shaft resistance	Deep	1.5 to 2.5	Pile resistance		
			Shaft		Base
			Varies depending on pile type and test regime		
Base resistance	Deep	2.0 to 3.5			

Table 52.2 Typical factors of safety for foundation design

Site-specific factors	Typical issues
Intrinsic complexity of geology and hydrogeology	Soil profile homogeneous? Artesian or sub-artesian groundwater pressures? Challenging soil behaviour (e.g. sensitive 'quick' clays)
Onerous design criteria	Allowable settlement very low? Is horizontal displacement or rotation critical?
Consequences of failure	Potential loss of life? Major economic loss?
Nature of applied loads	Large and frequent cyclic loads? Occasional wind load? Or accidental impact load?
Availability of local experience and case history data	What has been published? What is known by local engineers from past experience?
Scope and reliability of ground investigation	Appropriate tests and locations investigated? Well supervised? Factual data checked?
Level of geotechnical risk	Ductile or brittle failure mechanism? Complexity of soil–structure interaction?
Construction uncertainties	Can design assumptions be verified? Level and quality of site supervision?
	Foundation behaviour intrinsically sensitive to workmanship?

Table 52.3 Some considerations for selecting a factor of safety

Any calculations of foundation movement would need to consider the stress changes induced by the cut and fill activities, as well as those induced immediately below the strip footings. Potentially the foundations underlain by the area of fill could suffer substantial settlement, and the foundations in the area of excavation could suffer substantial upward movement. Consequently significant differential movement could develop across the interface between the cut-and-fill areas. The magnitude of the movements depends upon: the compressibility and stiffness of the natural ground; the magnitude and areal extent of the cut and fill required; the plan dimensions of the shallow foundations and the magnitude of the foundation bearing pressure. If differential movements were excessive then consideration could be given to: movement joints in the structure; relocation of the structure so it was either solely in an area of fill or solely in a cut area; use of a raft instead of isolated pad or strip footings; or piling. If piles are chosen the pile design needs to consider the negative skin friction below the fill area and the heave-induced tension below the excavation areas. Potential

causes of movement, independent of foundation loads, associated with a range of different soils, rocks and fills are also discussed in Section 3 *Problematic soils and their issues* and Chapters 40 *The ground as a hazard* and 41 *Man-made hazards and obstructions* of the Manual.

Throughout the design life of the structure, foundation deformations need to be acceptably small and allowable deformation will usually be the governing design criterion. Historically, there has been little confidence in the reliability of deformation calculations. **Figure 52.8** provides a comparison between computed and measured settlements for several different foundations in a range of ground conditions. As shown, the actual settlement can be quite different to the predicted value. Hence, codes of practice used to recommend large factors of safety in order to limit foundation deformation. More recently, limit state codes of practice have been published (e.g. Eurocode, EC7) and typically their partial factors of safety against various collapse modes (e.g. ultimate limit state) are lower than those previously recommended, for example, see **Table 52.2**. **Table 52.3** summarises the site-specific issues that should be considered before selecting a factor of safety. Factors of safety given in codes are minimum values, and there may be a need for higher factors of safety if there are significant uncertainties. In favourable circumstances, the new limit state codes can facilitate more economic designs. However, when using limit state codes there is more onus on a designer to exercise judgement, particularly concerning the reliability of deformation calculations (serviceability limit state). For example, since the 1980s there has been a considerable improvement in understanding the stiffness characteristics of overconsolidated clays subject to monotonic increases in stress. In contrast, there remains significant uncertainty about the deformation behaviour of soils subject to cyclically varying loads.

The overriding requirement for assessing deformation behaviour is an understanding of past observations of foundation performance. To be of practical benefit, this experience needs to be based on a careful analysis of foundation performance of similar foundation types under comparable loading, within similar geology and hydrogeology. Case history data can then be used to calibrate the proposed calculation model, and for practical applications the model needs to be as simple as possible whilst still capturing the key features of the soil–foundation interaction. Great care is required if extrapolation is necessary beyond the existing database of knowledge (for example, due to the magnitude of the loading, scale or complexity of the foundations, etc.). In these circumstances, expert advice, special investigations, field trials, etc., are likely to be required. When utilising a database of past performance and an associated calculation model, it is vitally important to verify that the geology is relevant to the site under consideration. For example, Burland and Burbidge (1984) developed an empirical method for calculating the settlement of silica sands. It would be quite dangerous to use this method to calculate the settlement of sands with a different mineralogy, e.g. calcareous or micaceous sands.

52.3.2.2 Risk management

Risks, during and after foundation construction, need to be assessed and managed. There can be a wide range of different types of risk:

- health and safety;
- technical issues, e.g. ground, groundwater and foundation behaviour;
- performance of foundation constructor;
- environmental;
- commercial (costs and programme);
- design interfaces (communication).

Because of the broad range of issues that may need to be assessed, it can be beneficial for the project risks to be assessed formally via a risk workshop, attended by key members of the project team including the geotechnical and structural engineers. In addition, at an early stage of the project, it is helpful to carry out 'Peer Assist' (Powderham, 2010; SCOSS, 2009). Peer Assist involves one or two senior engineers carrying out a proactive independent review of the available project data (including desk-study and ground-investigation data), the project brief and constraints, and assessing the foundation engineering options identified by the project designers. A review of the risk register and challenging the assumptions and design methods used by the designers are important aspects of Peer Assist. Peer Assists are best carried out in a workshop environment with the project design team. The duration of Peer Assist can vary from half an hour for simple projects to several days for complex projects.

The geotechnical triangle is a valuable framework for ground risk management. The geotechnical triangle is introduced in Chapter 4 *The geotechnical triangle* and practical applications are discussed in Chapter 9 *Foundation design decisions*. All aspects of the triangle must be considered. If one or more elements of the triangle are forgotten or poorly implemented then foundation failures are much more likely. The identification of ground risks requires a deep understanding of ground behaviour (in the context of the site's history, geology and hydrogeology) and of the envisaged construction process.

Uncertainty is an intrinsic part of foundation engineering. Hence, there is a need to be aware of what could potentially go wrong and assessing the consequences. The most critical aspect of risk management is to identify risk mitigation measures that are practical to implement and verify on site. For the design to be robust the foundation's behaviour should be relatively insensitive to small changes in ground or groundwater conditions. Construction, particularly below ground, is seldom perfect. Therefore the design should be simple to build and allow for normal construction tolerances and workmanship. The foundation designer will usually need to make several assumptions, and during foundation construction the designer will need to ensure that these key assumptions can be readily verified. This verification process may involve tests, monitoring and observations.

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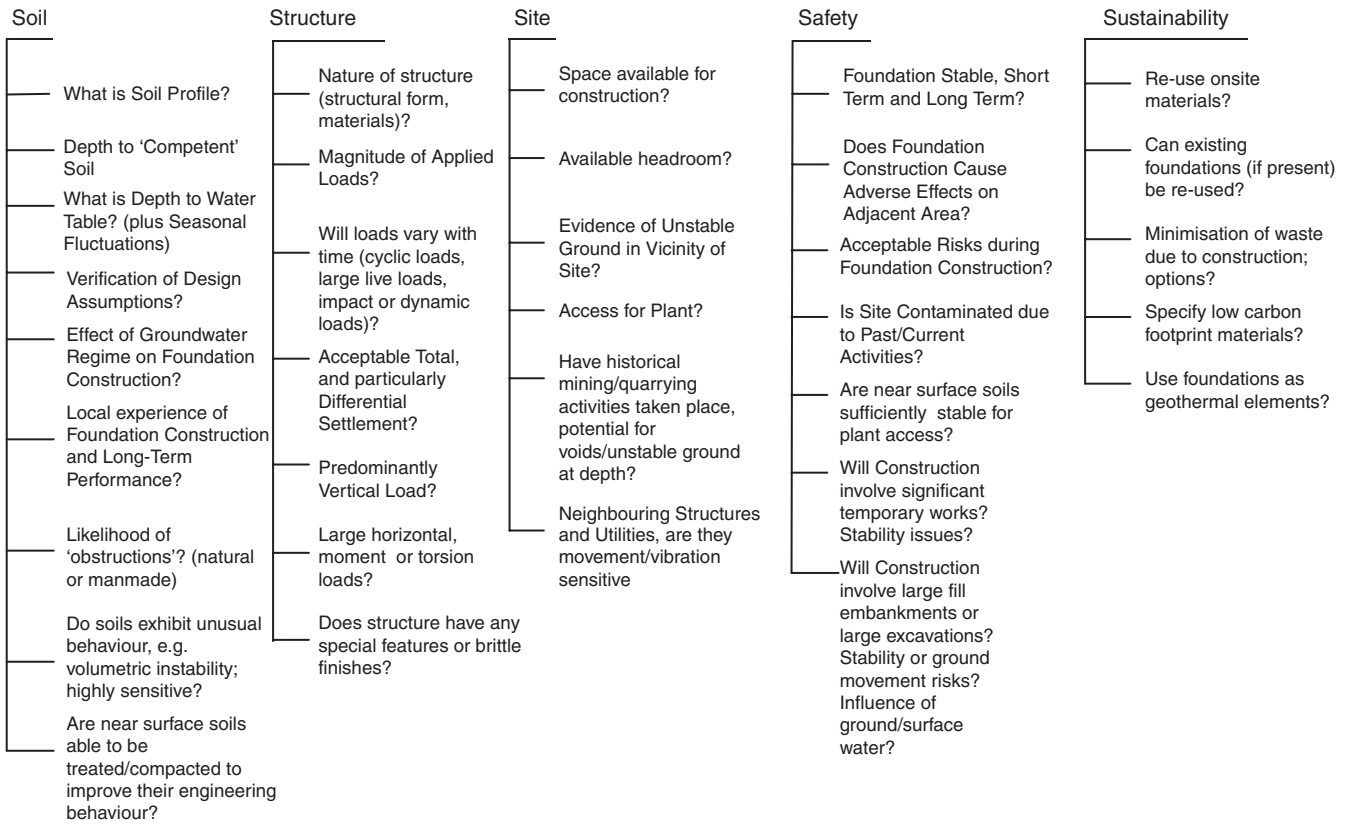


Table 52.4 Foundation selection, 'the five S's'

The verification process could vary from a simple inspection and observation of the exposed ground at foundation level, and verifying that the foundation will be placed on ground that is as strong and stiff as that assumed during design, through to complex instrumented load tests. These observations and tests are an important part of the foundation design process. If a design assumption cannot be readily verified and there are serious consequences if the assumption is not valid, then consideration should be given to changing the design to one which can be more easily verified. Foundation construction below the water table or in a maritime environment can be particularly challenging in this respect. For example, if soft soils are supposed to be dredged below water prior to foundation construction, how can the designer check that all the soft soils have been removed? If soft soils are left behind, instability may result.

52.3.2.3 Costs and programme

The preferred foundation solution will usually be the most economical to build, provided that the long-term performance and associated risks are deemed to be acceptable. Calculating the foundation costs may be based on assessing the quantities (e.g. volume of concrete for strip footing, or number, depth, etc. of piles) for the permanent works, and using unit rates for each of the main foundation elements (e.g. cost per m^3 for forming strip footing). A major problem with this approach is that it may not allow for:

(a) cost of temporary works, e.g. lowering the groundwater if excavations are required below the water table;
(b) cost of supporting excavations if movement sensitive structures or utilities are nearby;
(c) cost of removing obstructions if present, e.g. large boulders in glacial till or old foundations from previous buildings on the site;
(d) costs of treating or removing contaminated soils, if present;
(e) cost of verification tests (e.g. for ground improvement or piling this can be significant).

Hence, the normal methods used for assessing the costs of civil engineering construction may be quite unreliable when used for underground construction. To estimate foundation costs reliably it is important to take account of any necessary temporary works and to understand how the site's history and geology may impact upon foundation construction costs. Foundation construction is often on the critical path for a project. Hence, the preferred foundation solution will often be the one which is quickest to construct and has the least effect on other site activities, since overall project costs are often dominated by the overall duration for construction. 'Unforeseen' ground conditions are the most

common cause of project delays and cost increases (TRL, 1994). In particular, problems caused by groundwater are often the root cause of geotechnical problems on site. Therefore, reliable cost and programme estimates for foundation construction will be heavily dependent on carrying out good project risk management and upon the advice of the geotechnical engineer.

52.3.3 Foundation selection – the five S's

Table 52.4 provides a series of questions that a foundation engineer should ask him (or her) self, before developing a conceptual design. They can be remembered as the five S's: Soil, Structure, Site, Safety and Sustainability. Each will be briefly discussed in turn.

52.3.3.1 Soil (and groundwater)

Once the soil profile and groundwater regime are known, and the local geology and hydrogeology are understood, then the decision between shallow or deep foundations or ground improvement will usually be straightforward. Construction planning will also be heavily dependent on an understanding of the soil profile and groundwater regime. On the basis of geological age, it is often helpful to consider the following four soil types:

(a) *Geologically old, pre-glacial deposits* – These are usually heavily overconsolidated 'competent' foundation materials. When they are encountered close to the ground surface (say within 3 to 4 m) then shallow foundations can often be utilised.

(b) *Glacial deposits* – These soils are often extremely heterogeneous. Some glacial deposits, such as lodgement tills, can be stiff or very stiff clays or dense or very dense sands and be of very low compressibility. As for (a) above, if they are close to the ground surface then simple shallow foundations will often be appropriate. However, some glacial deposits (such as outwash or melt-out deposits) can be very soft and compressible. Changes can occur rapidly from one soil type to another (within lateral distances of 10 or 20 m or less). The interpretation of ground investigation data can be challenging, and input from experienced geologists will be important. Large strong boulders can often be present, which can make piling problematic.

(c) *Post-glacial deposits* – This class of material is seldom suitable as a founding material. These are recent (less than 10 000 years old) deposits and often comprise soft compressible clays or peats or very loose sands or silts. Geologically they are usually classified as normally consolidated. However, many of these deposits can be lightly overconsolidated in a soil mechanics sense (i.e. the pseudo pre-consolidation pressure or yield stress is higher than the current vertical effective stress, but foundation bearing pressures may exceed the pre-consolidation pressure). This may mean that shallow foundations could be feasible for lightly loaded buildings, especially if 'compensated' foundations are used (**Figure 52.1**). However, expert advice and special ground investigations (to verify the magnitude of the pre-consolidation pressure profile) are required. More commonly, for lightly loaded structures, ground improvement is utilised. For heavily loaded structures, piles are used to take the foundation loads to more competent strata at a depth below the post-glacial deposits.

(d) *Made ground and non-engineered fills* – Foundation construction is often carried out in areas that have been affected by previous human activities. These may have involved excavating the shallow natural soils and rocks and backfilling with any readily available materials, including domestic refuse and industrial waste materials. Unless the fill was properly engineered, using acceptable inert materials and well compacted, then it is unlikely to be a suitable founding stratum. Sometimes, lightly loaded foundations can be built on non-engineered fills, although special measures may be needed; foundation engineering in fills is discussed in more detail in Chapter 58 *Building on fills*.

For any of the above scenarios, and particularly for (a) and (b) above when shallow foundations may be used, it will be important to check if subsequent events (either due to man's activities or geological processes) have significantly affected the site, such as mining, quarrying or landslips.

Groundwater regime: it is always important to consider the groundwater regime. Its importance cannot be overstated, and is best summed up by the memorable statement from Terzaghi (1939):

> …in engineering practice difficulties with soils are almost exclusively due, not to the soils themselves, but to the water contained in their voids. On a planet without any water there would be no need for soil mechanics.

Despite its importance many ground investigations do not provide reliable data about a site's groundwater regime. No reliance should normally be placed on groundwater strike information during borehole drilling to assess if a particular soil layer is 'dry' or not. Reliable groundwater information requires careful installation of piezometers, and then monitoring the piezometers over a prolonged time period, preferably covering a period over the winter and spring, when groundwater pressures may be at a maximum. In geotechnical practice, the time period for piezometer monitoring is often inadequate. There is a high risk to any foundation engineering activity, if there is an inadequate understanding of a site's groundwater regime. In addition to piezometric data, local experience of groundwater problems should be considered and discussions held with an experienced hydrogeologist.

As noted by Day (2000): *'probably more failures in geotechnical and foundation engineering are either directly or indirectly related to groundwater and moisture migration than any other factor*.' Some simple examples in foundation engineering are given below:

(i) *Shallow foundations*: forming a shallow excavation above the water table will be relatively quick and cheap, using

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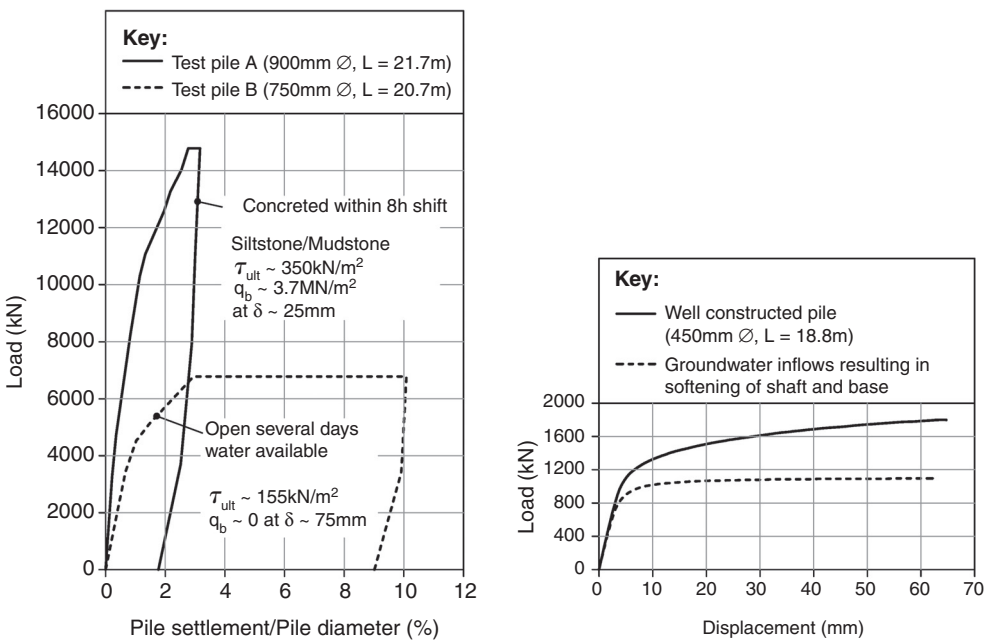
conventional plant and general site labour. However, in permeable soils, if the water table is above the base of the foundation level then the works will be more complex, expensive and time consuming. The groundwater will usually need to be excluded, by, for example, forming an impermeable cut-off wall, or by lowering the groundwater table by pumping from wells. Both these activities will often require specialist sub-contractors. Groundwater lowering may lead to ground settlement concerns or concerns with disposal of the pumped water (especially if it is contaminated). Installation of a cut-off wall by driven sheet piling may lead to concerns about noise and vibration. If the groundwater is not properly controlled, then the sides or base of the excavation may become unstable. The bearing capacity and long-term settlement performance would then be adversely affected.

(ii) *Deep foundations*: bored piles are commonly used in the UK. However, groundwater inflows and unbalanced groundwater pressures can seriously reduce the shaft friction and base resistance of a bored pile. If bored piles are to be drilled through cohesionless soils, weak fractured rocks, low plasticity silts or clays below the water table, then some combination of temporary casing, water balance or support fluid (such as polymer or bentonite) will usually be required. Pile test data are shown in **Figure 52.9**, which illustrates the dramatic reduction in pile capacity due to unbalanced groundwater pressures in weak rock and in an interbedded sequence of overconsolidated clays and sands. A common alternative to conventional bored piles are continuous flight auger (CFA) piles. The decision to use either conventional bored or CFA piles is usually dependent on the risks associated with groundwater.

52.3.3.2 Structure

The size and height of the structure, its structural form and the layout of structural supports (spacing of columns) and the type, magnitude and direction of the applied loads will have a major impact on the choice of foundations. For example, the modern trend for multi-storey building design is to create large internal spaces, which leads to relatively large column loads, and the consequential requirement for piled foundations. In addition to the magnitude and distribution of loads, the way in which the load varies over time is also important. The bulk of foundation design experience has been gained with structures that have live loads of less than 15% to 30% of the dead load and change infrequently. Foundation problems are often associated with tanks, silos and industrial units where the ratio of live to dead load is high and the live loads change frequently. The reliability of deformation predictions when cyclic loading is significant remains poor. These potential risks are exacerbated if large cyclic horizontal or moment loads are also applied.

One of the most important, but frequently overlooked, decisions is agreeing a sensible allowable settlement limit for the structure (with the structural engineer), based on a realistic appraisal of its purpose and vulnerability to damage. All too often over conservative and arbitrary limits are placed on total settlement, which then leads to unnecessary increases in foundation costs. The important topic of allowable foundation



(a) Weak rock

(b) Interbedded stiff clays and dense sands

Figure 52.9 Influence of groundwater on pile capacity for weak rock, stiff clay and dense sand

deformation is discussed in more detail in section 52.4. The main opportunity for a geotechnical engineer to design economic foundations, is to agree realistic limits for foundation deformation.

52.3.3.3 Site

In most industrialised countries, such as the UK, foundation construction (for either buildings or rail and road links) often takes place adjacent to existing structures. Old foundations may lie buried across the site. In addition, structures may be located on the site and have to be demolished, prior to the commencement of new foundation construction. Therefore, space for the construction of new foundations is often severely limited. This may mean that either there is inadequate space to create a large pad or strip footing, or there is limited headroom, which means certain piling or ground improvement equipment cannot be used.

A consequence of development in an urban environment is that there are often neighbouring structures or utilities, which may be sensitive to movement or vibration. If this is a concern it would preclude certain foundation types and methods of construction.

The site's history may mean that it is underlain by unstable ground due to past mining or quarrying activity; a thorough desk study (refer to Chapter 43 *Preliminary studies*) is essential. Because of the above considerations, it is usually important to visit the site and to understand the constraints that may affect foundation selection. When visiting the site it is also important to scrutinise neighbouring areas, since this may provide evidence of past problems due to unstable ground or long-term movements. Following completion of a desk study, the main environmental constraints (such as ecologically sensitive areas or areas with special environmental classifications) should be known. How these may affect foundation construction should also be considered.

52.3.3.4 Safety

Under CDM regulations (refer to HSE, 2007) a designer has a duty to formally assess the risks that may arise during construction. Given the uncertainties and the site-specific nature of ground hazards, the foundation designer needs to carefully review all the available information from:

- desk studies;
- ground investigations;
- site visits;
- local experience.

And consider this in the context of:

- planned construction activities at the site;
- condition of neighbouring land and structures, etc.

A particular concern when developing many sites across the UK (and other industrialised countries) is that past industrial activity has left a legacy of chemical or biological contamination. This contamination may have impacted the soil, groundwater and ground gas. In this case, foundation engineering will need to be coordinated and integrated into the strategy for minimising risks due to contaminated land. Often this will affect the choice of foundation type, depending on whether there will be large-scale removal and treatment of contaminated near-surface soils or if the site is to be capped off and the contaminated soils left *in situ*. For example, the use of a displacement piling system may be preferred to bored piles, so contaminated soils are not brought to the surface. Similarly, if ground improvement was being considered then a dry bottom-feed vibro system may be preferred to alternative wet vibro systems, which require subsequent treatment of the wet contaminated slurry.

52.3.3.5 Sustainability

Sustainability comprises three main elements:

- environment;
- social;
- economic.

Environmental impacts can be wide ranging varying from: the challenges and constraints associated with working adjacent to an environmentally sensitive area; the potential impact of contaminated land or groundwater; through to minimising the use of materials with a high embodied energy or carbon footprint.

Social impacts may include, for example, the effects of noise and vibration induced by pile driving in an urban area, or lorry movements associated with removing excavation arisings or spoil from bored piling, or spoil from a large excavation for a raft.

In highly sensitive locations, environmental and social issues may dictate the chosen foundation solution. In general, the objective is to minimise the environmental and social impacts of a project, whilst implementing it at a reasonable cost. Current themes in foundation engineering are:

(a) foundation re-use;
(b) the use of foundations as geothermal elements (refer to Adam and Markiewicz, 2009).

Foundation re-use: it may be possible to re-use existing foundations of, say, an old bridge or building, for the new superstructure. For many practical applications the old foundations are either supplemented by additional new foundations or are strengthened. Foundation re-use can apply to both shallow and deep foundations. Foundation re-use is reasonably common for transportation projects, such as railway bridge refurbishment, and is increasingly being considered for many building projects. CIRIA report C653 (2007) provides an introduction to the topic, and Butcher *et al.* (2006) provides more detailed background information. The current technical literature mainly focuses on the issues associated with re-using piles. Similar issues also apply to re-using shallow foundations, although physical investigations and inspections of shallow footings

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are often easier than for piled foundations. The key technical issues concerning foundation re-use are:

(i) Old foundations in the correct location? What is the plan location of old foundations relative to the new superstructure loads, e.g. a new building column grid may be offset from the old piles? Can new sub-structures, e.g. new beams or rafts, be built economically to transfer loads from the superstructure into the old foundations?

(ii) Load-carrying capacity and settlement characteristics: the ultimate geotechnical and structural capacity of the old foundations, and their likely movement under the new superstructure loads, needs to be assessed. Since the ground has consolidated (and strengthened) under the old foundation loads and will be subject to an unload-reload cycle (due to demolition of the old superstructure, and then reloading by the new superstructure), the settlement of old foundations would usually be relatively small until the old foundation load is exceeded. Once this load is exceeded much larger movements could develop. If, as is often the case, there is a mix of old and new foundations for the new superstructure then the different settlement characteristics of the different foundation types need to be carefully considered. Normally, the aim is to keep the net bearing pressures under the new superstructure to a smaller value than those under the old structure. Hence, an assessment of the loads that were applied by the old structure is fundamentally important.

(iii) Durability: over time the old foundations may have degraded, e.g. steel may have corroded or concrete may have been subject to sulphate attack. Assessing the residual design life of the old foundations can be the most complex part of foundation re-use assessment. Geotechnical engineers will usually need to get additional advice from materials specialists.

52.4 Allowable foundation movement

The magnitude of the allowable settlement is critical in determining the type and size of foundations that will be required, and, therefore, the foundation cost (see Chapter 19 *Settlement and stress distributions* for an introduction to methods of settlement prediction). It is important that the foundation engineer is involved in discussions with the structural engineer before the allowable settlement is decided, and challenges values that are unreasonably small. Reference to Chapter 5 *Structural and geotechnical modelling*, will be helpful in understanding the differences in approaches between structural and geotechnical engineers. As noted by Burland *et al.* (1977):

> Compared with the literature on the prediction of foundation movements, the influence of such movements on the function and serviceability of structures and buildings has received little attention. Yet major and costly decisions are frequently taken on the design of foundations purely on the basis of rather arbitrary limits on total and differential settlements.

Ideally, the foundation engineer would be able to predict the amount of differential settlement a structure can tolerate, and then predict the differential settlement that will actually occur due to the imposed structural loads and the response of the ground below the foundations. For most practical situations, it is impossible to make this prediction. As noted by Fang (2002):

> It is an extremely complex indeterminate analytical problem.

And he warns that:

> If an attempt is made to model analytically the structure and calculate the effect of differential settlements, one obtains ridiculously low allowable differential settlements because of the unrealistically large bending moments that will be calculated.

The factors that will affect real soil–foundation–superstructure interaction behaviour (Boone, 1996) include:

- variations in soil properties;
- variations in the construction sequence of the foundations and superstructure;
- uncertainties regarding the rigidity of connections across the foundation and within the superstructure;
- flexural and shear stiffness of the superstructure;
- changes in stiffness of the superstructure during construction and how load redistribution to the foundations will be affected;
- degree of movement (or 'slip') between the foundation and the ground;
- overall shape of the ground-movement profile and the location of the structure within it;
- building shape (both in plan and height);
- uncertainties in the way that the loads will redistribute in the long term as the structure settles differentially;
- variable influence of time, both in the rate of settlement of the ground, and how creep and yield within the structure will develop.

Because of the above complexities there is a need for simple guidelines based on previous experience. The assessment of allowable settlements and the development of criteria for routine limits on allowable settlements have been established empirically on the basis of observations of settlement and damage in actual structures. The paragraphs below summarise these observations and provide simple guidelines. It should be emphasised that these are 'routine limits' for conventional structures and are *not* intended to be rigid rules. They will not apply to all structures; Burland *et al.* (1977) and Powderham *et al.* (2004) describe case histories where the simple guidelines were *not* appropriate. This is an area of engineering where both geotechnical and structural engineers must work closely together.

Firstly it is important to distinguish between the following:

(1) Total Settlement – this may cause damage to services connecting into a structure, but will not lead to damage to the structure itself.

(2) Differential Settlement – due to rigid body rotation or tilt, which may be noticeable in high buildings and affect lifts or escalators, etc.

(3) Differential Settlement – due to relative displacements within the structure. It is this movement which can lead to structural damage. For example, there may be differential settlement between adjacent building columns or bridge piers.

In the literature there is a wide range of terminology, the most commonly used terms are described in Chapter 26 *Building response to ground movements*.

There can be confusion about the criteria that need to be satisfied when assessing appropriate settlement or movement limits (see Chapter 26 *Building response to ground movements* for a full discussion of building responses to ground movements). In general, three basic criteria need to be considered:

(1) Visual appearance – this includes deviation of walls, columns or floors from the vertical or horizontal, respectively, and cracking of walls or cladding. These matters are highly subjective and vary between different people and across different regions of the world. Burland *et al.* (1977) have given a five-point classification system for visible damage, which is widely used. It should be noted that the system is not intended to be a direct measure of the degree of damage and relates specifically to the ease of repair of the brickwork, masonry and plaster. The two lowest categories of degree of damage are very slight (crack widths less than 1 mm) and slight (crack widths less than 5 mm). At these damage categories the building would be remote from structural instability. However, crack widths of 3 to 5 mm would be unsightly. Walls, columns and floors which deviate from the vertical or horizontal by more than 1 in 250 would usually be noticeable, and judged to be unacceptable. Whether these movements affect the functionality or serviceability of the structure depends on the nature and configuration of the structure.

(2) Serviceability limit state (SLS) – a loss of serviceability typically means that there is:

- loss of weather tightness;
- loss of fire resistance;
- loss of thermal or sound resistance;
- connections to external pipework (water, gas, etc.) are damaged;
- lifts, cranes or internal machinery cannot operate properly;
- doors or windows sticking.

(3) Ultimate limit state (ULS) – this means that individual structural members may fail, or in an extreme case the whole structure may collapse. Structural distress is usually associated with excessive bending or shear stress in superstructure members. These limiting stresses are defined in the relevant structural design codes.

For the majority of buildings, the allowable settlement will be governed by (1) or (2) above, i.e. visual appearance or serviceability limits rather than by structural damage. For residential buildings (1) is likely to be most important, whereas for industrial buildings criteria (2) will be more relevant. For bridges, serviceability limit states for foundations are usually governed by avoiding SLS or ULS limits in the bridge deck and movement joints at piers and abutments.

52.4.1 Routine guides on limiting settlement

Published guidance on routine limits for allowable settlement consider sands separately to clay. The reason for this is that structures are more likely to suffer damage if the settlement occurs rapidly than if it develops slowly over many years. Hence, the settlement limits for sands are lower than for clays. **Figure 52.10(a)** plots maximum differential settlements against maximum settlement for buildings on raft foundations and **Figure 52.10(b)** provides a similar plot for frame buildings on isolated foundations (e.g. pads, strip, footings, etc.).

Most of the data has been taken from studies by Skempton and MacDonald (1956) and Grant *et al.* (1972). The data for buildings founded directly on clayey soils is shown separately to those founded on a stiff layer (such as dense sand or gravel) overlying the clay. In a few extreme cases, differential settlements (for both rafts and pads or strips) are nearly as large as the maximum settlements, and lie above the Bjerrum (1963) guidelines for the relationship between differential and maximum settlements. However, for the vast majority of situations the Bjerrum guidelines seem to be conservative. For rafts founded on overconsolidated London and Lambeth group clays, Morton and Au (1974) report differential settlements equal to about 25% of the maximum settlement. Isolated foundations, such as pad or strip footings, would normally be considered to be 'flexible' when considering the overall interaction of the foundation and superstructure. Although, as individual elements they may be assumed to be rigid when calculating settlement by routine methods (refer to Chapter 53 *Shallow foundations*).

The routine limits for the maximum total settlement for buildings are summarised in **Table 52.5.**

Foundations on sands seldom exhibit large settlements due to monotonic increases in stress; typically settlements will be less than 75 mm (Bjerrum, 1963; Terzaghi, 1956). Most problems with foundations on sand are due to vibration or other forms of cyclic loading, or due to groundwater seepage and erosion. In areas of the world that are prone to seismic activity, earthquake shaking can cause large-scale instability and highly damaging movements of foundations on sands (refer to Chapter 60 *Foundations subject to cyclic and dynamic loads*).

According to the data shown for frame buildings on isolated foundations, in **Figure 52.10(b)**, damage usually occurs when

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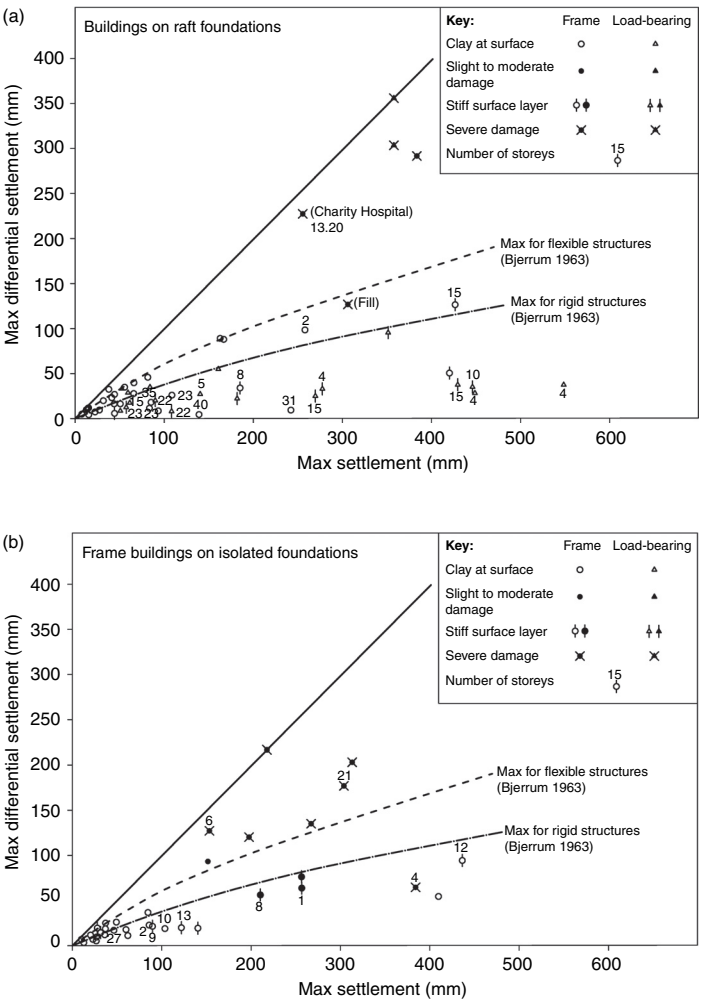


Figure 52.10 Observations of damage. Settlement and differential settlement. (a) Raft foundations; (b) isolated foundations, pad and strip footings
Data taken from Burland *et al*. (1977)

Foundation type	Soil type	Routine limits: maximum total settlement (mm) (Smax), note 6	Typical differential settlement
'Isolated', Pad, strip (note 1)	Clay	65	< 0.66 Smax, if Smax ≤50 mm < 0.5 Smax, if Smax >200 mm (note 2), (note 3)
Raft	Clay	100	< 0.33 Smax (note 4)
'Isolated', Pad, strip (note 1)	Sand	40	< 0.75 Smax (note 5)
Raft	Sand	65	

Notes:

1. Pad and strip footings are normally considered to be 'flexible' when assessing the overall interaction of the foundations and superstructure. As individual elements they may be rigid, depending on their thicknesses compared with their lengths or widths (ref Chapter 53).
2. These limits apply when the clay is immediately below the foundation. When a stiff layer is between the foundation and the clay, then the differential settlements can be substantially reduced and typically the differential settlement < 0.5 Smax. This stiff layer could be natural (say, due to a dense sand or gravel stratum) or be constructed as part of the foundation design.
3. Assume linear interpolation to assess differential settlement, when maximum settlements are between 50 mm and 200 mm.
4. This is usually conservative, as the relative raft bending stiffness increases (refer to Chapter 53 *Shallow foundations*), the differential settlements can be substantially reduced, to values of less than 0.15 Smax.
5. Differential settlements on sands can exceed 0.75 Smax (Bjerrum, 1963).
6. These routine limits are only intended for low-risk situations and simple structures, also refer to **Table 52.7**.

Table 52.5 Routine limits for the maximum total settlement of buildings

differential settlements exceed about 50 mm, or maximum settlements exceed about 150 mm. Based on an analysis of 69 case studies, Ricceri and Soranzo (1985) suggested a settlement limit of 80 mm to avoid structural damage. For buildings on rafts (**Figure 52.10(a)**) significantly larger settlements seem to be tolerable, with damage only being recorded for maximum settlements in excess of 250 mm. It is notable that some buildings founded on relatively rigid rafts have tolerated maximum settlements in excess of 400 mm, without suffering significant damage. For these structures, the raft rigidity has limited the differential settlement to about 10% of the maximum settlement. More recently Zhang and Ng (2005) have collated a large database of building damage. This database comprises about 300 buildings, of which about 100 buildings were in Hong Kong, and includes a variety of different building types (office blocks, warehouses, factories, hotels, hospitals, etc.), mainly on isolated foundations. Structural types include steel and reinforced-concrete frame and load-bearing wall structures. Underlying ground conditions include sand, clay and alluvium. **Figure 52.11(a)** shows histograms of the settlement for 95 buildings, for both shallow and deep foundations. They classified damage as either intolerable (where repair work is needed to the structure) or tolerable, and these categories are, therefore, different to the damage categories shown in **Figure 52.10**. Most buildings with settlements of less than 100 mm are undamaged and there were no reports of damage when the settlement is less than 50 mm. However, when the settlements exceed 200 mm intolerable damage becomes common; see **Figure 52.11(a)**. For angular distortion, Zhang and Ng report that values which are more severe than 1 in 400 are usually intolerable. Therefore, based on the case history data shown in **Figures 52.10** and **52.11(a)**, the routine limits for the maximum settlement given in **Table 52.5** will usually be conservative. This is particularly the case when raft foundations are used.

Moulton (1985) carried out a comprehensive study of tolerable movements of highway bridges in the United States of

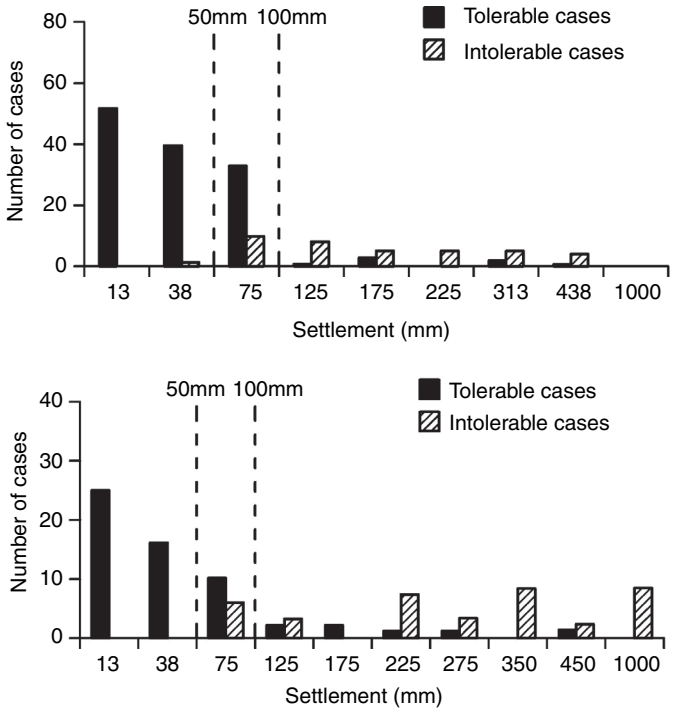


Figure 52.11 Observation of (a) building damage and settlement; (b) bridge damage and settlement
Reproduced from Zhang and Ng (2005)

America and Canada. The bridge types included steel, concrete and composite, and were continuous spans or simply supported. They were small to medium span (less than 50 m span) bridges and of simple structural form (e.g. cable stay and suspension bridges were not considered). The bridges were built on a range of soils (clays, sands) and rocks. Of the 580 bridges which were studied, 439 experienced some form of significant movement. The primary causes were excessive movements of approach embankments, inadequate lateral resistance and large foundation displacements. **Figure 52.11(b)** provides a histogram of settlement for 171 bridges. From this data it can be seen that intolerable damage is usually observed if settlements exceed about 75 mm to 100 mm. Zhang and Ng also report that if angular distortion exceeds 1 in 250 then intolerable damage is usually caused. Movements are usually tolerable if settlements are less than 50 mm or if angular distortions are less severe than 1 in 500. Barker *et al.* (1991) note that settlements are more damaging when accompanied by horizontal movements (and if the structure has inadequate horizontal strength).

Table 52.6 summarises routine limits for allowable movements for a range of structures and associated infrastructure such as drains, lifts, etc.

52.4.2 Site-specific assessments

It should be emphasised that the routine limits given in **Table 52.5** should only be used for low-risk site conditions and for simple structures. For more complex situations (some examples

Factor	Comments
Heterogenous ground conditions	Significant variations in soil type, layer thickness or competence of the soil layers below different parts of the foundation
Variations in loading	Different loads applied across foundation
Variations in construction duration	May induce increases in differential settlement, if the degree of consolidation of clay layers varies significantly
Variations in site history	Part of the site has been pre-loaded by old foundations, or part of the site has been excavated and the underlying clay has swelled and softened
Variations in site topography	If foundation is on side long ground, then different amounts of excavation may lead to substantial changes in net foundation bearing pressure
Changes in superstructure stiffness across the foundation	Changes in configuration of the structure, connection joint stiffnesses or cladding details, e.g. height, thickness or spacing of columns, walls, etc., could lead to local stress concentrations and increased sensitivity to differential ground movements.
Form of structure, potential for 'brittle' behaviour	Certain forms of structure, for example, short or thick columns and flexible floor slabs, are vulnerable to relatively small differential ground settlement, see, e.g. Burland *et al.* (1977)

Table 52.7 Factors that could increase the risk of damage due to differential ground movements

are outlined in **Table 52.7**) a more detailed consideration of differential ground movements and allowable limits for foundation design may be necessary. As noted earlier, realistic analysis of stresses within the structure due to differential ground movements are seldom practical or necessary. If a more detailed assessment is necessary, then consideration should be given, firstly to:

(i) Identify the component(s) of the structure that will be most sensitive to foundation movement, and when they will be built (or a likely construction sequence), and the relative weights of the main components of the structure.
(ii) The sequence of construction and likely immediate settlements or differential settlements that may arise due to the weight of each of the principal components of the structure, assuming the structure is flexible.
(iii) Each component of the structure will only be vulnerable to differential settlement and any resulting damage, which occurs *after* the component is built into the rest of the structure.
(iv) Following (ii) and (iii), assess the relative rigidity of the foundation or sub-structure and apply corrections in order to derive the appropriate differential settlements (refer to Chapter 53 *Shallow foundations*).

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Type of structure	Type of damage	Criterion	Routine limits	Comments
Framed buildings and reinforced load-bearing walls	ULS structural damage	Angular distortion	1 in 150 to 1 in 250	ULS concerns at these limits
Framed buildings and reinforced load-bearing walls	SLS cracking of walls, cladding, partitions	Angular distortion	1 in 300 to 1 in 500	Typically SLS concerns at these limits
Unreinforced masonry walls	Visual onset of cracking	Deflection ratio	Sagging 1 in 2500 (L/H = 1) 1 in 1250 (L/H = 5) Hogging 1 in 5000 (L/H = 1) 1 in 2500 (L/H = 5)	At these limits, there is only the onset of cracking; the damage is very slight. Tolerable movements are several times larger L= length of structure, H = height of structure
Steel, fluid storage tanks	SLS leakage	Angular distortion	1 in 300 to 1 in 500	
Utility connections	SLS	Maximum settlement	150 mm	Less for sensitive utilities, such as gas mains
Crane rails	SLS crane operation	Angular distortion	1 in 300	Depends on specific crane configuration
Floors, slabs	SLS drainage	Angular distortion	1 in 50 to 1 in 100	Depends on specific falls, alignment
Stacking of goods	ULS, collapse	Tilt	1 in 100	
Machinery	SLS, efficient operation	Angular distortion	1 in 300 to 1 in 5000	Depends on machine type and sensitivity. Bjerrum (1963) suggests 1 in 750 as 'typical'
Towers, tall buildings	Visual	Tilt	1 in 250	Tilts in excess of this will be noticeable and concerning, though possibly remote from collapse, depending on structure configuration. For the Leaning Tower of Pisa the tilt is 1 in 10
Towers, tall buildings	Lift and escalator operation	Tilt, after installation	1 in 1200 to 1 in 2000	Sequence of construction and timing of lift and escalator installation is important
Bridge	SLS	Angular distortion	1 in 250 to 1 in 500	Depends upon bridge deck characteristics and articulation arrangements
Bridge	SLS	Maximum settlement	60 mm	Typical values
Bridge	SLS, bearing	Horizontal displacement	40 mm	Typical values

Note:
Above limits based on: Burland and Wroth (1974); Day (2000); Barker *et al.* (1991); Boone (1996)

Table 52.6 Routine limits for allowable movements

(v) Compare the resulting differential settlements and angular distortion against the routine limits (given above) for the appropriate component of the structure.

Simple examples are given below for a frame building on a raft and a single-span bridge. The relevant settlements that need to be considered for a particular stage are those that are time dependent due to previous construction stages, together with those that are short term plus time dependent due to later construction stages. **Figure 52.12** shows a plot of settlement against time for different parts of a frame building. During excavation some heave occurs. The raft is constructed first and it is affected by subsequent differential settlements. As additional elements of the structure are built, short-term settlement occurs due to the incremental increases in weight of the structure. The parts of the structure that are built and tied together will experience differential settlement (and potentially be vulnerable to damage) due only to the settlements that occur after the element under consideration has been built. During construction the cladding will be added and internal components such as lifts and escalators will be built and connections to external utilities will be made. Finally the live load will be applied. Hence, the maximum settlements and differential settlements experienced by different parts of the building may be very different. The differential settlement of the raft will be larger than internal structural members, and much larger than for elements such as the cladding or lifts, which are usually the most sensitive to differential settlement. The risk of structural damage reduces as the ratio of live load to dead reduces, and the risk of visual or serviceability damage reduces if the construction of cladding and other sensitive components are delayed as much as possible.

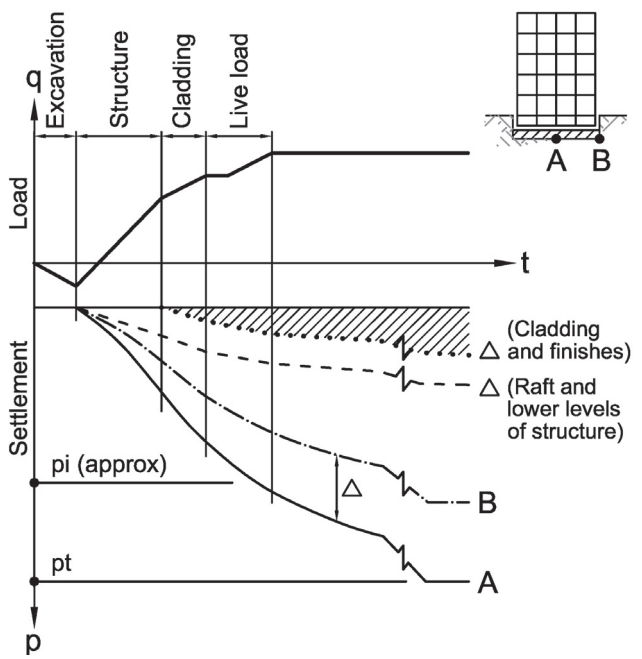


Figure 52.12 Frame building settlement vs. time

Figure 52.13 shows a plot of settlement and horizontal displacement against time for a single-span bridge. Most of the weight of the bridge and resulting settlement will be due to the abutments and their backfill. The settlement, and particularly the differential settlement experienced by the bridge deck, will usually be the main concern, and the residual differential settlement experienced by the bridge deck may be a small fraction of the maximum settlement of the bridge abutments. It is also apparent that the bridge-deck settlement and, the horizontal displacement of the bridge-deck bearings will be affected by the timing of backfilling to the abutments, relative to the bridge-deck construction. If the abutments are backfilled first, then the subsequent movements, which may affect the deck and bridge bearings, will be much smaller than if the deck is constructed before the abutment backfill.

52.5 Design bearing pressures

The superstructure loads calculated by a structural engineer will enable the *gross* foundation bearing pressures to be determined (see Chapter 21 *Bearing capacity theory* for an introduction to bearing capacity theory). However, it is the *net* foundation bearing pressure which should be used for both bearing capacity and settlement calculations (**Figure 52.14**), *not* the gross bearing pressure. The *net* pressure is that part of the gross applied pressure that requires the shear strength of the ground to support it. Hence, a factor of safety should always be considered in the context of *net* pressures only. There can be confusion about the terminology that is used for foundation bearing pressures, therefore, the following definitions are important:

q, *gross foundation pressure* – total applied load at founding level (includes the weights of the superstructure, sub-structure and foundation) divided by plan area of foundations

q_n, *net foundation pressure* – gross pressure minus the overburden pressure at founding level

q', *effective foundation pressure* – gross pressure minus the groundwater pressure

q_f *or* q_u, *ultimate bearing pressure* – value of bearing pressure at which the ground fails in shear

q_s, *maximum safe bearing pressure* – value of bearing pressure at which the risk of shear failure is acceptably low

q_a, *allowable bearing pressure* – value of bearing pressure at which the risk of excessive deformation and shear failure is acceptably low

q_w, *working pressure* – value of bearing pressure applied to foundations.

For limit state code calculations, q_s refers to the ultimate limit state and q_a to the serviceability limit state.

The application of transient loads should consider both the type of ground at founding level and whether ultimate or serviceability limit states are being considered. The total settlement of clays would usually consider the dead load plus a fraction of the live load (typically about 25%), since clay

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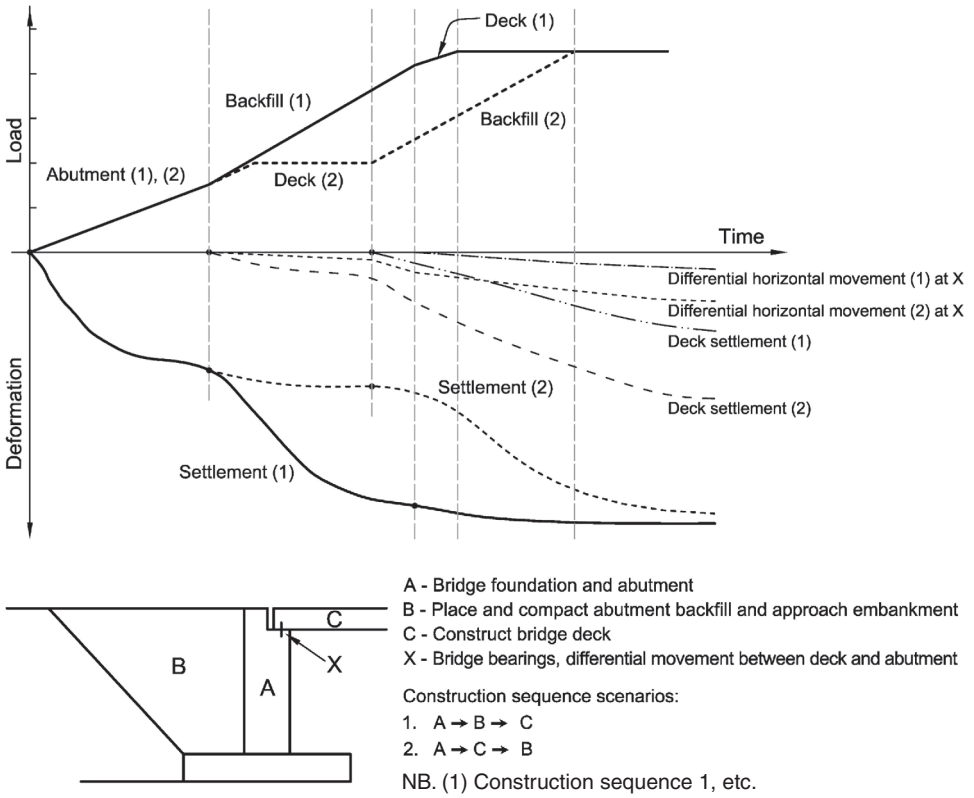
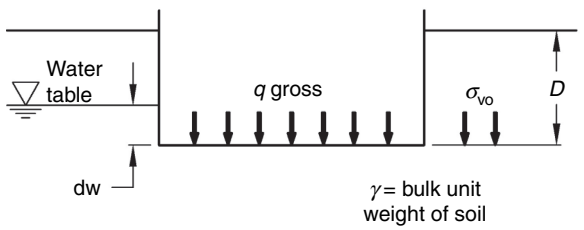


Figure 52.13 Bridge abutment movement vs. time



$q_{net} = q - \sigma_{vo}$, net total pressure
$q'_{net} = q' - \sigma'_{vo}$, net effective pressure
$\sigma_{vo} = \gamma D$, $\sigma'_{vo} = \gamma D - \gamma_w d_w$ (γw = bulk unit weight of water)
$q' = q' - \gamma_w d_w$ and
$q_{net} = q'_{net}$

Figure 52.14 Definitions of net and gross bearing pressure

consolidation will not be significantly affected by short-term transient loads (such as wind loads). In contrast, the stability of foundations on clays should consider both the dead and the full-live loads, and since the maximum bearing pressure will usually include short-term transient loads, the strength of clay should be based on undrained rather than drained (effective stress) parameters. For clays and other cohesive soils, undrained bearing capacity is usually more critical than drained. Although, if in doubt (especially for inclined loading conditions) both undrained and drained conditions should be checked. For cohesionless soils, it is the drained bearing capacity which is relevant.

It is important to clearly define the factor of safety that is being used. Different codes use different definitions and they are not necessarily equivalent. For example, for cohesionless soils (sands, gravels, etc.) two common definitions are:

(i) Factor of safety for strength (angle of friction, φ'),

$$F_{sd} = \frac{\tan \varphi'}{\tan \varphi'_{mob}} \tag{52.1}$$

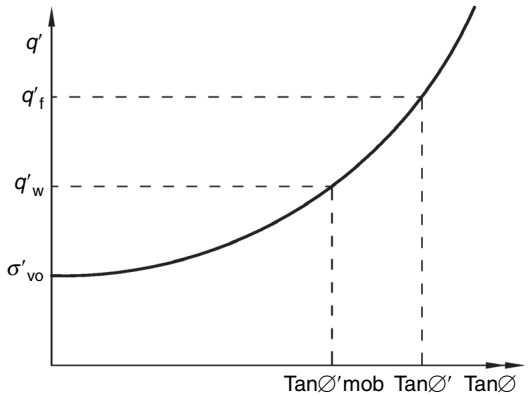
(ii) Factor of safety for drained bearing capacity, $F_{bd} = \dfrac{q'_f(net)}{q'_w(net)}$. (52.2)

From **Figure 52.15(a)** it is clear that F_{bd} does *not* equal F_{sd}, and that F_{bd} is much greater than F_{sd}.

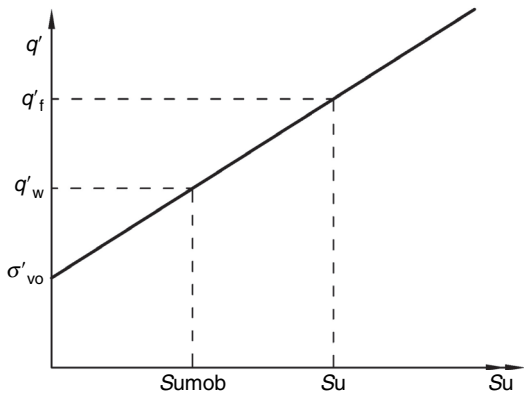
In contrast, for the undrained bearing capacity, $F_{bu} = F_{su}$ (refer to **Figure 52.15(b)**), where

$$F_{bu} = \frac{q_f(net)}{q_w(net)} \text{ and } F_{su} = \frac{S_u}{S_{umob}}$$

and S_{umob} is the mobilised undrained shear strength under q_w (net).



(a) Drained bearing capacity and mobilised friction angle



(b) Undrained bearing capacity and mobilised undrained strength

Figure 52.15 Bearing capacity and mobilised strength

52.6 Parameter selection – introductory comments

Poor parameter selection is a common pitfall in foundation engineering. Parameter selection can appear to be a black art, both for young geotechnical engineers and to general civil and structural engineers. Parameters such as soil strength and compressibility are significantly affected by a wide range of factors, including the testing methods, rate and direction of loading, the volume of ground tested, etc. Sampling disturbance affects the reliability of many laboratory tests and *in situ* tests can be difficult to interpret. Hence, parameters cannot be easily and accurately measured or used indiscriminately in a wide range of different calculations.

The ground investigation needs to, as a minimum, identify the following:

(i) Key soil characteristics, including: strength, compressibility, the presence of water-bearing layers (sand or silt), and the presence of weak layers.
(ii) Likely variability of properties: variation in soil strata thicknesses, relative density, strength, etc.

For routine ground investigations, soil strength can usually be more reliably established than soil compressibility or stiffness. Routine sampling often induces significant disturbance and, hence, variability in laboratory measurements of strength and stiffness. Therefore, variability in properties is usually best assessed via *in situ* test methods such as: static cone (CPT) profiling or via continuous sampling and split/describe/photograph and high-frequency classification tests (e.g. moisture contents, Atterberg limits, particle grading, etc.). Understanding the nature of the soil fabric is important in terms of assessing overall behaviour, e.g. in terms of rates of drainage or softening (Rowe, 1972).

In addition to ground investigation methods, the following factors will also need to be considered when selecting design parameters:

(i) the soil type, and its geological origins and subsequent history;
(ii) the nature of the chosen method of analysis;
(iii) the magnitude, direction and type of loading;
(iv) the scale and type of failure mechanism, and construction influences;
(v) the overall philosophy, for factor of safety selection.

Once all the factual data from a ground investigation has been assembled, data interpretation follows four main steps:

(i) Derive strength and compressibility from the factual information, usually directly from laboratory data, or indirectly via *in situ* tests (from theoretical analyses or empirical correlations).
(ii) Identify representative values for each stratum, which usually involves deleting excessively high or low values (due to sampling or testing errors or local 'hard' layers).
(iii) Cross-correlate different types of data to check that consistent and logical changes in strength and compressibility have been derived.
(iv) Once robust reliable data sets have been collated, design profiles can be determined; this may involve some statistical analysis (especially of index tests or penetration tests), but is largely dependent on experienced judgement.

Table 52.8 gives comments on some of the most common issues that affect soil strength and compressibility parameter selection. These basic parameters always need to be assessed. A clay's strength and compressibility characteristics are fundamentally linked to its:

- plasticity index;
- overall macrofabric (presence of silt, sand layers; presence, spacing and orientation of fissures);
- stress history, in particular its overconsolidation ratio.

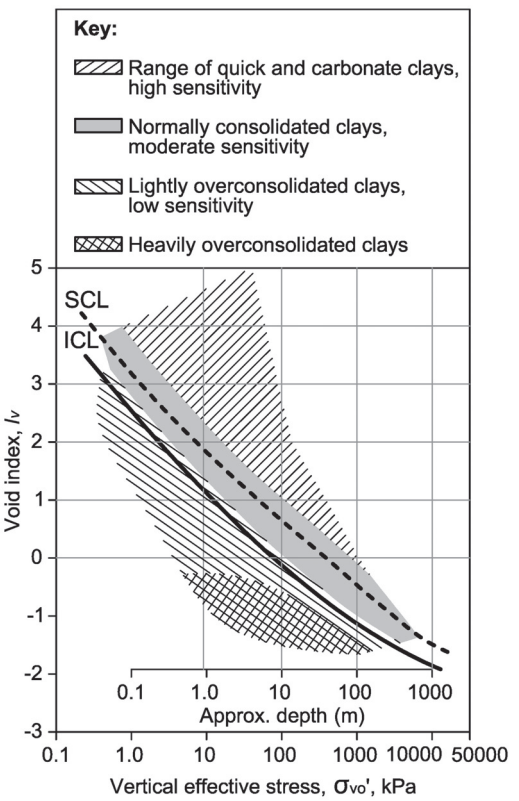
The liquidity index can be approximately correlated with a clay's overconsolidation ratio, e.g. near-surface clays (within

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Soil type, geological origins and subsequent history	Soil strength and stiffness are usually anisotropic due to bedding and stress history. If the yield stress is within the foundation bearing pressure then substantial changes in compressibility will occur following yield (**Figure 52.18(b)**). The ground shear (or Young's) modulus can degrade by an order of magnitude with strain amplitude (**Figure 52.18(c)**). Thin low strength layers may substantially affect the safe bearing pressure or overall stability.
Method of analysis	Ground investigation data typically provides simplified parameters: laboratory data are affected to some extent by disturbance and *in situ* tests are difficult to correlate to fundamental behaviour. It is essential to check the relevance and reliability of the analytical method for the soil type considered and the design situation. Parameter selection for all methods (even Cat 3 methods) depends to an extent on careful calibration of selected output against good case history data.
Magnitude, direction and type of loading	Deformation estimates are critically dependent on selection of stiffness and compressibility parameters (e.g. Young's modulus). If these are linear elastic, then the modulus must be appropriate to the stress–strain range and loading direction. The mobilised strength and deformation modulus under different loading regimes, say sustained cyclic loads, impact loads or long-term monotonic 'static' loads, is quite different.
Scale and type of failure mechanism, and construction influences	A cautious approach is needed if failure could occur within a small zone of the soil, e.g. beneath a narrow pad, strip or small diameter end bearing pile. A less conservative approach is appropriate if failure will inevitably involve shearing across a large mass of ground, e.g. where there is large raft or shaft resistance along a friction pile. Softening, loosening and remoulding of soil may occur during construction, especially if exposed to surface or ground water.
Magnitude of factor of safety and definition	Many codes used to adopt large factors of safety (2.5 to 3.5 for bearing capacity) in order to limit deformation and to allow for crude ground investigation methods. Some modern codes adopt lower factors of safety (1.25 to 1.4); these require more careful interpretation of strength and the designer may need to allow for factors, such as strain rate, anisotropy, etc.

Table 52.8 Factors which may influence the selection of soil strength and compressibility parameters for subsequent analysis

say 10 to 15 m depth) that have a liquidity index close to zero, or negative values, are likely to be heavily overconsolidated, whereas clays that have large positive values (say in excess of +0.3) are likely to be lightly overconsolidated or normally consolidated. Burland (1990) introduced a similar (but more robust) parameter, termed the void index. A plot of the void index versus the vertical effective stress (which can be readily derived from basic index tests), can provide a good indication of the clay's compressibility behaviour (**Figure 52.16**). Clays that plot well below the intrinsic compression line (ICL) are likely to be heavily overconsolidated and form competent founding strata. Clays that plot close to the ICL are likely to be lightly overconsolidated or normally consolidated and of moderate to low sensitivity (sensitivity is the ratio of peak to remoulded undrained shear strength). Clays which plot close to or above the sedimentation compression line (SCL) are also likely to be lightly overconsolidated or normally consolidated but will be of high sensitivity.



NB.
1. Void index, $I_v \sim 2I_L - 1.0$, I_L = liquidity index.
2. For clays plotting to right of ICL, yield stress (or p'c) approximately equal to $3S_u$, S_u is peak undrained strength in triaxial compression (high quality samples).
3. ICL = intrinsic compression line, SCL = sedimentation compression line

Figure 52.16 *In situ* stress states in terms of void index for different clay types
Data taken from Chandler *et al.* (2004)

Any sites underlain by clays that plot above the SCL are likely to be extremely challenging; and these clays have been associated with significant geotechnical problems. Conventional empirical methods are likely to be unreliable in these soils, and specialist advice and more sophisticated investigations and analyses are likely to be necessary. The selection of other parameters, such as permeability, coefficient of consolidation, coefficient of earth pressure, etc., tends to be more demanding. If required, then usually more sophisticated ground investigation methods and specialist engineering expertise are needed to select the appropriate values. Some common foundation design scenarios that lead to different strength or stiffness parameters being assumed for design are illustrated in **Figure 52.17**.

The different categories of analysis are outlined in **Table 52.9**, and it is worth remembering that even the more theoretically sound analytical methods (categories 2B or 3B) will depend on some empiricism (e.g. back analysis and calibration against case histories). Methods of analysis (and their associated input parameters) will commonly only be appropriate for a limited set of design situations and it is vital that the geologist or engineer understands when a particular analysis method (and associated parameters) is relevant to the problem under consideration.

Most foundation analyses involve Category 1 or 2A procedures. For simple deformation analyses, which usually assume linear elastic soil behaviour, the assessment of an elastic modulus, such as Young's modulus, is often required. Refer to Chapter 17 *Strength and deformation behaviour of soils* for a discussion of the properties of linear elastic porous materials. Research during the last 25 years has clarified many of the factors that affect the mobilised stiffness and compressibility of soils. The competent heavily overconsolidated soils, which are usually utilised as founding strata, can, for practical purposes, be characterised as nonlinear elastic. It is important to differentiate between different elastic moduli (shear, Young's, bulk) and whether secant or tangent moduli are being considered; see **Figure 52.18(a)**. Key features of nonlinear elastic behaviour are:

- **(i)** The stiffness at very small strains, less than 0.001% strain (characterised by small strain shear modulus, G_o) is practically the same for static and dynamic loading.
- **(ii)** G_o applies for both drained and undrained conditions.
- **(iii)** For overconsolidated soils G_o will be higher for horizontal loading than vertical loading.

G_o provides a useful reference value (or 'anchor point') for assessing the stiffness of overconsolidated soils. Appropriate test methods are described in Chapter 47 *Field geotechnical testing*.

Prior to the 1980s, there was considerable confusion concerning the mobilised stiffness of soils. It was only after the

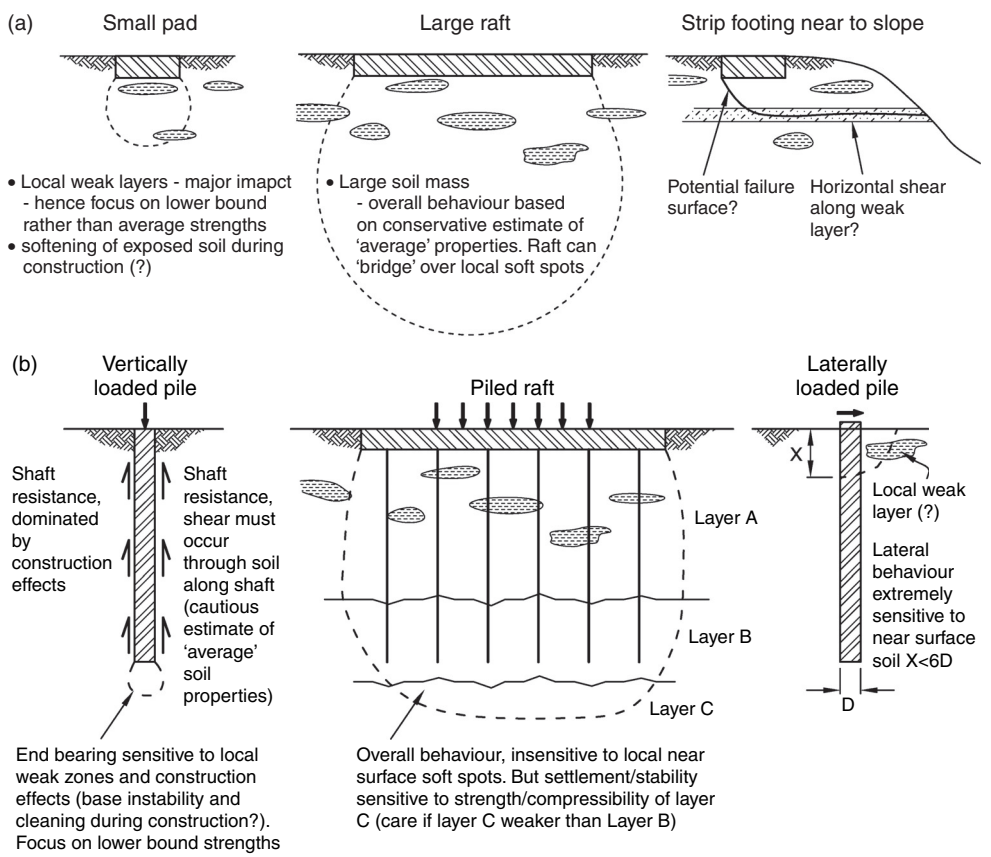


Figure 52.17 Parameter selection, general issues. (a) Shallow foundations, (b) deep foundations

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Category	Sub-division	Characteristics	Typical method of parameter selection	Level of complexity/ empiricism
1	A	Empirical (direct)	Simple *in situ*, e.g. penetration tests (SPT 'N' or CPT 'q_c'). Direct application of *in situ* test data	
	B	Empirical (indirect), plus some basic theory	*In situ* (as 1A) plus correlation to soil parameters (such as Young's modulus)	
2	A	Simplified theory, uses soil mechanics principles, e.g. bearing capacity (plasticity) or settlement (elasticity)	Routine *in situ* and laboratory tests, together with empirical correlations	
	B	As 2A, but theory allows for nonlinearity in a simplified manner	May use correlations with routine testing, more sophisticated testing or published data and back analysis	
3	A	'Basic' numerical modelling. Theory is isotropic, linear elastic or perfectly plastic. Allow for some interactions or topography variations	Rely on empirical correlations for soil stiffness and compressibility. Back analysis of case histories. Reliability of foundation movement predictions mainly dependent on selection of 'elastic' (?) moduli. Ground movements remote from foundation likely to be unreliable, potential for differential movements to be underpredicted	
	B	Nonlinearity or anisotropy allowed for in a simplified manner. More realistic assessment of interactions possible. Monotonic loading usually	Usually needs sophisticated test data (both *in situ* and laboratory), together with back analysis of case histories	
	C	Nonlinearity or anisotropy allowed for via proper constitutive models. Rarely used in current practice	Needs specialist tests (via research labs?) and advice for selecting input. The interpretation of the output is likely to be difficult	

Note.
Advanced methods of ground investigation and analysis tend to be more appropriate if there is local experience or the case history data is poor; there is a high level of geotechnical risk; the design objectives are complex; or there is a high potential for cost savings.

Table 52.9 Categories of analysis

importance of the strain amplitude on the mobilised elastic moduli was appreciated (Jardine *et al.*, 1984) that a rational basis for selecting elastic moduli could be developed. The assumption that the soil behaves as a nonlinear elastic material is reasonable provided that, following application of the foundation bearing pressure, the effective stresses in the ground are remote from the soil's pre-consolidation pressure (or yield stress) (refer to **Figure 52.18(b)**). The soil's 'elastic' modulus can degrade by more than an order of magnitude from G_o (or equivalent E_o values) with increasing strain amplitude (**Figure 52.18(c)**). Hence, the judgement required in selecting an appropriate modulus for linear elastic analyses is largely associated with understanding the likely strain amplitude induced by the foundation loads.

For sands many of the correlations between elastic moduli and penetration resistance are of extremely doubtful reliability. Some recent correlations between penetration resistance and G_o are more useful, see **Figure 52.19** and Box 52.1, although care is still required. Penetration resistance and the shear (or Young's) modulus are both affected by a wide range of factors, but the influence of these factors on the penetration resistance and the shear modulus differs to a great extent. Hence, the penetration resistance can only be a crude indicator of soil stiffness. Cross-checks between

Box 52.1. Empirical correlations for G_o in sands and clays

Note: It is recommended that G_o is assessed by more than one method; also refer to **Figures 52.19** and **52.20**. When appropriate G_o should also be measured by *in situ* and by laboratory tests.

Sands and silts

(a) SPT 'N' $G_o = a\ ('N')^b$

based on Clayton, 1995. For alluvial and glacial silts and sands:

a varies between 12 and 17, and

b varies between about 0.6 and 0.7.

Correlations in gravels are much less reliable.

Note: 'N' is the SPT blow count and the units for G_o are MN/m^2.

(b) *Mean effective stress:* $G_o = 220\ k\ (p'_o)^{0.5}$

based on Seed and Idriss (1970), where k is a function of the sand's relative density.

Relative density	*k*
Medium density	35 to 55
Dense	55 to 65
Very dense	65 to 75

Note: G_o, p'_o are in kN/m$_2$, p'_o is the *mean* effective stress.

Box 52.1. (Continued)

(c) *CPT* Average $G_o = q_c$ [1634 / (q_c / (σ'_{vo})$^{0.75}$], for uncemented quartz sands, after Rix and Stokoe, 1992.

Potential range of G_o = Average ± 0.5 (Average). Relatively high values for geologically old, dense sands and relatively low values for geologically young, loose sands.

Note: G_o, q_c and σ'_{vo} are in kN/m^2.

Clays

Based on Larsson and Muladic (1991). For medium to high plasticity clays:

G_o = 20,000/PI + 250

where PI is the plasticity index (%), having values between 25% and 50%.

Note: G_o is in MN/m^2.

different methods for assessing G_o for sands are strongly recommended. For clays, correlations between undrained shear strength and G_o have been developed and between G_o and the plasticity index, mean effective stress and overconsolidation ratio, see Box 52.1 and **Figure 52.20**, respectively. The latter relationship is applicable to a wider range of clays.

Poulos *et al.* (2001) provide some approximate relationships between modulus degradation and the factor of safety; these are shown in **Figures 52.20** and **52.21**. Based on these figures, the following can be concluded:

(1) The secant shear modulus decreases with a decreasing factor of safety.

(2) For a given factor of safety, the secant shear modulus for the shallow footing is lower than for a laterally loaded pile, which is lower than for a vertically loaded pile. This difference reflects differences in the induced strains around a particular type of foundation.

The relationships shown in **Figure 52.21** are intended mainly for preliminary assessment purposes and it should be emphasised that many simplifying approximations have been made to develop the plotted curves. However, these relationships emphasise the point that different elastic moduli should be anticipated for different foundation applications and the mobilised secant modulus will drop rapidly as the factor of safety reduces.

One of the practical implications of nonlinear elastic behaviour is that for most foundation engineering applications the mobilised 'elastic' modulus will exhibit an increase with depth, even when the soil strength is judged to be reasonably constant (Jardine *et al.*, 1986). For many practical applications, the soil strength will also increase with depth. Hence, if using linear elastic analyses, then the appropriate profile of the elastic modulus would usually increase rapidly with depth, as in **Figure 52.22**. For example, at the QE2 conference centre in London (Burland and Kalra, 1986), the selected profile for the elastic moduli for the London clay (based on a back analysis of several case histories across central London), for a raft of approximate plan dimensions 65 m by 75 m, exhibited a variation in the drained Young's modulus, for

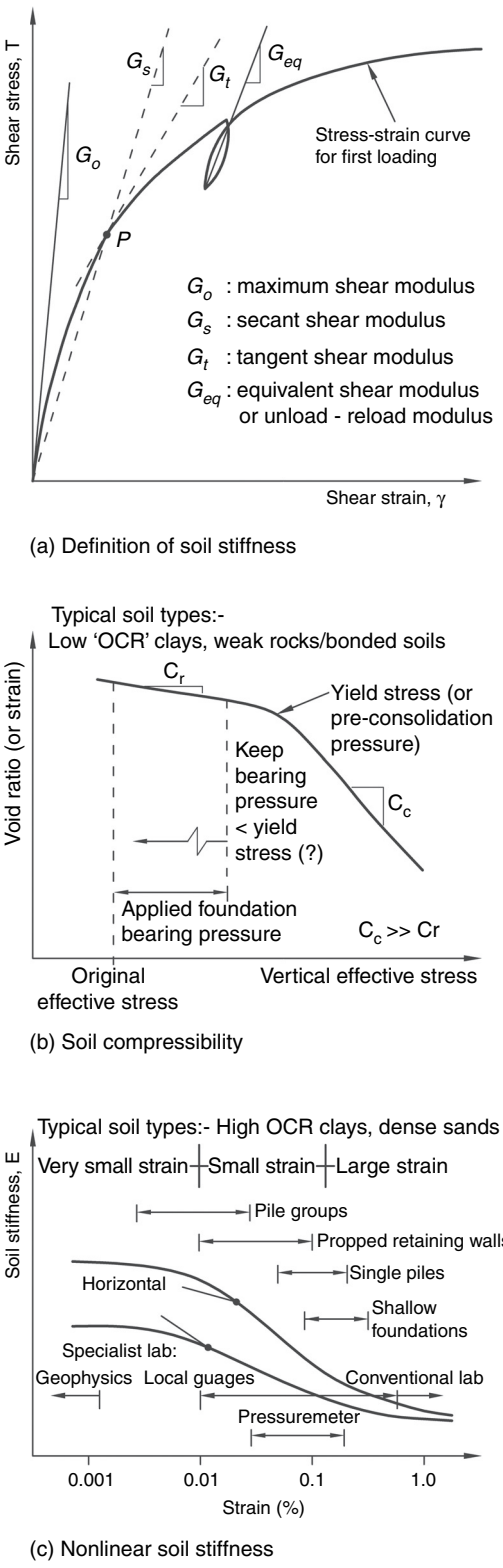


Figure 52.18 Ground stiffness and compressibility – fundamental considerations

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vertical loading, varying with depth from a value of about 100 times the clay's undrained shear strength, S_u, at the clay surface, to about 450 S_u at a depth of 20 m below the clay surface. These drained stiffnesses were, on average, several times higher than those derived from routine oedometer tests.

The assumption of linear elastic behaviour is usually acceptable when the behaviour of a foundation can be considered in 'isolation', for example, when calculating the bending moments in a raft or the overall settlement of a strip footing on stiff clay. Linear elasticity is usually inappropriate if assessments have to be made of interactions between, say, a new foundation and adjacent infrastructure, or when ground movements outside and beneath a loaded area become important, or when the foundation behaviour is intrinsically sensitive to nonlinearity, e.g. interactions between piles within a pile group or the interaction between foundations and adjacent retaining walls or earthworks (say new embankments or cuttings). For these situations nonlinear elasticity (or more complex stress–strain models) may need to be used, and it will also be important to have early expert advice and guidance.

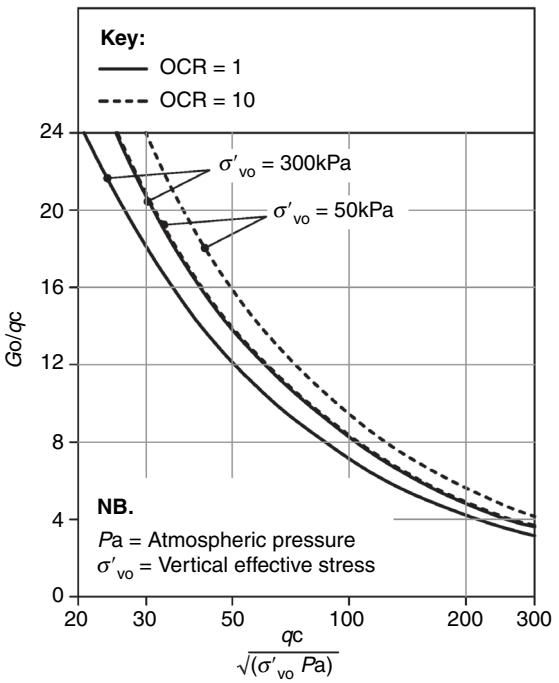
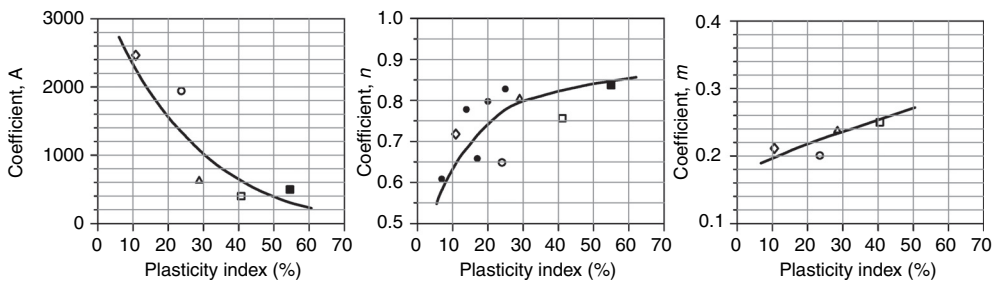


Figure 52.19 Correlation between G_o and CPTqc for sand
Reproduced from Baldi *et al.* (1989); all rights reserved

52.7 Foundation selection – a brief case history

A single-span bridge was required to carry heavy rail freight traffic across an existing road. The new bridge was to be built adjacent to an old rail bridge, which was in a poor condition. The original foundation design envisaged a conventional pile group (**Figure 52.23**), comprising 24 × 600 mm diameter bored piles (in a 6 × 4 group) about 6 m deep, socketed about 1 m deep into weak coal measures rocks. Because of the existing approach embankment, a temporary excavation was required to construct the pile cap. The excavation was to be supported by a sheet piled retaining wall. When work commenced on site, it was discovered that this solution could not be built due to a 132 kV electricity cable passing beneath the proposed pile cap. Numerous other services were also identified, and the owners of these services refused to allow sheet piles to be driven. The original foundation design had to be abandoned. A new foundation designer was appointed to develop a revised foundation solution. The revised foundation design was carried out under a sub-consultancy agreement to the original designers, who specified the foundation loads and allowable movements. A review by the new designers indicated that the available geotechnical data was inadequate: a report indicated that coal mining was 'suspected' in the area; however, a comprehensive desk study had not been carried out; four boreholes had



NB.
$\frac{G_o}{p'} = A(\frac{p'_o}{p'_r})^n(OCR)^m$; where Go is shear modulus at very small strain (parameters A, n and m are material parameters dependent on plasticity index; P'_o is the *in situ* mean effective stress = $(\sigma_v' + 2\sigma_h')/3$, σ_v' is vertical and σ_h' is horizontal effective stress; OCR is overconsolidation ratio. Go is most sensitive to variations in A and least sensitive to variations in OCR; P′r is a reference stress (equal to 1.0 kN/m^2)

Figure 52.20 Correlation between G_o plasticity index and the *in situ* stress state for clays
Data taken from Atkinson (2000)

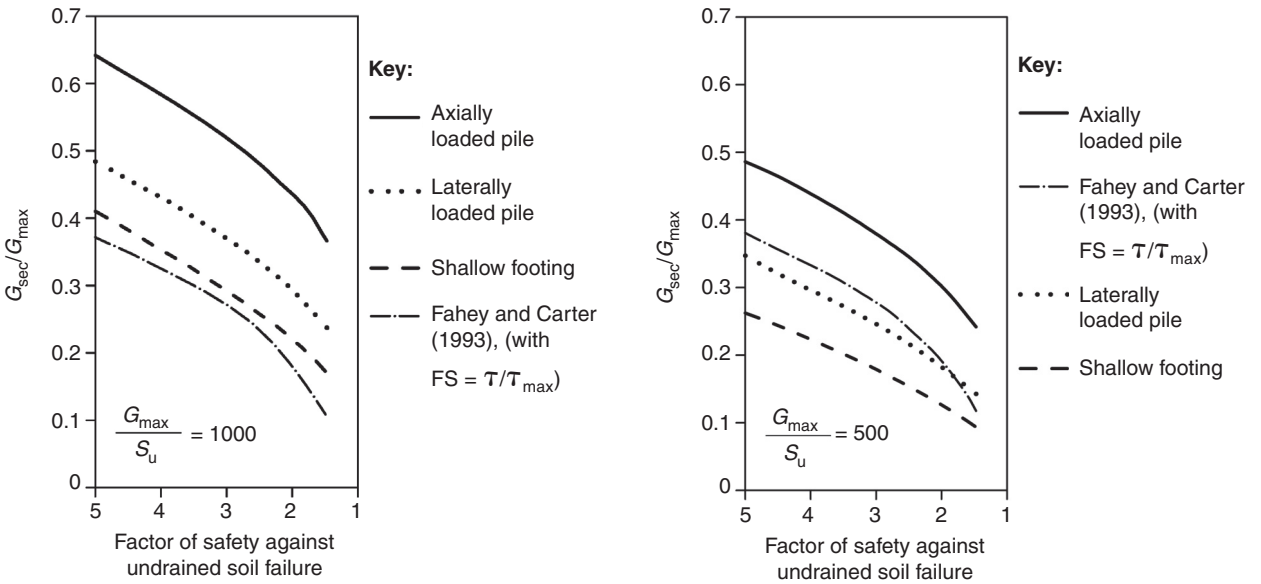


Figure 52.21 Mobilised secant shear modulus, various foundations on clay, for (a) $G_o / S_u = 1000$, (b) $G_o / S_u = 500$

Reproduced from Poulos *et al.* (2000); all rights reserved

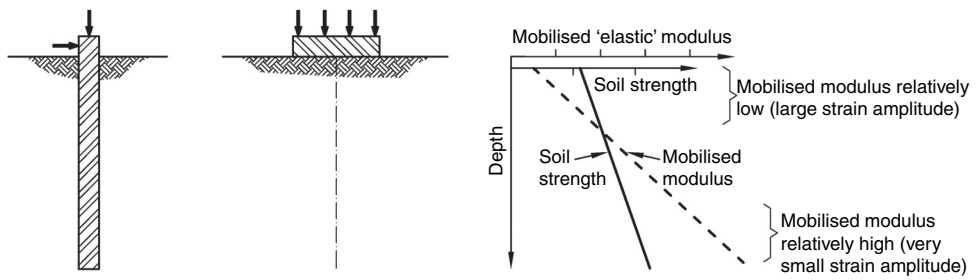


Figure 52.22 Mobilised 'elastic' modulus vs. depth, typical scenarios

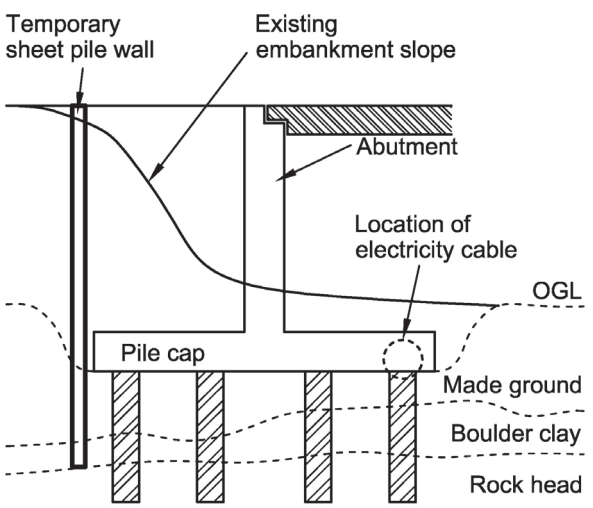


Figure 52.23 Foundation selection, case history. Original (unbuildable) foundation design

been drilled, but these were too shallow and only penetrated a couple of metres into the rockhead; data on rock strength, groundwater and ground contamination were lacking and the condition of the coal seams were unknown.

52.7.1 Ground conditions and site history

A desk study was carried out, which indicated that coal mining had been carried out at several levels below the site and that a chemical manufacturing works had been active adjacent to the site in the late 19th and early 20th centuries. Within the upper 35 m, three coal seams were identified and it was expected that some or all had been extensively worked, and that two shafts were identified adjacent to the site boundary. The subsequent investigation comprised four deep 100-mm-dia triple-tube rotary-cored boreholes (two 25 m deep, two 40 m deep) and down-hole geophysical logging was used to supplement physical logging of the condition of the rock mass by a highly experienced geologist. The subsequent ground investigations confirmed the presence of two worked coal seams in the upper

20 m (numerous voids and soft mining backfill materials were recorded), and unstable abandoned shafts. The third coal seam, at 35 m depth, appeared to have been unworked. The coal measures rocks were predominantly mudstone, with variable and relatively low strengths (fluctuating between a very stiff clay and a very weak rock) and were heavily fractured (with low Rock Quality Derignation, RQD, values) in the upper zones overlying the upper two coal seams. In one area there was evidence that the coal workings had collapsed causing significant disruption to the overlying rock mass. At greater depths, below the second coal seam, the mudstone strength increased markedly, becoming moderately weak to moderately strong, together with higher RQD values. A variable thickness of loose made ground (up to 3.5 m thick) and a discontinuous layer of firm to stiff glacial till up to 4 m thick were identified above the rockhead level. The made ground comprised industrial wastes and end-tipped clays and sands (probably from the old shaft excavations) and was chemically contaminated.

52.7.2 Site preparation

The site was in an unsafe and unstable condition. Therefore, before foundation construction could begin, it was essential to carry out preliminary works to infill and stabilise the old mine workings and remove heavily contaminated materials, which could harm the environment. Because of the scale of the works that were necessary (and the uncertain time period required to complete these works), this was let as a separate construction contract prior to the main foundation contract. The stabilisation works comprised mainly:

(i) infilling the worked seams and overlying disturbed rock mass with grout of varying viscosity;
(ii) probing and uncovering the shafts, infilling with mass concrete and constructing new reinforced concrete caps.

The deep grouting involved both vertical and inclined rotary open-hole drilling (inclined holes were needed, due to limited access, in order to stabilise the area below the foundation and approach embankment). Low-pressure grouting was then carried out. Grout take volumes varied from about 0.2 tonne per hole to a maximum of 29 tonne per hole (introduced during three separate grout phases). These variable grout takes reflected the disturbance of the rock mass arising from local collapses of the worked coal seams.

Quality control of the grouting, including permeability testing of the grouted areas and high-quality rotary coring to check the post-grouted condition of the coal seams, was essential to verify the effectiveness of the ground stabilisation.

52.7.3 Selected foundation type, design verification and construction control

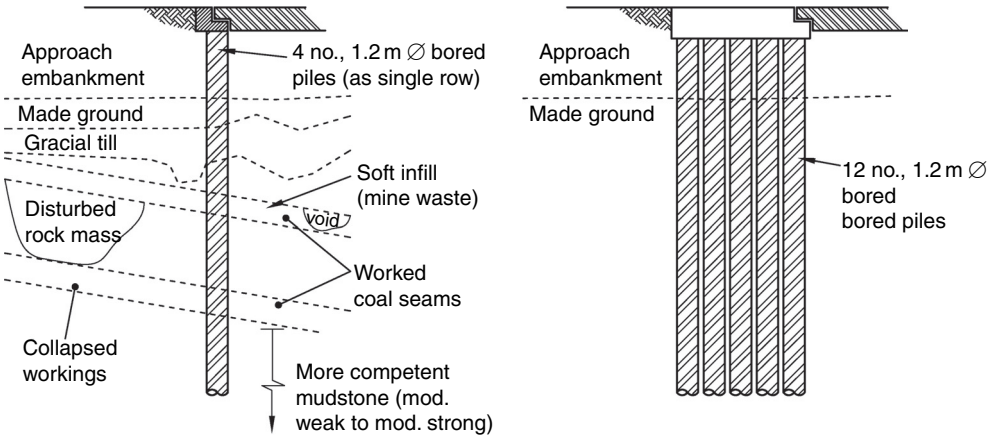
The foundation type was largely dictated by:

- the limited available space along the site boundary (and existing utilities, which could not be diverted);
- an expectation that large obstructions were likely to be encountered in the made ground (demolition rubble, old concrete foundations, etc.), and the need to drill through the moderately strong mudstone at depth;
- the need to minimise temporary works, in particular the avoidance of sheet piles.

The initial foundation solution was to construct at each abutment four 1200-mm-diameter bored piles 26 m deep (with the pile toe below the second worked seam, but well above the third unworked coal seam), immediately below the plan location of the new bridge deck bearings. These piles were formed as a load-bearing contiguous bored-pile retaining wall (**Figure 52.24(a)**). Unfortunately, the original designer insisted that changes to the bridge bearings would not be allowed and that these could not tolerate more than 6 mm of horizontal displacement. Despite several discussions and alternative bridge bearings (that were more tolerant of foundation movement) being proposed, this constraint was not removed. This allowable movement constraint had a significant impact on foundation costs, with the final design solution comprising twelve 1200-mm-diameter piles. The horizontal stiffness of the foundation was maximised by forming piled wing walls, U-shaped in plan, together with a 1.5-m-deep pile cap, to ensure pile head fixity (**Figures 52.24(b)**, **52.25**). The piling rig had to be sufficiently powerful to penetrate through obstructions. The contiguous bored-pile wall also acted as a retaining wall for the approach embankments.

The analyses used for design checks were relatively simple:

(i) Vertical capacity: the shaft capacity, in the disturbed ground above the first coal seam, was ignored. Below the first seam, the shaft capacity was limited to avoid overstressing the deeper worked coal seam, see (v) below. A maximum ultimate shaft resistance of 240 kN/m^2 was assumed in the competent mudstone below the second coal seam, and an ultimate shaft resistance of 70 kN/m^2 was assumed in the weathered mudstones and disturbed ground between the first and second coal seams. The allowable end bearing pressure was limited to 2000 kN/m^2 in order to minimise pile settlement. The overall factor of safety was 3.0 for the shaft and the base resistance (to allow for the complex geology, site history and residual uncertainties). The group capacity, based on the plan area of the 'U-shaped' block, was more critical than the sum of the individual pile capacities.
(ii) Vertical deformation: since limited end bearing pressures were assumed, and a high factor of safety was being used, the total settlement was expected to be less than 10 mm overall (allowing for group effects) and less than 5 mm after deck construction.
(iii) Horizontal capacity: this was based on the method of Broms, e.g. refer to Fleming *et al.* (1992).
(iv) Horizontal deformation: this was based on linear elastic pile group analyses, refer to Fleming *et al.* (1992).



(a) Revised Design (initial option)

(b) Revised Design (to achieve allowable horizontal movement of 6 mm)

Note. This would have achieved all design requirements, except alternative bearings needed to tolerate larger horizontal movements

Figure 52.24 Foundation selection, case history. Revised foundation design options

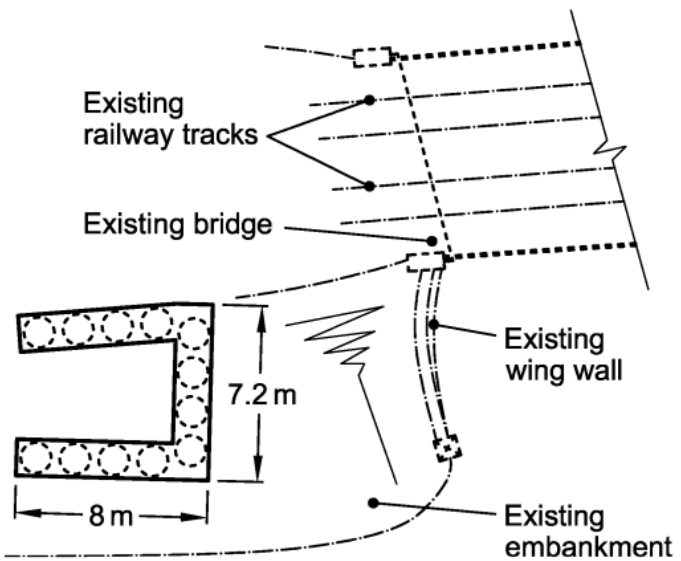


Figure 52.25 Foundation selection, case history. Revised foundation design, plan

(v) A maximum allowable bearing stress on the coal seams of less than 150 kN/m^2 was assumed. This was assessed by assuming an equivalent raft at two-thirds of the working shaft depth (below the first seam) and a 1 in 4 (horizontal to vertical) load spread above the 'raft' and 1 in 2 (horizontal to vertical) below.

A relatively small number of piles formed the bridge abutments; hence, it was not cost effective to carry out preliminary or working pile tests. It was more economical to use a large factor of safety rather than carry out relatively expensive load tests on large piles (if the piling contract comprised a much larger number of piles, then the cost–benefit balance would move towards carrying out preliminary tests and reducing the factor of safety).

The main construction controls were therefore:

(1) to log the pile bore arisings and verify that the depth was sufficient so as to be below the deepest worked coal seam, and that the piles were founded in competent mudstone at the specified depth;

(2) to carry out cross-hole sonic logging to check the pile concrete integrity was adequate.

52.7.4 Concluding remarks

This case history highlights the importance of:

(1) Careful collation of all relevant information for a site into a desk study report, including geology, historical activities (coal mining in this case) and existing site constraints (such as existing services). The original foundation design was unbuildable due to the presence of a 132 kV electricity cable. The presence of unstable mine workings beneath the proposed pile group had not been identified by the original designers.

(2) High-quality ground investigation data. The initial investigation had been poorly designed and supervised, the boreholes were too short, and important tests to identify the mechanical and chemical characteristics of the soils and rocks had not been scheduled. During the second ground investigation, high-quality logging of the rock mass

characteristics by an experienced geologist was profoundly important in deciding upon the appropriate toe level for the piles.

(3) Risk management. The site was in an unsafe condition and the mine workings had to be stabilised, by grouting, prior to bringing large (100 tonne) piling rigs onto the site. There were several intrinsic uncertainties associated with the grouting works (timescale, grout volumes, etc.), hence, from a commercial perspective, it was best to implement these works as a preliminary contract, prior to the main bridge-construction contract.

(4) Allowable movement criterion. The onerous limit on the allowable horizontal movement meant that the final design had three times the number of piles than an alternative option (which would have been technically viable if the movement limit had been 15 mm rather than 6 mm).

52.8 Overall conclusions

- Foundation engineering requires a broad range of knowledge and skills in construction methods; ground–structure interaction; geology; soil and rock mechanics; and understanding relevant case histories. The foundation designer must be prepared to ask questions and discuss these with other specialists. Good foundation design is usually the result of a multi-disciplinary team effort.
- The main types of foundations are: shallow (pad, strip, raft) and deep foundations (piles, barettes, caissons). Hybrid foundations include aspects of both shallow and deep foundation designs, examples include deep ground improvement and piled rafts.
- Design requires a consideration of: foundation stability and deformation; risk management; costs and programme for construction. The 'geotechnical triangle' is a valuable framework for ground risk management; all aspects of the triangle must be considered. Peer Assist carried out at an early stage can provide significant benefits.
- Foundation selection requires a consideration of the Five S's: **S**oil or rock types (and groundwater); **S**tructure type, including loads, allowable movements; **S**ite, including available space, headroom, neighbouring structures and sensitive environmental areas; **S**afety, in particular the site history may have left the site in an unstable or chemically contaminated state; **S**ustainability, there are often major opportunities when designing foundations, including the re-use of excavated materials, foundation re-use and the use of the ground as an energy source.
- Major and costly decisions are often taken on foundation design, on the basis of arbitrary and overconservative limits on total and differential settlements. Routine limits for total settlement vary between 40 mm and 100 mm depending upon the soil and foundation type. The actual risk of damage is dependent on differential movements and the construction sequence. The timing of the installation of sensitive components, such as architectural cladding, internal machinery such as lifts and escalators, etc., is particularly important.
- Selection of parameters, such as strength and deformation modulus, is challenging, because of the wide range of factors that may influence the value which is mobilised in a particular design, and the inherent limitations of analytical methods to model the complexity of real-ground behaviour. Routine investigation methods induce significant disturbance to the ground, which exacerbates the difficulties associated with interpretation of factual data. Ground investigations should include both *in situ* and laboratory testing. Continuous profiling of soil properties, either by methods such as CPTs or continuous sampling and detailed logging, is vitally important to understand the soil fabric.
- In the last 25 years or so, there have been major improvements in understanding the deformation characteristics of soils. Stress–strain behaviour is highly nonlinear, and the stiffness at very small strain (often known as G_o or G_{max}) is a useful reference or 'anchor' point. Stiffness degrades rapidly with increasing strain amplitude, so that at large strains it is often more than an order of magnitude smaller. Conventional tests, such as oedometer tests, measure stiffness (or compressibility) at large strains. Different foundation types (or different applied loads) induce different strain amplitudes in the ground, hence the *mobilised* deformation modulus can vary significantly (even for the same soil type). Commonly the *mobilised* deformation modulus will increase rapidly with depth below the ground surface.

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It is recommended this chapter is read in conjunction with

- Chapter 9 *Foundation design decisions*
- Chapter 19 *Settlement and stress distributions*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

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Chapter 53
Shallow foundations

Anthony S. O'Brien Mott MacDonald, Croydon, UK
Imran Farooq Mott MacDonald, Croydon, UK

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The design of shallow foundations is exceptionally wide ranging, and can vary from simple strip footings for a lightly loaded domestic property to the design of a large raft for a nuclear power station. The likelihood of geohazards affecting a site must always be assessed, irrespective of the scale of the project. This is best assessed by a comprehensive desk study and a site walkover by an appropriately experienced specialist. The prime requirement for successful design and construction is an understanding of the site history and the ground and groundwater conditions across the site. Sophisticated analysis will not compensate for a lack of knowledge of these issues. Prior to detailed analysis it is worthwhile considering the likely behaviour of the whole structure (which may comprise multiple closely spaced strip or pad footings) in the context of the overall geology, adjacent structures and the site topography. The potential risk of adverse interactions within, beneath and around the proposed structure should be considered. Foundation settlement will often be a more critical consideration than bearing capacity. Simple methods of analysis are usually adequate; the biggest challenge is to make reliable estimates of the compressibility of the ground. The bearing capacity and settlement of rocks will be sensitive to the orientation, spacing and characteristics of discontinuities in the rock mass, the degree of weathering and voids (either natural or man-made).

53.1 Introduction

The geotechnical design of shallow foundations can vary from very simple footings for small lightly loaded domestic buildings to the complex requirements of a raft for a nuclear power station. **Table 53.1** poses some questions that should help the foundation designer identify the most appropriate level of sophistication for the ground investigation(s), analyses and subsequent supervision and monitoring. The table is based upon the geotechnical categories proposed in Eurocode 7. For Category 1 structures, site-specific calculations are often unnecessary, and the design can be based on 'presumed bearing pressure values' in local codes or past experience. If calculations are deemed necessary, then only the most basic methods would be required. The geotechnical category should be assessed at the start of the project, and may be revised as more information becomes available. The intent of the process is to indicate the level of effort required during design and construction, together with the level of expertise of the geotechnical engineers required for the project. It is particularly important to note that the geotechnical category is not solely related to the size and complexity of the superstructure. For example, a simple structure may be Category 3, if the site geology is particularly complex. For shallow-foundation design, many textbooks focus on the analysis of bearing capacity and settlement; however, prior to these design checks the overall site geology, site history and associated hazards must be carefully considered. As discussed in Chapter 52 *Foundation types and conceptual design principles*, the five S's should be assessed for the site before deciding that shallow foundations are the most appropriate foundation type.

In order to facilitate these decisions a comprehensive desk study should be produced for the site under consideration. This will then provide a coherent framework for planning ground investigations, and in some cases (if the geology is well known) allow a preliminary design to be carried out.

This chapter firstly considers the causes of foundation movement (which can be independent of applied loads from the superstructure), provides an introduction to some of the construction processes that the designer would need to consider, and then gives an overview of foundation layout issues and potential interaction effects. A summary of bearing-capacity and settlement analysis methods is given in sections 53.5 and 53.6 of this chapter for foundations in clay, sand and rock. Finally the key topic of parameter selection is discussed in section 53.7.

Because of space constraints, the behaviour of soft clays (and associated analytical methods and parameters) is not discussed in detail. Normally, they would not be considered to be appropriate founding strata. However, if soft clays are present within the zone of influence of the foundations then great care will be required (refer to Chapters 17 *Strength and deformation* and 19 *Settlement and stress distributions*), and expert advice should be sought.

53.2 Causes of foundation movements

It is important to recognise that foundations may settle excessively due to a variety of causes that are independent of the applied bearing pressures from the superstructure. **Table 53.2** summarises these geohazards in the context of shallow foundations. Reference should also be made to Chapter 52 *Foundation types and conceptual design principles* (section

Questions	Category 1, Simple	Category 2, Intermediate	Category 3, Complex
1	Is structure small and simple?	If NO to all Category 1 and 3 questions.	Is structure very large or unusual?
2	Site geology well known and understood?		Is site geology complex or poorly understood?
3	Are ground conditions known from comparable experience to be sufficiently straightforward that routine methods may be used for investigations, design and construction?		Does it involve unusual or exceptionally difficult ground?
4	If excavation below water table is required, does comparable experience indicate that it will be straightforward?		Does it involve abnormal risks?
5	Is site free from abnormal risks, such as onerous differential settlement criteria, unusual or significant cyclic loading, or 'global' hazards such as earthquakes?		Can significant economies or reductions in construction programme be achieved by utilising Category 3 methods?
6	If yes to all above then, Category 1		Can improvements in safety or sustainability be achieved by utilising Category 3 methods?
7			Are the differential settlement criteria onerous (refer to Chapter 52 *Foundation types and conceptual design principles*)?
8			Are the loading conditions unusual or exceptional (e.g. high ratio of live to dead load, frequent large cyclic loads, large lateral loads)?
9			Is site subject to significant global hazards, such as earthquakes?
Likely investigation requirements	Desk study, plus a few trial pits, penetration tests and index tests to confirm site geology and ground conditions at founding levels	Desk study plus full range of conventional investigation methods. Sufficient testing to characterise ground properties below foundations (particular attention to zone within two-thirds of foundation width), and assess construction methods, and project risks	Desk study, plus comprehensive suite of ground investigations, including: highest quality sampling techniques; advanced ***in situ*** and laboratory testing; may be a need for large-scale instrumented load tests
Likely analysis methods	If deemed necessary: Clays – simple bearing-capacity checks plus high FoS (3.0) to ensure low settlement (refer to section 53.5) Sands + Rocks – simple empirical checks to verify low settlement (for sands, refer to section 53.6)	Analytical methods, outlined in sections 53.5 and 53.6	Nonlinear numerical modelling or more advanced analytical methods for bearing-capacity and settlement assessment
Likely construction supervision requirements	Visual inspection of ground at foundation level	Specialist supervision during key phases, local probing and testing may be needed to verify design assumptions; monitoring of, say, groundwater may be needed to control key project risks	Full-time specialist supervision plus subsurface instrumentation to monitor ground response

Table 53.1 Shallow foundations: likely design issues and requirements related to geotechnical categories

52.2, **Figure 52.8**) and Chapter 9 *Foundation design decisions* (**Table 9.1**). The global movements caused by these geohazards can be extremely large (some can be in excess of a metre, and are often in excess of 100 mm) and can lead to ultimate or serviceability limit state failures. Identification of these hazards is critically important and is usually best achieved by a combination of a desk study (to understand the site's geology and history) and a site walkover by suitably experienced specialists. If the hazard is present it is usually best to eliminate the hazard directly, or to relocate the foundations if feasible.

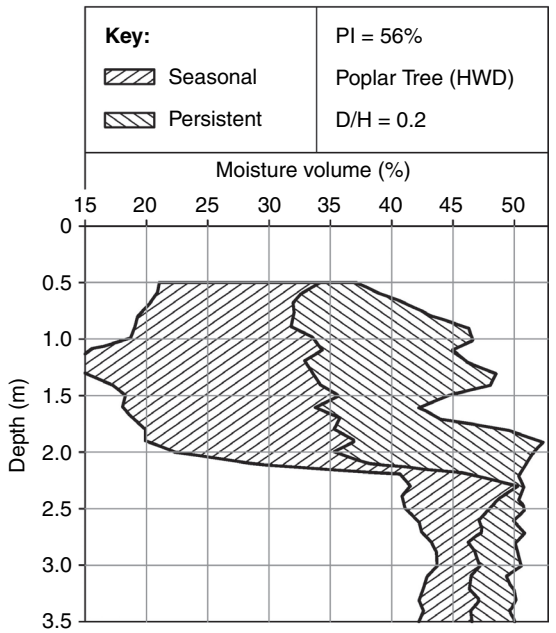
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Cause of movement	Comments	Typical mitigation measures
Non-engineered fills	Large movements may occur due to physical, chemical or biological degradation mechanisms. Refer to Chapters 29 and 58. Examples include collapse settlement on wetting; expansion of iron and steel slag; decomposition of organic material and gas or leachate generation	Ground improvement can be effective in loose fills, e.g. methods such as dynamic or vibrocompaction, preload/surcharge, provided organic materials are largely absent. If deformation mechanisms are due to chemical or biological factors conventional compaction methods may be ineffective
Interaction effects	The construction of new structures, services, earthworks, etc., may induce global ground movements, which can cause additional foundation movements	Design and construction of any new infrastructure needs to assess likely interaction effects, and mitigate if damaging movements are anticipated. Appropriate phasing of construction sequences may reduce adverse effects
Vibration	Loose or very loose coarse-grained soils (silts, sands and gravels) are vulnerable to large differential settlement when subjected to vibration from an external source, e.g. road traffic, machinery, etc.	Ground improvement to densify the coarse-grained soils into a dense or very dense state, e.g. dynamic or vibrocompaction, refer to Chapter 59 *Design principles for ground improvement*
Erosion	Uniform fine sand or silt is particularly vulnerable to erosion from surface or groundwater flows	Variety of erosion-protection measures, from natural, well-graded, granular filter layers, through to propriety geotextile products
Vegetation and shrink–swell	Magnitude of movement will increase as plasticity index of clay increases, and 'water demand' of trees increases. Not relevant for non-plastic soils; refer to NHBC, 2003	For ground affected, or likely to be affected, by trees, then special foundation design measures will be required, refer to NHBC, 2003. This may include piled foundations (able to resist heave and swelling pressures) and suspended floors on void formers to avoid swelling forces and upward movement
Adjacent slopes	Sloping ground near foundations can increase the risk of bearing-capacity failure. The slope may (without foundation loading) be close to failure, and near-surface soils on the slope may be vulnerable to seasonal movement and creep	Detailed inspection and assessment of slopes. Take into consideration for bearing-capacity checks, and assess 'global' stability. If factors of safety are inadequate, then slope stabilisation will be necessary
Bulk earthworks, cut and fill	Large-scale filling or excavation will change the stresses in the underlying natural ground; hence, settlement or swelling (respectively) will result. The fill itself may also settle or swell if it is fine grained	If possible, allow sufficient time for settlement or swelling to occur, prior to building new foundations. If not feasible, then foundation deformation calculations will need to allow for additional ground movements due to cut and fill
Metastable or voided ground	Ground previously mined is likely to be unstable. Natural deposits, such as limestone and chalk can contain voids and caves	Investigations need to be carefully designed and targeted. If voids suspected, then large-scale grouting is normally required, refer to Chapter 59
Frost	Silts, chalk and fine sands can be vulnerable to frost heave	Locate foundations below depth affected by frost. In the UK this is typically about half a metre

Note: Groundwater changes are also a common cause of global ground movements; refer to Table 9.1, Chapter 9 *Foundation design decisions*

Table 53.2 Potential causes of foundation movement independent of applied bearing pressures

The most common cause of excessive foundation movement for shallow foundations in the UK is probably seasonal shrink–swell movement induced by trees. These movements can affect foundations on clay soils, and are particularly significant in high-plasticity clay where trees are growing, or where previous tree growth has desiccated the clay. For example, if trees are removed prior to foundation construction then the clay soils will swell as the soil moisture content increases back to its equilibrium value. This can result in significant foundation movement and structural damage. If a tree is allowed to grow adjacent to a foundation in clay soil, then the tree can induce significant seasonal shrink–swell movement in the foundation leading to structural damage. For foundations on clay in open ground (remote from any significant vegetation such as a hedge, trees, etc.) a foundation depth of 0.9 m is usually adequate, in order to minimise the risk of excessive shrink–swell movement. For foundations on clays that could be affected by vegetation, NHBC (2003) has provided simple and comprehensive guidance. **Figure 53.1** gives an example of seasonal moisture-content change for a high-plasticity clay affected by high-water demand trees. **Figure 53.2** shows a comparison between moisture content and soil suction measured at a high-plasticity clay site, for a location remote from trees, and in an area affected by trees. The stress change induced by the trees (equivalent to the difference between the 'remote' and 'close to trees' suction measurements) is a maximum of about 500 kN/m^2 at a depth of 3 m, which is a far larger change in stress than would be applied by most conventional structures. Hence, the associated ground movements can be large, and can be of the order of 100 mm to 150 mm. NHBC provides simple rules for appropriate foundation depths, which will reduce the risk of structural damage. They define 'shrinkable' soils as soils



NB.
D = distance from tree
H = height of tree
PI = plasticity index of clay
HWD = high water demand
'Persistent' = reduction in moisture content due to tree, will only increase back to natural once tree dies or is cut down
'Seasonal' = change in moisture content between summer and winter

Figure 53.1 Example of changes in moisture content due to trees, Gault Clay

Modified plasticity index	Volume-change potential
≥ 40%	High
20% to 40%	Medium
< 20%	Low

Notes:
1. MPI, Modified plasticity index = [(Plasticity index) (particles < 425 μm, %)] / 100; i.e. if MPI = 30% and only 50% of particles pass through a 425 μm sieve then MPI = 15%.
2. Shrinkable soils are vulnerable to volume change as their moisture content changes.

Table 53.3 Volume-change potential
Data taken from NHBC (2003)

that contain more than 35% fines (particle size <60 μm) and with a *modified* plasticity index (MPI) of more than 10%. The volume-change potential is based on MPI; see **Table 53.3**. The risk of excessive foundation movement will also depend on the type of trees growing close to the foundations. Different tree types have been classified according to their 'water demand', and NHBC provides water demand categories and mature tree heights for common species. For example, oak, poplar and willow are classified as 'high water demand'. The appropriate foundation depth is dependent on the MPI of the soil, the water demand and the mature tree height of existing trees or new trees that may be planted; **Figure 53.3** provides an example of one of the NHBC charts.

53.3 Construction processes and design considerations

53.3.1 Foundation excavations

Foundation excavations will usually be on the critical path for a project's construction programme, since many activities will not be able to start until they are completed. Hence, although apparently simple, some thought is needed. **Figure 53.4** provides a flowchart, which outlines the main decisions that need to be made. Although much of the detail of the temporary works will be the responsibility of the contractor, the foundation designer needs to consider these matters, at least conceptually, to check that the proposed works are buildable, safe and do not endanger adjacent infrastructure, buildings, etc. Sometimes, it is quicker and cheaper to excavate an area in one large operation than in several small independent excavations. In many soils and rocks, excavation faces can stand at steep angles (in excess of 45°) for a short period following excavation. With increasing time (which may vary from minutes to months) the steep temporary slopes will fail and move towards slope angles similar to those of 'permanent' slopes (refer to Chapter 72 *Slope stabilisation methods* and Chandler, 1984). The period of time before a temporary slope fails is not predictable solely by theoretical calculations. The use of local experience in selecting appropriate temporary slope angles is extremely important. **Figure 53.5** highlights some typical issues. Health and safety regulations require that all excavations deeper than 1.2 m should be designed by a competent person. Excavations also need to be inspected regularly by a competent person. Contingency measures should be planned. These should be implemented, if adverse slope movements are detected.

53.3.2 Groundwater control

Groundwater control is an important consideration for any site that has a water table above the foundation excavation level. In most parts of the UK, the groundwater table will be within a few metres of the ground surface. Poor groundwater control is probably the most common cause of ground-related problems during construction. The consequences of poor groundwater control can vary from project delays to catastrophic collapse of excavations. The adverse effects can be classified as:

(i) *Seepage effects* – If seepage is concentrated across a relatively small area, then steep hydraulic gradients may arise. Steep hydraulic gradients can lead to: excavation slope instability, since the pore water pressures will reduce the

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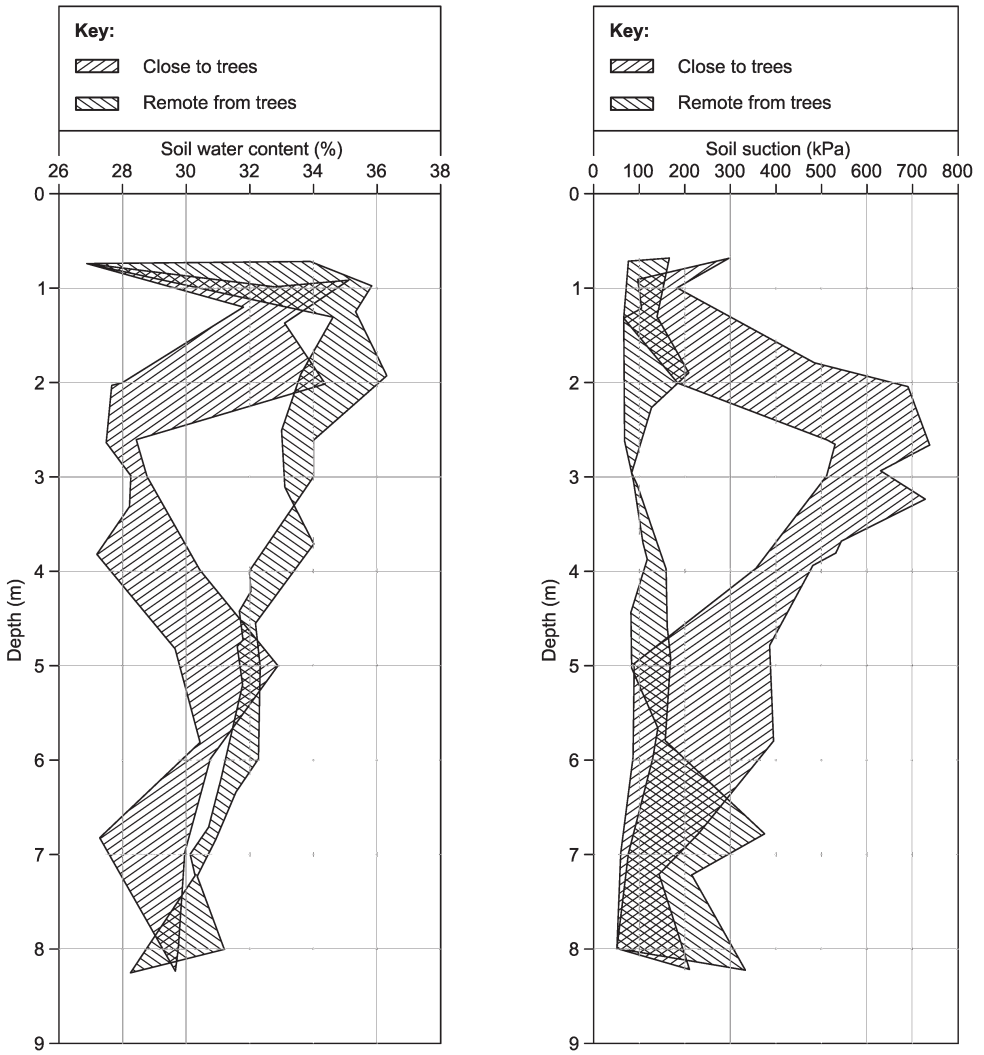


Figure 53.2 Water content and soil suction profiles remote and close to trees
Modified from Crilly and Driscoll (2000)

resistance to rotational or sliding failure; erosion of silts and sands, leading to piping, which can cause excavation basal instability or loss of lateral support to retaining walls; rapid weakening of otherwise strong competent soils (such as stiff clays, weak rocks) – layered 'laminated' soils with alternating low- and high-permeability layers (such as stiff clay with thin silt or sand layers) are particularly vulnerable.

(ii) *Hydraulic uplift* – In alternating layers of clays and sands instability will arise if the groundwater pressure in a sand layer (below the excavation base) exceeds the overburden stress due to the overlying clay layers, between the excavation level and the sand layer.

(iii) *Excessive foundation settlement* – Instability may not develop. However, if competent foundation soils, such as dense sands, stiff clays or weak rocks are affected by groundwater seepage, then this can cause reductions in the relative density of the sands or gravels; softening of stiff clays or joint opening in fractured rocks. Hence, excessive settlement may then occur once the foundation loads are applied. The original ground stiffness parameters of the soils or rocks in their undisturbed state will no longer be relevant. An example of the foundation problems caused by groundwater is given in Chapter 9 *Foundation design decisions.*

Figure 53.6 provides a sketch of the range of problems and some possible solutions to strong groundwater seepage and hydraulic uplift. The two main methods of avoiding serious groundwater problems are:

(a) lowering the groundwater pressure, by installing pumped wells or well points around the perimeter of the excavation;

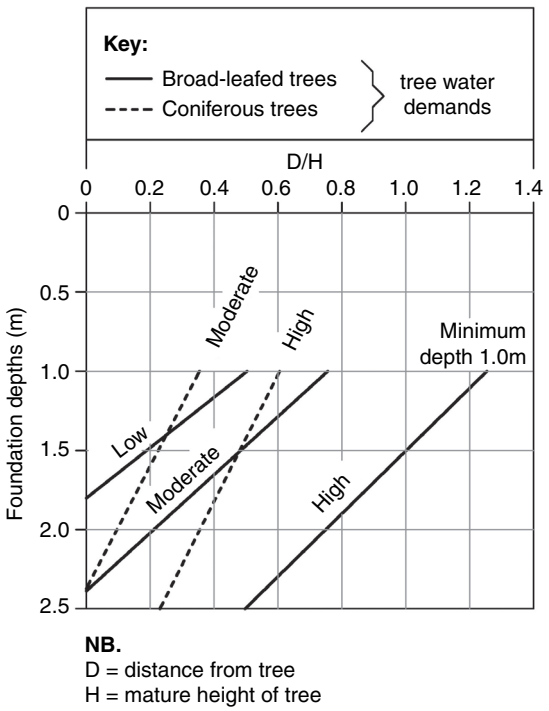


Figure 53.3 Example of NHBC foundation depth charts (soil with high volume-change potential, MPI≥40%). Shrinkable soils are defined as soils with more than 35% fines (finer than 60 μm) and have an MPI of 10% or greater
Reproduced from NHBC (2003)

(b) increasing the length of the groundwater flow path, either by installing deeper cut-off walls, providing more extensive grouted zones, or loading the excavation base with appropriately graded filter material.

Passive relief, or bleed, wells (drilled holes, backfilled with permeable filter material) can reduce the risk of hydraulic uplift, since the groundwater pressure gradient will be reduced from sub-artesian to hydrostatic. In highly permeable soils, passive relief wells may be 'swamped' and become ineffective.

For dewatering and depressurisation schemes, three main factors need to be considered:

(1) How much groundwater is present and how quickly it will be replenished, i.e. what groundwater flow rates need to be managed. In turn, this depends on ground permeability, continuity of permeable layers, presence of free water sources (rivers and lakes) and their connectivity to permeable layers.

(2) How the properties of the soils within and around the boundaries of the excavation will be affected. For example, the risk of loss of fines or risk of settlement once pore water pressures are reduced.

(3) How much water will need to be disposed of and the ease of disposal. Groundwater disposal requires approval of the regulator. The state of the groundwater, particularly the degree to which it may be contaminated will often be a critical consideration.

If the above factors are favourable, then dewatering can be relatively cheap, when pre-planned, compared with using cut-off walls or grouting. For complex situations a combination of cut-off walls, grouting and depressurisation may be necessary. Chapter 80 *Groundwater control* provides more detailed guidance on groundwater control. A wide range of different dewatering systems are available from discrete pumped wells to well points, the pros and cons are discussed in Chapter 80 *Groundwater control*. As a general rule, it is always preferable to locate the groundwater control systems, so that groundwater is extracted *outside* the footprint of the excavation, since this positively reduces hydraulic gradients below the base of an excavation. Sump pumping from shallow holes at the base of an excavation is often used, since it is relatively cheap and simple. However, it is not recommended, except for low-risk situations, e.g. occasional removal of small volumes of seepage water from perched water contained in limited zones of sand or gravel. Continuous pumping from sumps located inside the excavation is high risk, since hydraulic gradients are not reduced, and there is a high probability of loss of fines and piping instability.

53.3.3 Buoyancy and flotation

For sites with a high groundwater table or those sites that are vulnerable to flooding, it is important to check that the structure has an acceptable factor of safety against buoyancy failure, both during construction and in its permanent condition. Basement-type structures and cellular rafts (compensated foundations) are particularly vulnerable. Groundwater exerts uplift pressure on any overlying non-vertical surface; see **Figure 53.7**. The four main methods of reducing the risk of buoyancy failure are:

(i) Ensure that the sub-structure, once sealed against water inflow, always exerts a larger downward force than the uplift force from the water. The sub-structure must be structurally strong enough to withstand the water pressure acting on all external faces.

(ii) Allow the sub-structure to flood once the water level goes beyond the critical level for buoyancy. This can be acceptable during construction, but it is not appropriate for permanent structures! A simple means of achieving this is to install 'bleed' or pressure relief wells through the base of the sub-structure (**Figure 53.7**) or install holes through the sides of the sub-structure. The 'spill-over' level for the relief wells would be set at the level which maintains the target factor of safety against buoyancy.

(iii) Have dewatering or depressurisation systems available to keep the water level below the critical level. However, the

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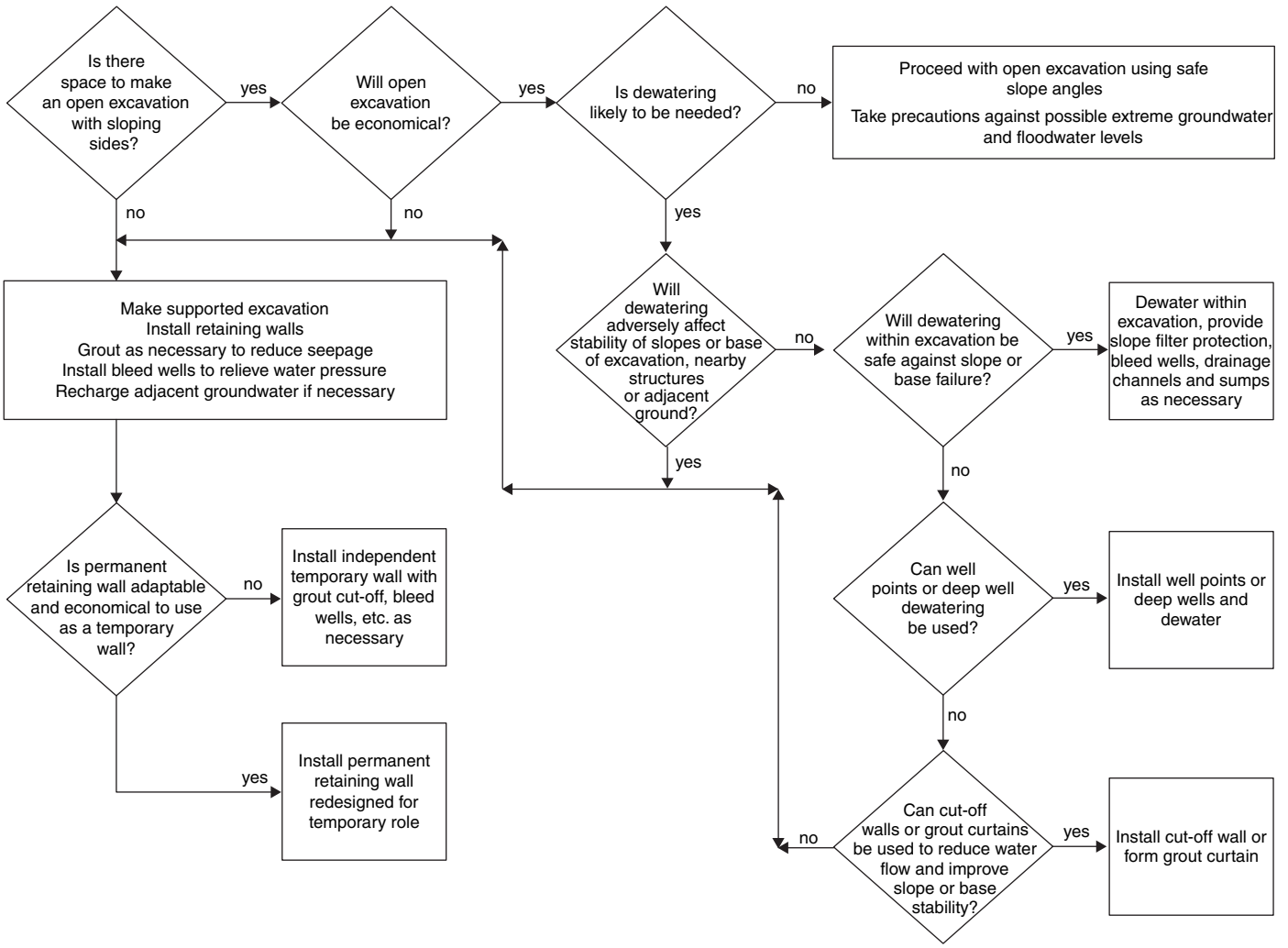


Figure 53.4 Excavation method, decision diagram
Reproduced from Cole (1988)

reliability and maintainability of these systems needs to be carefully considered. Dewatering can be appropriate to deal with groundwater, but will not be effective against surface-water flooding.

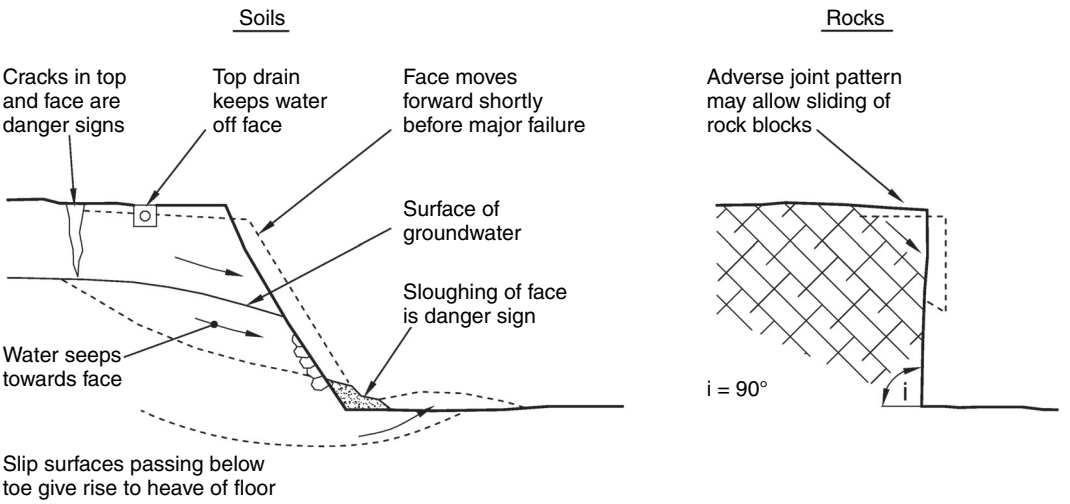
(iv) Ground anchors can be installed to anchor the structure to competent ground (**Figure 53.7**).

During construction, when the structure is incomplete, an additional stabilising load (say granular fill, concrete blocks, etc.) can be used. The kentledge can be removed, once the permanent structure has sufficient weight. To prevent overturning, the centre of the uplift force should be close to the centre of gravity of the structure.

53.3.4 Preparation of foundation formation

The settlement of shallow foundations can be sensitive to how carefully the excavated surface is prepared, prior to pouring foundation concrete. In wet climates, such as the UK, in addition to properly dealing with groundwater, care is required when excavating in inclement weather to minimise softening of exposed soils to rainfall. A practical means of minimising softening is to carry out bulk excavation to within say one metre to half a metre of the foundation level. The final excavation stage is subsequently carried out as a final task, in short bay widths, and the exposed soil or rock is immediately (or within the same work shift) protected by structural quality blinding concrete. When dewatering is carried out, the groundwater surface level should be kept a minimum of half a metre, and preferably more than one metre below the foundation level.

Foundation works for many sites may require the removal of old foundations, large boulders and ‘soft spots’. This may result in an irregular profile, partly below the foundation level. A final trim should be made to remove disturbed material. Then voids below foundation level will need to be infilled with good quality coarse-grained fill or mass concrete. Coarse-grained fill is only



Typical short-term slope angles: stiff clays 35 to 60°; sandy clays 35 to 50°; these angles assume dry conditions. These slopes will degrade and become unstable over time, which may vary from minutes to months depending on mass permeability of ground. Laminated/varved clays are particularly vulnerable to rapid degradation. Good drainage, groundwater control and protection of slope from weather are critically important.

Beware adverse joint pattern, may allow sliding of blocks. Some rocks, notably mudstones/shales deteriorate rapidly once exposed to air/water (wetting/drying).

Figure 53.5 Excavation side-slope stability
Reproduced from Cole (1988)

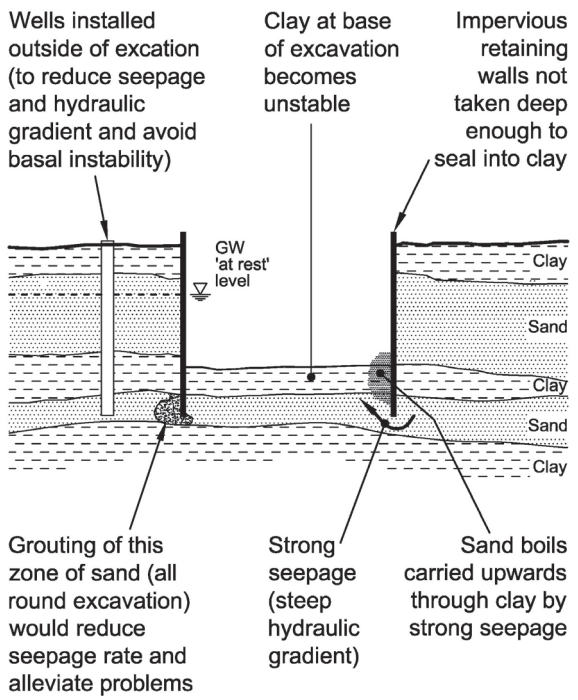


Figure 53.6 Excavation instability due to groundwater and common mitigation measures
Modified from Cole (1988)

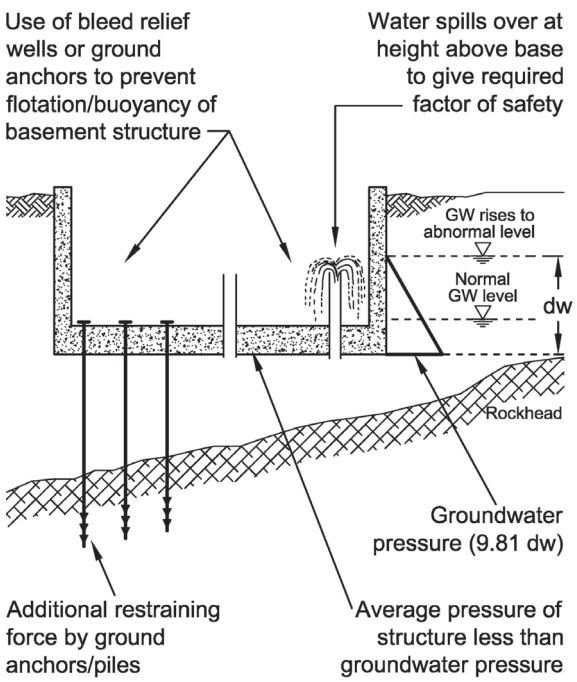


Figure 53.7 Buoyancy and floatation failure of sub-structure and common mitigation measures
Modified from Cole (1988)

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acceptable *if* it can be properly compacted. The possible presence of soft spots can be assessed by probing (Chapter 47 *Field geotechnical testing*) or by observing if ruts develop when large plant traverse across the formation. At this stage if it is apparent that the ground conditions are different to those assumed by the designer, then it is vital that the designer is informed and has the opportunity to modify the foundation design.

53.4 Applied bearing pressures, foundation layout and interaction effects

53.4.1 Applied bearing pressures

Gross and net bearing pressures are defined in Chapter 52 *Foundation types and conceptual design principles* (section 52.5). The net bearing pressure is used to check the factor of safety against bearing-capacity failure and assess the total settlement of foundations. This is the principle that is used to design 'floating' or 'compensated' foundations (Chapter 52 *Foundation types and conceptual design principles*, Figure 52.1), where the weight of the structure is close to the weight of soil removed at the base of the foundations, so that the net bearing pressure on the soil is close to zero. D'Appolonia and Lambe (1971) summarise observed movements of compensated foundations on very soft clays, all of which exhibited very small settlements, which terminated shortly after construction.

The applied loads on a foundation will usually comprise a variety of different types of loads, including dead loads (weight of sub-structure, superstructure) and transient (or live) loads including wind, impact (or accidental) loads, etc. It is important to consider the different types of loading in the context of the ground behaviour adjacent to the foundation. For example, it would be inappropriate to carry out a drained bearing-capacity analysis for clays for a load combination that included an impact load which lasts a fraction of a second. In contrast, foundations on sands can be vulnerable to significant incremental increases in settlement if subjected to regular transient loads, due to a ratcheting-type deformation mechanism (e.g. tall towers or chimneys subjected to high wind loads). When calculating the long-term settlement of clays, occasional transient loads can be ignored, and usually only a fraction of the live load would be considered (say 25% to 30%).

53.4.2 Foundation layout and interaction effects

When carrying out bearing-capacity and settlement checks it is important to consider the behaviour of the whole structure, in addition to the individual foundation elements, within the context of the site and adjacent structures and topographical features. **Figure 53.8(a)** and **(b)** highlights an example of shallow foundations for a building where the strip footings are located

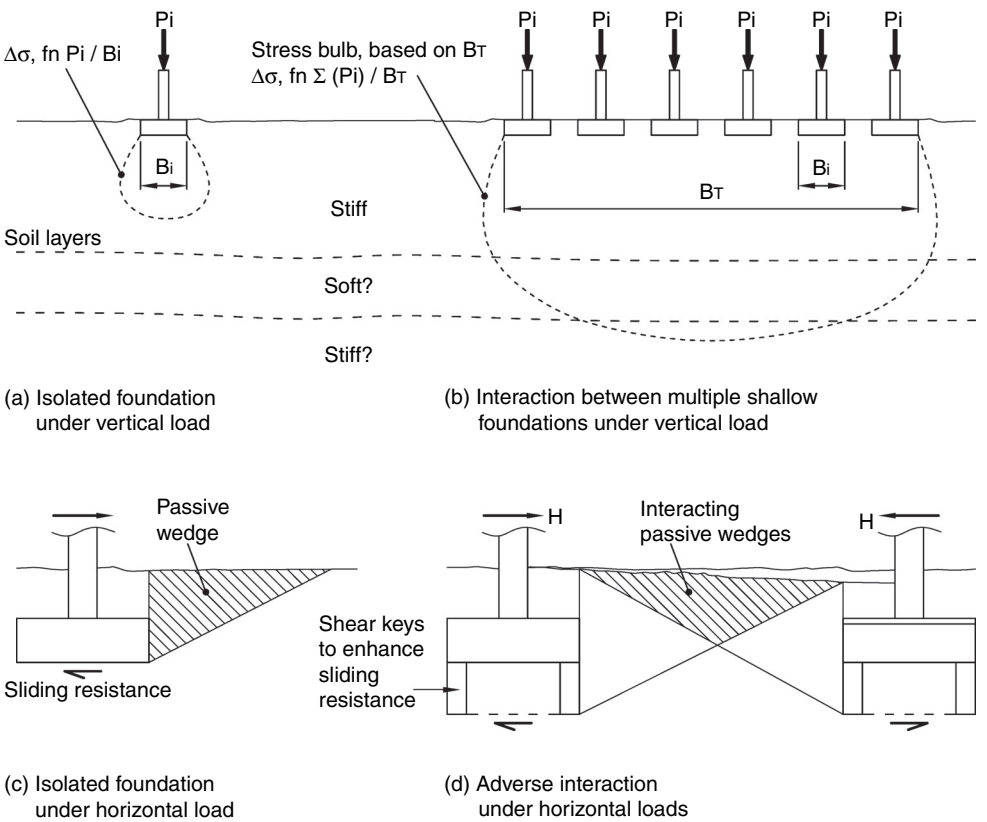


Figure 53.8 Multiple shallow foundations, examples of interactions under vertical and horizontal loads

relatively close to one another. For this situation it would be wise to carry out two separate checks:

(i) Check the bearing capacity and settlement of an isolated footing.

(ii) Check the bearing capacity and settlement of a hypothetical raft beneath the entire building, subjected to an equivalent net bearing pressure (based on the overall building weight and the width and length of the structure).

Particular care is required if a foundation is subjected to a lateral load, since this increases the risk of adverse interaction effects; see **Figure 53.8(c)** and **(d)**. **Figure 53.8(d)** illustrates a small single-span bridge founded on shallow pad footings. Shear keys can be designed to resist the applied horizontal loads. However, if the 'passive wedges' in front of the shear keys for each bridge abutment interact, then the available passive resistance will be much less than that for a single isolated foundation. Care is also required if forming shallow foundations that are adjacent to existing shallow foundations, adjacent to slopes or if drain and service trenches will be constructed close to shallow foundations. For these situations the bearing-capacity and settlement checks must take account of the potentially adverse interaction effects.

53.5 Bearing capacity

53.5.1 General comments

The bearing capacity can be defined as the maximum load per unit area that can be imposed on a soil at a given depth before failure occurs. It is a function of many parameters, including physical properties of the soil, foundation geometry, foundation depth, soil failure mode and load inclination or eccentricity. The key parameters are illustrated in **Figure 53.9**.

53.5.2 Basic formulae for strip footings close to ground surface

Chapter 21 *Bearing capacity theory* should be studied for an introduction to the topic.

In the effective stress bearing-capacity equation, the appropriate value for N_γ can be controversial and a bewildering array of different formulae have been published. It should be noted that the formulae by Terzaghi and Vesic are now considered to be grossly unconservative (Poulos *et al.*, 2001). The more conservative equations by Brinch and Hansen (1970) or Davis and Booker (1971) are recommended. The Brinch-Hansen equation is given in Chapter 21 *Bearing capacity theory*.

It should be noted that as the angle of friction increases beyond 30°, the bearing-capacity factors increase rapidly, and even small increases in the angle of friction lead to dramatic increases in the bearing capacity.

53.5.3 General bearing-capacity formulae

For many practical situations the simple bearing-capacity formulae given in Chapter 21 *Bearing capacity theory* have to be modified to take into account the effects of:

(i) Soil compressibility: Important for relatively loose sands and sands with non-silica mineralogy, or intermittent organic and clay layers or pockets. This correction will lead to a reduction in bearing capacity.

(ii) Stress level: As mean effective stress at failure increases, the ability of sands to dilate is reduced. Hence, as foundation depth or foundation width increases, the mobilised angle of friction will tend to reduce.

(iii) Groundwater table and groundwater pressures: Important for sands, could lead to a significant reduction in bearing capacity, especially if sub-artesian or artesian groundwater conditions are present.

(iv) Shape and depth corrections: Appropriate for all soils, depending upon shape and depth of foundations.

(v) Increasing strength with depth, soil layering and anisotropy of strength: For clays the simple formulae assume the clay's strength is constant with depth and is isotropic. This is sometimes inappropriate and the simple formulae can then be unsafe.

(vi) Combined vertical, moment, horizontal loads: For sands and clays the empirical correction factors that have been developed and published in codes and textbooks can be appropriate for use if the resultant load inclination is relatively modest (within the middle third of the footing width, or a load inclination of less than about 18°). For more severe resultant load inclinations, these empirical corrections become less reliable. A radically different approach to dealing with complex load combinations has been developed over the last twenty years or so (mainly stimulated by offshore engineering). These methods are

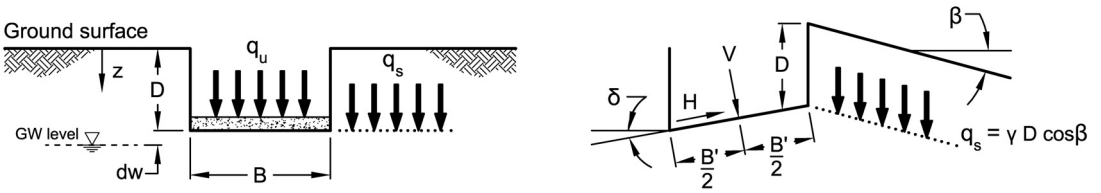


Figure 53.9 Geometry and parameters for ultimate bearing capacity

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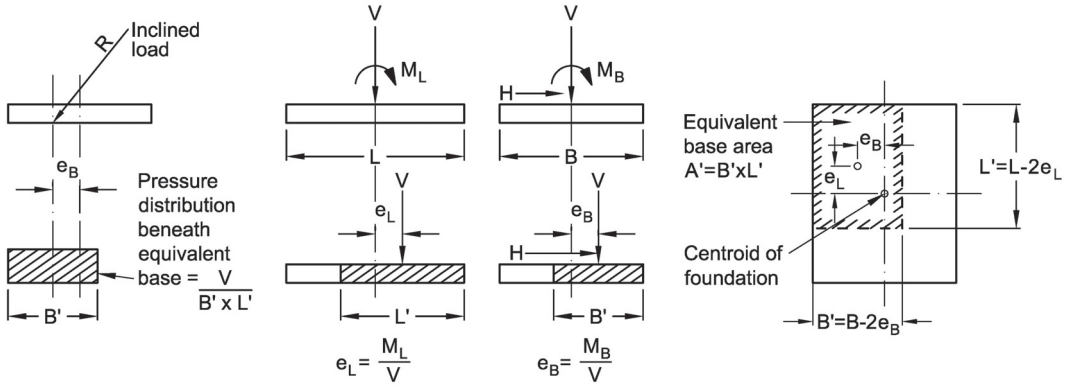


Figure 53.10 A shallow foundation subject to eccentric load and the equivalent area carrying a uniform vertical pressure

briefly outlined in Chapter 21 *Bearing capacity theory*. Load inclination, especially for sands, can lead to a dramatic reduction in bearing capacity.

A useful overview of recent developments in bearing capacity theory is given by Poulos *et al*. (2001), and Randolph *et al*. (2004).

The general bearing-capacity formula for undrained saturated clays and drained soils is outlined in Chapter 21 *Bearing capacity theory*. Shape and depth factors are given, and methods to deal with inclined and offset loads are also outlined. **Figure 53.10** outlines Meyerhof's simple method for eccentric, or offset, vertical loads. For eccentrically loaded foundations, the plan dimensions of the foundations should be reduced to:

$$B' = B - 2e \tag{53.1}$$

and the equivalent gross bearing pressure will be

$$q_e = w / (B' \times L) \tag{53.2}$$

where B' is the corrected width, e is the eccentricity of the load, w is the foundation load, B is the actual width and L is the length.

Based on q_e, the net bearing pressure over the effective base area for the eccentrically loaded foundation should be calculated, q_{ne}; q_{ne} should then be used for both bearing-capacity and settlement checks. The above approach of deriving an 'effective foundation area' is recommended when the eccentricity falls within the 'middle-third' of the foundation width (i.e. $e < B / 6$). The alternative approach of assuming a linear variation of bearing pressure (of trapezoidal or triangular shape) below the base is not recommended. If the eccentricity falls outside the middle third, then it is generally wiser to ensure more uniform loading by reconfiguring the foundation geometry rather than attempting more sophisticated analysis. For sustained eccentric loads, the foundation can be designed more efficiently by placing the superstructure column off-centre, so that the resultant of the axial and lateral loads passes through the centroid of the foundation footing.

53.5.4 Allowing for soil compressibility and stress level

It is often assumed that bearing capacity is dependent purely on the ground strength; in practice it is also a function of its compressibility (**Figure 53.11**). The compressibility of sand is a function of its relative density, its mineralogy and, if present, local lenses of clays. A dense silica sand or stiff clay may develop a well-defined ultimate resistance, which is associated with 'general shear failure'. In contrast, the failure of more

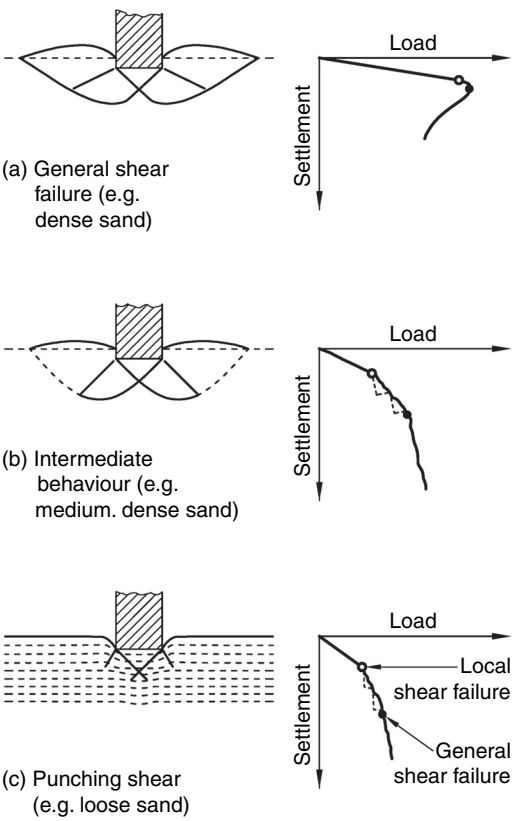


Figure 53.11 Influence of soil compressibility on the bearing-capacity failure mechanism

compressible soils is characterised by progressive downward movement, often referred to as 'punching' or 'local' shear failure. The basis of conventional bearing-capacity theory is 'general shear failure', which is only appropriate for the drained failure of dense or very dense silica sands and the undrained failure of clays. 'Punching' shear is likely to occur in very loose to medium dense silica sand, any sand which is intrinsically compressible (such as calcareous or micaceous sands), or during the drained failure of clays, and requires the bearing-capacity factors for general shear failure to be corrected. A simple means for allowing for soil compressibility is to reduce the soil's angle of friction in the formula for drained bearing capacity. For loose and very loose sands, the mobilised angle of friction is assumed to be two-thirds of the characteristic or moderately conservative value (interpreted from ground investigation data); for dense and very dense sands the mobilised angle of friction is assumed to be equal to the characteristic value; for medium dense sands an assumed mobilised angle of friction of about 80% of the characteristic value is usually appropriate.

53.5.5 Influence of groundwater table level and groundwater pressure

The general bearing-capacity equation is based on the assumption that the water table is located well below the foundation ($dw \geq B$). However, if, as is often the case, the water table is close to the foundation, then some modifications to the unit weight and surcharge terms (γ and q, respectively) are necessary depending on the location of the water table (**Figure 53.9**):

Case 1: If the water table is located above the bottom of the footing, then the effective surcharge is $q = q'$ (the vertical effective stress at the foundation level is γD-u, where u is the groundwater pressure at the foundation level). The unit weight of the soil needs to be adjusted as follows:

$$\gamma' = \gamma_{sat} - \gamma_w \tag{53.3}$$

where γ_{sat} is the saturated unit weight, γ_w is the unit weight of water and γ' is the buoyant unit weight.

Case 2: If the water table is located at the foundation level, i.e. $dw = 0$, then q is the total vertical stress at the foundation level (since the groundwater pressure is zero).

$$\gamma' = \text{buoyant unit weight (as Case 1).} \tag{53.4}$$

Case 3: If the depth to the water table $dw \geq B$, then q is the total vertical stress at the foundation level and

$$\gamma' = \text{saturated unit weight.} \tag{53.5}$$

Case 4: If, assuming no seepage forces, the water table is located at a depth $dw < B$ (i.e. the depth to water is within a depth less than the footing width) then interpolate between Cases 2 and 3 above:

$$q = \gamma D \text{ (as Case 3).} \tag{53.6}$$

The unit weight in the first term of the bearing-capacity equation must be replaced by the factor

$$\gamma_{mod} = \gamma' + d/B\ (\gamma - \gamma'). \tag{53.7}$$

Case 5: If the water table is at the foundation level, $dw = 0$, and there is uniform upward seepage (of gradient $du / dz > \gamma w$), then

$$\gamma' = \gamma - \frac{du}{dw}$$

and

$$\gamma' = \gamma - \gamma w\ (1+i) \tag{53.8}$$

where i is the hydraulic gradient (hence, a positive gradient can substantially reduce the bearing capacity).

53.5.6 Sand overlying clay

Various theoretical and experimental studies have been conducted into the ultimate bearing capacity of a footing on a layer of sand overlying clay, e.g. Hanna and Meyerhof (1980) and Okamura *et al.* (1998). This is a common scenario in foundation engineering and punch-through failure may be a genuine concern, particularly for relatively thin sand layers overlying clays.

Conventional analyses of the bearing capacity of sand over clay use limit equilibrium techniques and generally look at two proposed failure mechanisms, as shown in **Figure 53.12**. In each case the strength of the sand is analysed in terms of effective stress, using the effective unit weight (γ) and the friction angle (ϕ), while the analysis of the clay is in terms of total stress, characterised by its undrained shear strength (S_u). In the case of the 'projected area' method an additional assumption about the angle α (**Figure 53.12(a)**) is required, although a value of 30° is often assumed.

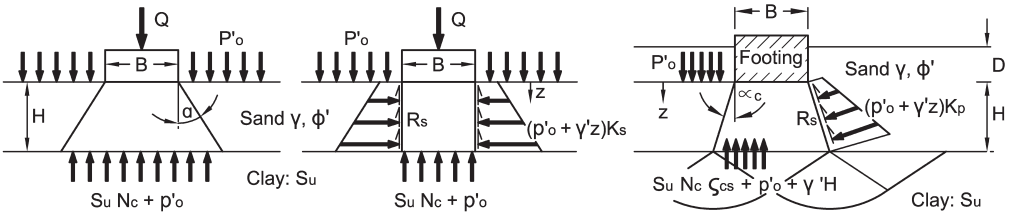
Okamura *et al*. (1998) assessed the validity of these two approaches and proposed an alternative failure mechanism; see **Figure 53.12(c)**.

53.5.7 Foundations on rock

When foundations are founded on intact rock then the bearing capacity is a function of the unconfined compressive strength of the rock and in most cases is more than sufficient to provide the required bearing capacity. However, the vast majority of rocks are not intact and have discontinuities, which, depending on their orientation and spacing, and the properties of the discontinuity infill material (if present), will have a significant impact on the bearing capacity of the rock. Franklin and Dusseault (1989) describe several different failure mechanisms:

(1) General bearing-capacity failures, similar to those which occur in soil are uncommon in rock. These failures may occur beneath heavily loaded foundations on weathered, weak, highly fractured mudstone.

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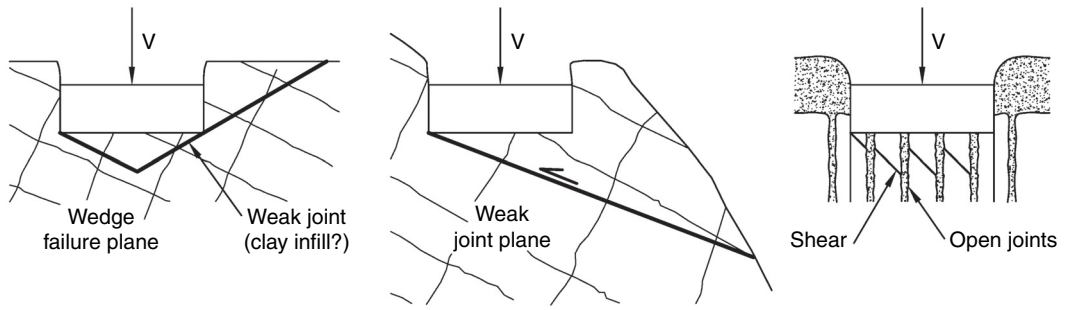


(a) Projected area method

(b) Hanna and Meyerhof method

(c) Okamura *et al.* (1998) method

Figure 53.12 Alternative failure mechanisms, sand over clay



(a) Wedge failure

(b) Sliding failure (adverse joint orientation)

(c) 'Unconfined' failure (open vertical joints)

Figure 53.13 Typical failure mechanisms for jointed rock masses

(2) 'Consolidation' failures in weathered, fractured rocks, where unweathered rock corestones fail due to a combination of low shear strength along clay-coated vertical joints and voids or compressible fillings along horizontal joints.

(3) 'Punching' failure in porous rocks, such as volcanic tuffs, limestone and chalk, due to local crushing of the rock adjacent to voids where the rock is heavily stressed.

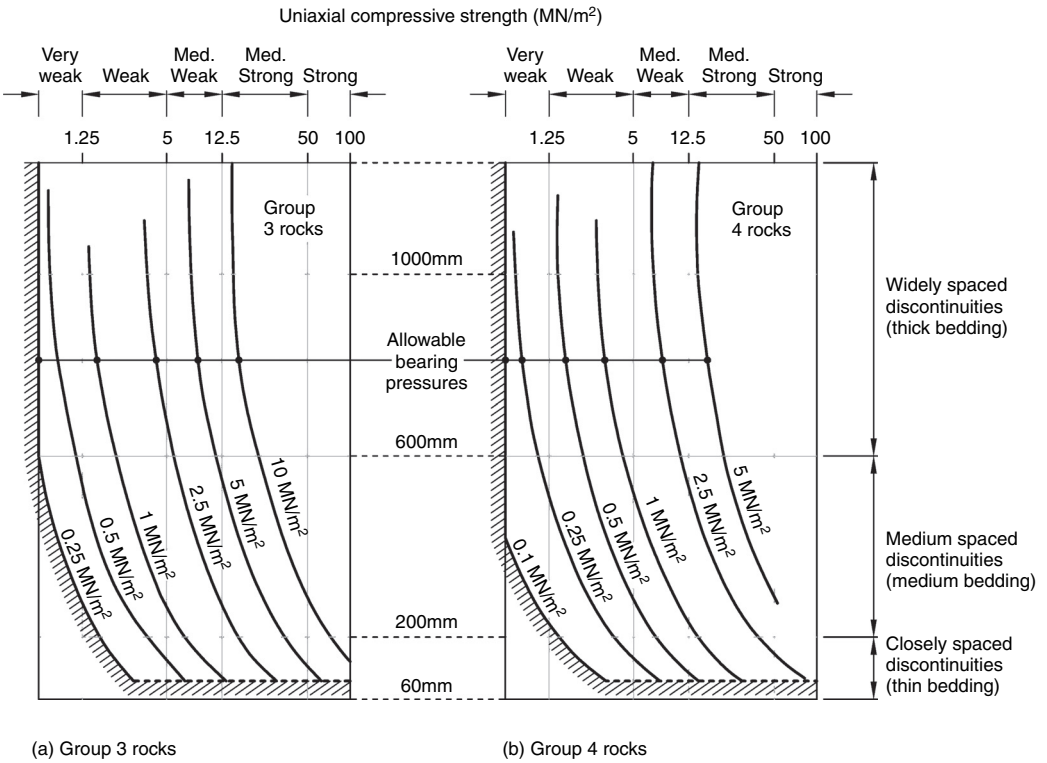
(4) Wedge or sliding failure along unfavourably inclined joints (**Figure 53.13**). For this situation, the overall equilibrium of the foundation due to potential sliding along the discontinuity is checked by resolution of forces, and consideration of the specific shear strength characteristics of the critical discontinuities. If these are infilled with clay, fault gouge or are slicksided, the shear resistance may be very low.

(5) Subsidence due to collapse of strata, undercut by subsurface voids, where the voids are natural or mining induced. Natural voids can be formed by solution weathering of soluble rocks, such as gypsum, rock salt, chalk and limestone. These can form deep pipe or cave-like features covered by metastable arches of soil. These may subsequently collapse, due to changes in the stress or groundwater regime.

The bearing capacity of a jointed rock mass was discussed by Wylie (1991), who also discusses the stability assessment of foundations on rock slopes and on steeply bedded formations. If the rock mass contains numerous open joints, then the ultimate bearing capacity will be equivalent to the unconfined compressive strength of the rock (**Figure 53.13(c)**).

For relatively weak and fractured rocks BS8004 provides a simple set of charts for preliminary assessment of allowable bearing pressures; see **Figure 53.14**. The curves shown are for 'Group 3' and 'Group 4' rocks. The allowable bearing pressures for more competent 'Group 1' and 'Group 2' rocks are higher (BS8004). These curves are only intended to be used for the design of simple, small structures. The charts assume joints are closed, and if open joints are suspected, then the allowable bearing pressures will need to be reduced, perhaps halved. For other structures more detailed analysis is required (e.g. refer to Wyllie, 1991).

When the rock material is very weak, has very closely spaced discontinuities, or is very heavily weathered or fragmented, the rock can be treated as a soil mass and designed on the basis of the conventional bearing-capacity equations. For situations where there is a mix of strong and weak materials with complex discontinuity patterns then the Hoek–Brown strength criterion (Hoek and Brown, 1997) should be used to evaluate the bearing capacity of the rock mass (Hoek, 1999; Hoek *et al.*,



(a) Group 3 rocks

(b) Group 4 rocks

NB.
Allowable bearing pressures in hatched areas to be assessed after inspection and/or making tests on rock.
Group 3: Very marly limestones, poorly cemented sandstones; cemented mudstones/shales; slates and schists.
Group 4: Uncemented mudstones and shales.
More competent rocks in Groups 1 & 2 (e.g. limestones, igneous rocks) have higher allowable bearing pressures, refer to BS8004.

Figure 53.14 Allowable bearing pressures for square pad foundations bearing on rock (for settlement not exceeding 0.5% of foundation width)
Reproduced with permission from BS8004 © British Standards Institute 1986

2002). Specialist advice should be sought where inclined or eccentric loads are applied to footings on rock, and when voids are suspected in the rock (rocks such as limestones, chalk and carbonate-based rocks are susceptible to voiding or karstification). Argillaceous rocks (mudstone, shale and siltstones) are apt to swell once unloaded during foundation excavation, and exposure to water or cycles of wetting and drying will lead to softening and degradation.

53.6 Settlement

53.6.1 Introduction

The settlement of a foundation must be within tolerable or acceptable limits, as discussed in Chapter 52 *Foundation types and conceptual design principles*. For most shallow foundations the estimated settlement will be a more critical design consideration than bearing capacity. There are a wide range of methods available to estimate settlement and some of the commonly used methods will be discussed in this section. However, for all methods the most challenging aspect is establishing the soil or rock properties and the associated deformation parameters. The use of more sophisticated analytical methods will not compensate for poor ground investigation data or an inappropriate interpretation of ground properties. If ground movements do not occur due to external sources (refer to section 53.2), then the main components of settlement due to foundation loading are summarised in **Table 53.4**. The basic methods involved in calculating the settlement in sands and clays are described in Chapter 19 *Settlement and stress distributions*.

Clays and silts experience both undrained shear and primary consolidation settlement. Creep settlement is usually assumed to be negligible for heavily overconsolidated clays and silts (which would usually be the founding materials), but can be significant for normally or lightly overconsolidated clays and silts (especially for highly organic materials, such as peat).

Settlement component	Relevant ground conditions	Comments
1. Undrained shear distortion (no change in volume/void ratio)(1)	Clays, clayey silts, very weak and weak argillaceous rocks	Becomes more significant as the factor of safety reduces. Typically, EOC settlement is 10% to 20% of total settlement for soft clays and about 50% to 75% of total settlement for stiff clays(2)
2. Primary consolidation settlement due to a reduction in volume/void ratio, due to an increase in effective stress	All soils and rocks	Main component of settlement for most conditions. Case histories show significant primary consolidation settlement occurs during construction, for overconsolidated clays, sands and rocks
3. Creep, or secondary consolidation, settlement, due to a reduction in volume/void ratio, at constant effective stress	Soft lightly overconsolidated and normally consolidated clays and peats	Becomes significant once pre-consolidation pressure exceeded. Creep increases as organic content increases, and as clay sensitivity increases
4. Deformation along or across joints, fissures or discontinuities	Rocks	At high stresses, creep can be significant, refer to text. Chalk can be vulnerable to creep once yield stress exceeded. Manifested as a time-dependent 'creep' settlement. Swelling during excavation, or poorly controlled groundwater conditions, can exacerbate 'creep' during subsequent loading
5. Change in volume/void ratio due to cyclic loading or vibration	Sands, gravels and sandy silts	Observations (e.g. Burland and Burbidge, 1984) indicate that conventional foundations exhibit time-dependent settlement, even under 'static' load (implying 'creep' deformation, see point 3 above). But significant increases in time-dependent settlement occur if 'fluctuating' loads are applied

Notes:
1. For saturated soils.
2. End of construction settlement (EOC) includes both undrained and a proportion of the primary consolidation settlement.

Table 53.4 Components of foundation settlement, due to applied bearing pressure

The commonly used methods of settlement analysis implicitly assume linear elastic ground behaviour, or simple nonlinear behaviour in the form of either a nonlinear elastic response or by distinguishing between the changes in compressibility, which occur between overconsolidated and normally consolidated states (or pre-yield and post-yield states for bonded soils and some weak rocks). As outlined by Burland *et al*. (1977), for the case of foundations that are loaded vertically, with a factor of safety in excess of three, the major principal stress increments remain practically vertical beneath most of the loaded area and the axes of stress and strain increments are mainly coincident. Hence, for this situation the behaviour of elastic, plastic and viscous materials is practically the same and relatively simple stress–strain relationships are adequate. If the foundation is subject to large inclined loads (due to significant moment and horizontal load combinations), then the situation is potentially far more complex, since significant principal stress rotations occur, and principal strain increments will not necessarily be coincident with principal stress increments. It should always be remembered that the main body of experience in shallow foundation behaviour is related to foundations subjected to vertical loads (some nominal horizontal or moment loads may be present, due to say wind loads, but these are relatively minor and transient). For other situations a more cautious approach is necessary.

If a foundation is subjected to an offset or eccentric vertical load, the Meyerhof method described in section 53.5 (refer to **Figure 53.10**) can be used to calculate the equivalent uniform bearing pressure. Conventional methods can then be used to assess the overall foundation settlement.

53.6.2 Stress changes due to applied bearing pressures

Theoretical considerations are discussed in Chapter 19 *Settlement and stress distributions*. For practical applications, the changes in stress with depth are mainly based upon homogeneous, isotropic, linear elastic theory, e.g. Boussinesq, or similar. Burland *et al*. (1977) have discussed in detail the influence of a wide range of factors on the accuracy of predicted stress changes with depth. For many ground conditions the equations based upon linear elastic theory give a reasonably accurate estimate of *vertical* stress changes with depth.

A useful design chart for estimating vertical stress changes with depth has been produced by Janbu, Bjerrum and Kjaernsli (1956) and is shown in **Figure 53.15**. This chart gives the increase in vertical stress beneath the centre of a uniformly loaded flexible area of strip, rectangular or circular shape.

53.6.3 Elastic methods

Elastic methods should not be used when the final stress state in a layer exceeds the pre-consolidation or yield stress of the ground (i.e. $\sigma'_{voi} + \Delta\sigma_{vi}' > p'_{ci}$), where σ'_{voi} is the initial vertical effective stress and $\Delta\sigma_{vi}'$ is the vertical effective stress increment for the layer under consideration. p'_{ci} is the pre-consolidation pressure or yield stress.

The undrained settlement of clays is often *mislabelled* as 'elastic settlement', because elastic theory has been used for calculation purposes; this erroneous terminology should not be used. It is now widely accepted that a wide range of overconsolidated soils and rocks can be treated as 'elastic' for predicting the *total* settlement (where total settlement is undrained

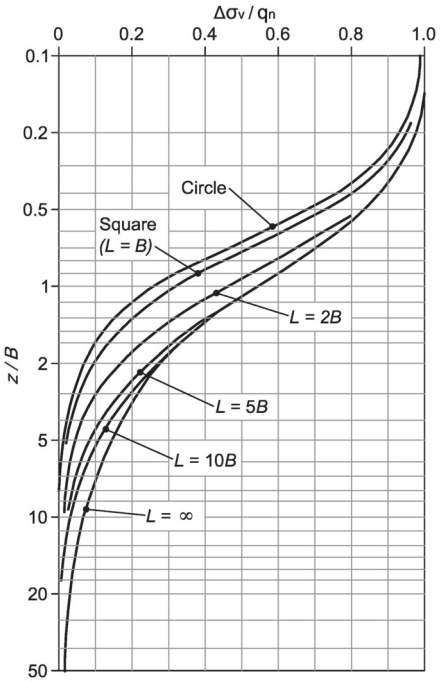


Figure 53.15 Vertical stress increase beneath centre of flexible footings of varying geometry.
Reproduced from Janbu *et al.* (1956)

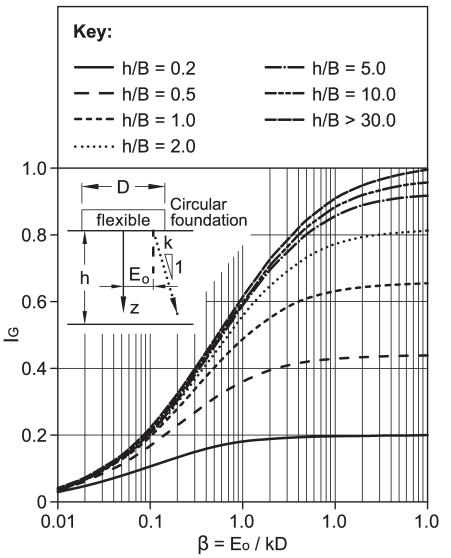


Figure 53.16 Displacement influence factors for the settlement of a flexible circular foundation

plus time-dependent settlement), e.g. see Burland *et al.* (1977), and Mayne and Poulos (1999).

Shallow foundation settlement can be assessed very quickly by using displacement influence factors derived from elastic theory. Unfortunately, many published solutions assume the elastic material has a constant value of Young's modulus with depth and the elastic material is of infinite thickness. These types of elastic solutions are *not* recommended. Mayne and Poulos (1999) provided solutions for the settlement of a shallow foundation resting on an isotropic elastic material of finite thickness, whose Young's modulus increases linearly with depth (i.e. $E = E_0 + kZ$, where E_0 is Young's modulus directly beneath the foundation base at $Z = 0$ and kZ is the rate of increase of the modulus with depth); as noted in Chapter 52 *Foundation types and conceptual design principles*, this type of stiffness variation is relevant for many design situations:

$$S = q\, D\, I_G\, (1 - \nu^2)\, f_R f_d / E_0 \qquad (53.9)$$

where S is the foundation settlement, q is the net bearing pressure, D is the diameter of the circular foundation, I_G is the influence factor for non-uniform ground stiffness, given in **Figure 53.16**, ν is Poisson's ratio, E_0 is Young's modulus directly beneath the foundation, f_R is the correction factor for the foundation stiffness, refer to section 53.6.7 and f_d is the correction factor for the foundation depth, refer to Mayne and Poulos (1999).

The influence factor for non-uniform ground stiffness is plotted against the dimensionless term, $\beta = E_0 / kD$, where k is the increase in Young's modulus with depth. Although developed for circular foundations, this method can be used for square and rectangular foundations (for $L \leq 3B$, where L is the foundation length and B is the breadth) and has been shown to compare well against more rigorous solutions. For rectangular foundations where $L > 3B$, Meigh (1976) has produced appropriate elastic solutions.

53.6.4 Settlement of clay

This section considers heavily overconsolidated clays only. The commonly used methods for estimating the settlement of clays are summarised in **Table 53.5**. The use of elastic methods has been outlined above. For the classical one-dimensional method the foundation settlement is calculated from:

$$S_{oed} = \sum_{i=0}^{i=n} (m_{vi}\, \Delta\sigma'_{vi}\, H_i) \qquad (53.10)$$

where m_{vi} is the coefficient of volume compressibility derived from oedometer testing. The one-dimensional method has been criticised as being too crude, and a variety of other methods, which offer a more sophisticated approach, have often been preferred. Burland *et al.* (1977) carried out an objective analysis of the accuracy of the various methods, which is discussed in detail in Chapter 19 *Settlement and stress distributions*. The main conclusion is that the one-dimensional method, for soils that are approximately elastic in their response to monotonically increasing stresses, provides results that are at least as accurate, and often more accurate, than results from apparently more sophisticated methods. Hence, for overconsolidated

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Method	Compressibility parameters	Advantages/disadvantages	Comments
(1) One-dimensional method, general comment	See below	Simple. Soil layering readily taken into account. Vertical stress changes reliable for many practical situations	Burland *et al.* (1977) showed that the one-dimensional method is more reliable than more complex methods, ***if*** soil compressibility is appropriate for stress change and strain amplitude
(a) Oedometer	Coefficient of volume compressibility, m_v	Principal problem is reliance on m_v from oedometer tests, which are usually unreliable due to sample disturbance	For overconsolidated clays, settlement is grossly overpredicted due to deficiencies in conventional oedometer test data
(b) Modified	Constrained modulus, D′	Constrained modulus can be related to moduli obtained from wide variety of tests, such as oedometer, triaxial and bender element (see 53.6.4)	Most appropriate method for stiff overconsolidated clays; modulus for each layer specific to stress level and strain amplitude, see sections 53.7 and 53.8
(c) Compression Indices	Compression indices, pre and post pre-consolidation pressure (C_r and C_c)	Compression indices can be correlated to carefully conducted oedometer tests, or, more commonly, empirical correlations	Most appropriate method for soft clays; pre-consolidation pressure is the critical parameter
(d) Skempton and Bjerrum	m_v and 'correction factor' U_g	Historically justified to 'correct' the settlement derived from routine oedometer tests. Calculates 'consolidation' settlement only; extra calculation needed for undrained settlement	As noted by Burland *et al.* (1977), this method is obsolete and does not improve reliability of predictions
(2) Linear elasticity (a) Classical equation	Young's modulus and Poisson's ratio	Principal flaw is reliance on horizontal stress change, which for many practical situations (involving anisotropy, increasing stiffness with depth and nonlinearity) is unreliable. Experience indicates mobilised Young's moduli will increase rapidly with increasing depth below foundation	Inappropriate for soft clays. Commonly used method and utilised in commercially available software and simple models in finite element codes. Can be acceptable, when reliable moduli are available from relevant case history data (similar foundation size and stress changes)
(b) Displacement influence factors	Young's modulus and Poisson's ratio	Very simple and rapid method. Published charts often assume uniform modulus and infinite depth; these are unreliable for most practical situations	Methods that allow for increasing stiffness with depth are useful for preliminary design checks. Inappropriate if significant layering below foundation
(c) Stress path method	Young's modulus, Poisson's ratio and correction factors	Theoretically attractive, but overly complex. Complexity not justified by improved reliability	As noted by Burland *et al.* (1977), reliability of method likely to be poor when anisotropy or nonlinearity are significant

Table 53.5 Common methods for calculating the settlement of clays

clays (which are typically anisotropic with higher horizontal than vertical stiffness, and whose stiffness typically increases with depth and which exhibit nonlinear elastic stress–strain behaviour) the one-dimensional method is sufficient for most practical purposes. The principal problem is that the traditional one-dimensional method for overconsolidated clays has relied upon oedometer test data, which often gives excessively high m_v values (and, hence, gross overestimates of the total settlement), due to errors introduced during sampling and testing, and inherent shortcomings in the test method. These problems can be overcome by using a modified one-dimensional method:

$$S_{od} = \left[\sum_{i=0}^{i=n} \left(\Delta\sigma'_{vi} H_i / D'_i\right)\right] f_R f_d \qquad (53.11)$$

where D'_i is the constrained modulus for layer i, $\Delta\sigma'_{vi}$ is the change in the vertical effective stress for layer i and H_i is the thickness of layer i. f_R and f_d are correction factors for the foundation depth and rigidity (section 53.6.7), respectively, and S_{od} is the foundation settlement.

The constrained modulus, D′, can be related to other isotropic elastic parameters as follows:

$$D' = 3K' (1 - \nu') / (1 + \nu') \qquad (53.12)$$

$$D' = E' (1 - \nu') / (1 + \nu') (1 - 2\nu') \qquad (53.13)$$

where K' is the bulk modulus, E' is the drained Young's modulus and ν' is the drained Poisson's ratio. For overconsolidated clays, ν' is close to 0.1 and, therefore, D' is approximately equal to $2.5K'$ or about equal to E'. For the recompression portion of the vertical strain vs log σ'_v oedometer curve (i.e. where the clay is overconsolidated), it can be shown that:

$$D' = \frac{1}{m_v} = \left(1 + e_o\right)\sigma'_{vo} \ln(10) / C_r \qquad (53.14)$$

where e_0 is the initial void ratio, σ_{v0}' is the initial vertical effective stress and C_r is the recompression index.

For anisotropic soils:

$$D' = E'v / [\,1 - 2\,\nu'_{VH}{}^2 (n' / 1 - \nu'_{HH})] \qquad (53.15)$$

where $E'v$ is Young's modulus in the vertical direction, ν'_{VH} is Poisson's ratio for the effect of vertical strain on the horizontal strain, ν'_{HH} is Poisson's ratio for the effect of horizontal strain on the complementary horizontal strain and $n' = E'_H / E'_V$, where E'_H is Young's modulus in the horizontal direction.

The above relationships allow the one-dimensional approach to utilise data from a wide range of different test types in order to assess the ground's compressibility characteristics across a wide range of changes in stress and strain amplitude. For overconsolidated clays:

$$S_{od} = S_T \tag{53.16}$$

where S_T is the total settlement, and:

$$S_{PC} = S_T - S_U \tag{53.17}$$

where S_{PC} is the primary consolidation settlement and S_U is the undrained settlement.

The ratio of the undrained to the total settlement has been studied for a variety of situations (e.g. Burland and Wroth, 1974). As the rate of increase of stiffness with depth increases, the ratio of the undrained to the total settlement decreases. At a typical stiffness increase for London Clay, the ratio of the undrained to the total settlement is about a third. Observations of settlement for numerous structures on overconsolidated clays (London Clay, Lambeth Group and Gault Clay) indicate a typical ratio for the end of construction to the total settlement of about 0.6. Hence, a proportion of the primary consolidation settlement is likely to have taken place rapidly, and undoubtedly the end of construction settlement includes some consolidation settlement, as well as the undrained settlement. Independent analysis (described by Atkinson, 2000) also suggests the end of construction settlement includes some consolidation settlement. Hence, the long-term settlement, which occurs after construction is completed, will often be much lower than indicated by theoretical calculations that assume the clay is undrained during construction. **Table 53.6** summarises several case histories of shallow foundation settlement and this data is plotted on **Figure 53.17**, for both high- and low-plasticity overconsolidated clays. The influence of the plasticity index is clear, with high-plasticity clays exhibiting larger settlements than the low-plasticity Lodgement Tills. The influence of overconsolidation ratio is also significant. The New York Varved Clays are low-plasticity clays and silts, which are only moderately overconsolidated, with overconsolidation ratio (OCR) between 3 and 6, and they exhibit similar settlement to the more heavily overconsolidated clays (typical OCR > 10), which have a high-plasticity index (such as London, Frankfurt and Gault Clays). **Table 53.7** provides a summary of typical index properties for the clays included in **Table 53.6** and **Figure 53.17**.

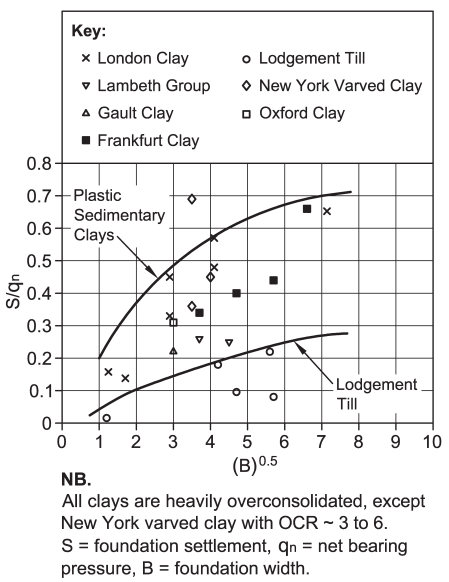


Figure 53.17 Stiff clays, observed settlement ratios versus the square root of foundation width

53.6.5 Settlement of sand

Excessive settlement of sand is seldom a practical problem; most observations of foundation settlement on sands indicate that settlement is less than 40 mm. Large settlements are usually only associated with loose or very loose sands, or sands containing a high organic content or clay layers. In addition the bulk of the settlement takes place during construction, so the post-construction settlement, which may lead to structural damage, is relatively small. Cases of structural damage for foundations on sand are extremely rare. Therefore, the first step for the practising engineer is to qualitatively check if there is likely to be any real settlement problem. **Table 53.8**, based on a study by Clayton (1988), summarises those situations that may lead to significant settlement. For more common situations a simple approach to estimating settlement is often sufficient. **Figure 53.18** is based upon a compilation of field measurements of foundation settlement on silica sand, and three broad categories are given based on a visual description or the average of the standard penetration test (SPT) 'N' values: 'loose' (SPT 'N'<10); 'medium dense' (SPT 'N'=10 to 30); and 'dense' (SPT 'N' > 30). Fairly well-defined upper limits are given for medium-dense and dense silica sand. There is much greater scatter for loose sands, and these soils would usually be considered as unsuitable as founding materials (without prior ground improvement to densify or stiffen the sand). Hence, for many routine structures it is sufficient to use **Table 53.8** in conjunction with **Figure 53.18**, firstly to assess if any problems are likely, and if not use **Figure 53.18** to assess the settlement of medium-dense and dense silica sands. The 'probable' settlement is likely to be about half the upper-limit settlement. Based upon a review of settlement observations, Burland *et al.* (1977) stated that small plate tests (less than 1 m wide) are unlikely to be a reliable basis for assessing the settlement of much wider foundations. This should be borne

Site	Foundation dimensions (m)				Soil profile		Net foundation pressure (kN/m²)	Observed settlement (mm)	
	D	B	L	Type	Type[1]	Thickness (m)		EOC[2]	Final
Clapham Rd	1.5	17	28	Raft	SG	7	190	58	91
					LC	25			
					LG	15			
Hurley Rd	1.5	17	26	Raft	SG	5	190	68	109
					LC	24			
					LG	14			
King Edward	1.7	20	27	Raft	LC	4	210	32	52
					LG	15			
Addiscombe Rd	11	14	14	Raft	LG	5	226	48	59
Waterloo Bridge	7	8.3	36	Bridge piers	LC	25+	290	-	90 to 130[3]
Elstree	2.1	1.5	3.0	Pad footings	LC	15	85	-	14
Bracknell	1.0	3	3	Pad footing	LC	10+	110	-	17
Britannic House	18	27	70	Raft	LC	13	100	15 to 20	30 to 45
					LG	10			
Didcot	6 to 7.5	9	9	Pad footings	GC	30+	180 per footing	20 to 25	30 to 40[5]
New Wembley	1.5	50	50	Embankment	LC	-	180	-	100 –110
					LG				
CN Tower	7.3	32	47	Raft[3]	Till	19	370	20	30
					SG	9			
Toronto Hospital	5	18	70	Raft	Till	12	85	-	15
					Interglacial Till	30+			
Gorlev Silos	4.0	22	22	Raft	Till	-	250	-	18–24
Scotia Centre	7.3	31	32	Raft	Till	12	225	32	49
					Shale	20+			
Dublin	<0.5	1.5	1.5	Pad footing	Till	>20	1010	-	16
Afe, Frankfurt	13	43	43	Raft	FC	100	350	150	230
Library, Frankfurt	8	32	52	Raft	FC	100	45	12	20
University, Frankfurt	7	14	96	Raft	FC	100	200	42	67
Bureau, Frankfurt	7	22	22	Raft	FC	100	250	65	100
Grant, New York	-	12.2	70.2	Raft	MS	9.2	140	43	51
					VSC	27.5			
Madison, New York	-	12.2	55	Raft	MS	4.6	140	71	97
					VSC	19			
Roosevelt, New York	-	16	48	Raft	MS	6.1	180	63	81
					VSC	15			

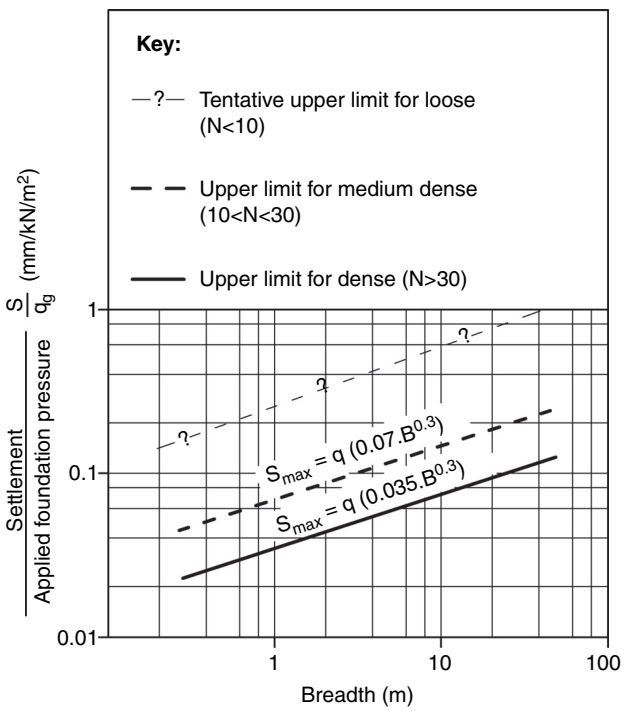
Note:
1. SG is sand/gravel; LC is London Clay; LG is Lambeth Group; GC is Gault Clay; Till is Lodgement Till; FC is Frankfurt Clay; MS is medium sand; VSC is New York Varved silt/clay (Glacial Lake Deposits).
2. End of construction (EOC) settlement as a percentage of total settlement varies between about 50% and 75%; typically it is about 60%. For EOC settlement, construction periods typically last 6 months to two years.
3. Construction problems caused an increase in settlement at some piers.
4. Multiple pad footings at close spacing.
5. References: COSOS (1974), Clark (1997), De Jong and Harris (1971) and Parsons (1976).

Table 53.6 Stiff overconsolidated clays, observed shallow foundation settlement for various sites

Soil type	Moisture content (%)	Plasticity index (%)	Liquidity index	Undrained strength (kN/m²)	Description
Weathered London Clay	30 to 35	50	0 to +0.1	40 to 70	Firm to stiff
Unweathered London Clay	25 to 30	40 to 50	0 to -0.1	70 to 250	Stiff to very stiff
Gault Clay	20 to 30	50	0 to -0.2	100 to 300	Stiff to very stiff becoming hard
Lambeth Group	20 to 30	20 to 50	0 to -0.1	150 to 400	Very stiff to hard
Frankfurt Clay	30 to 35	35 to 55	+0.1 to -0.1	80to 200	Stiff to very stiff
Lodgement Tills	10 to 15	10 to 20	0 to -0.5	150 to 500	Very stiff to hard
New York Varved Clays	30 to 45	5 to 40	+0.1 to +0.3	95 to 140	Firm to stiff varved clay

Notes: For New York Varved Clays, the overconsolidation ratio (OCR) is between 3 and 6; for other clays the typical OCR > 10 at shallow depth.

Table 53.7 Stiff clay case histories – typical index properties



NB.

q_g = gross foundation pressure, S = settlement, N is average SPT 'N' over depth = 1.5B below foundation. Probable settlement is about one half of upper limit. Settlement is 'short-term' settlement, refer to for Burland *et al.* (1984) for assessment of 'creep'.

Figure 53.18 Sands, upper-limit settlement ratios
Reproduced from Burland *et al.* (1977)

Factor	Comment
1. Narrow (<1.5 m), heavily stressed foundations	Carefully check bearing capacity. High risk of failure or low factor of safety, especially if disturbance occurs to sands (reduction in density). Good construction control needed
2. Very wide (>10 m) foundations	Careful assessment of variation of compressibility with depth (e.g. weak layers?)
3. Metastable ('collapsing' sands), e.g. dune sands and lightly cemented sands (e.g. sabkha)	Site geology needs to be well understood. If lightly cemented sands, penetration tests will be misleading. Large plate tests may be needed to assess 'yield' stress of bonded or cemented sand
4. Sands containing layers of organic or clayey material	Static cone penetration tests could identify layers. Sampling and laboratory testing of discrete layers required. SPT's and spaced sampling may miss critical layers
5. Non-silica sands (mineralogy)	Empirical penetration test-based settlement methods are only based on performance of silica sands. Micaceous or calcareous sands, for example, may exhibit far larger settlement. If in doubt, the sand mineralogy should be checked. Specialist technical literature and testing and analytical methods will need to be considered

Table 53.8 High-risk situations for excessive settlement of sands
Reproduced from Clayton (1988)

in mind when assessing the type of testing needed to check the effectiveness of ground improvement methods. Small plate tests are still commonly used as field tests and, although useful as a quality control test, they will not be sufficient to assess full-scale foundation settlement.

When more detailed quantitative methods are needed to calculate the settlement of sands, then the following two methods are recommended:

(i) Burland and Burbidge (1984);

(ii) Schmertmann *et al.* (1978).

Burland and Burbidge (1984) is mainly based on empirical correlations with SPT 'N' values (although a simple correlation is provided between SPT 'N' and CPT q_c values). In contrast Schmertmann makes direct use of CPT q_c data. Both of these methods are well described in several references

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(e.g. Clayton, 1999 and Lunne *et al*., 1997) and will not be repeated in detail here.

Comparative studies indicate that both methods have a similar level of reliability and accuracy. Many other methods of predicting sand settlement have been published. Tan and Duncan (1991) carried out an assessment of the reliability of 12 different methods and showed that some methods can exhibit a systematic bias in settlement predictions. More sophisticated analytical methods (including finite element analyses) do not necessarily improve the reliability of predictions. A prediction symposium (Briaud and Gibbons, 1994) involved making Class A predictions of a footing (3 × 3 m) loaded to 4000 kN (factor of safety of 2.5) for a sand layer with an average SPT 'N' value of 20. A settlement of 14 mm was measured, whereas a sophisticated finite element analysis gave a value of 75 mm.

53.6.6 Settlement of rock

Assessments of foundation settlement on rock routinely utilise linear elastic methods (refer to section 53.6.3); however, the processes and concepts used to assess the deformation characteristics of the rock mass are fundamentally different to those used for soil. A soil mass is usually assumed to be a continuum, whereas for a rock mass the assessment of the nature, orientation and frequency of joints and other discontinuities is absolutely critical, in order to assess the rock mass deformation properties. The orientation of the dominant discontinuity sets in relation to the loading direction can have a significant influence on the deformation behaviour of the rock mass. Based on *in situ* load tests, Barton (1986) defined three characteristic modes of deformation behaviour. Joint surfaces are usually irregular; hence, forces across joints will be transmitted by reduced contact areas. As foundation bearing pressures increase, the contact stresses across the joints may become extremely high, which may lead to local yield and time-dependent creep.

Engineers often erroneously assume that 'rock' will provide a very strong competent foundation; however, this is not always the case. When considering a rock, engineers often visualise strong igneous rocks, such as massive unweathered granite (with widely spaced joints). However, many rocks, especially of sedimentary origin, can be heterogeneous and of very poor quality. Weak rocks may need to be investigated and assessed far more carefully than soil deposits, before an appropriate foundation design can be prepared. A commonly used index to assess rock mass quality is the RQD value and Hobbs (1974) used a 'mass factor' j to relate the modulus for a rock mass, Em, to the intact properties of the rock (refer to section 53.7.4), via RQD values. The modulus for a rock mass is then used in either the classic linear elastic (refer to section 53.6.3) or the modified one-dimensional method (refer to section 53.6.4) to assess settlement. For the vast majority of weak rocks, Young's modulus for a rock mass will increase rapidly with depth, especially beneath weathered material. Meigh (1976) used a linear elastic approach to back analyse the observed settlement of foundations in weak rock.

Three weak rock types commonly encountered across the UK are:

(i) Triassic Series (Mercia Mudstone and Sherwood Sandstone), which outcrops across much of the Midlands, north-west England and parts of south-west England and southern Wales;

(ii) Chalk, which outcrops across much of southern and eastern England and parts of the Yorkshire and Lincolnshire Wolds;

(iii) Carboniferous Coal Measures rocks, which are encountered across many parts of northern England.

For foundation design in these rock types, one of the main challenges is their variability in strength and compressibility, which can change from heavily weathered 'soil' to relatively strong unweathered 'rock'.

A summary of observed settlements for shallow foundations on a number of UK rocks is given in **Table 53.9**. **Figure 53.19** gives a plot of observed settlement ratio against foundation width for structures founded on Mercia Mudstone. For Mercia Mudstone when bearing pressures and foundation widths are modest (pressures less than about 300 kN/m^2, widths less than about 10 m) the observed settlement of an isolated foundation is usually less than about 25 mm, provided weathering grade IV mudstone is largely absent. Settlement occurs mainly during construction, so differential settlement can be largely built out.

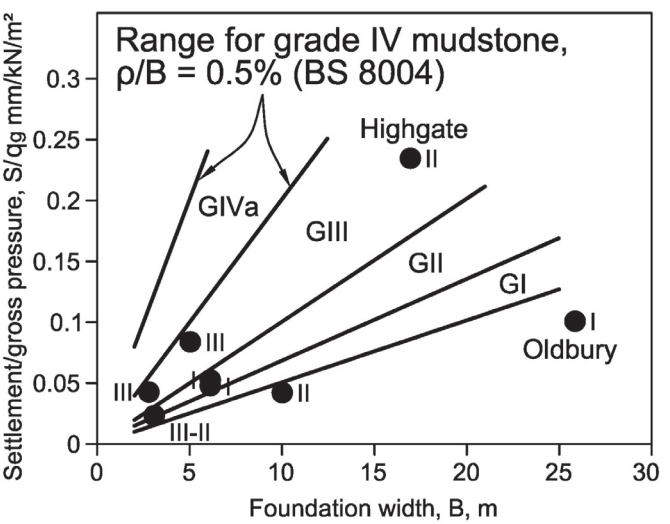


Figure 53.19 Mercia Mudstone, observed settlement ratios versus foundation width
Modified from CIRIA C570, Chandler and Forster, (2002), www.ciria.org

Site	Foundations B (m)	L (m)	Type	Ground profile	Foundation pressure (kN/m2)	Observed settlement (mm)
M1 bridges	6.1	11.6	Pad	Grade 1, Mercia Mudstone	240 (net)	14 to 15
	2.7	41	Strip	Grade 3, Mercia Mudstone	160 (net)	10
34 storey AGT Tower, Edmonton	18	19	Raft	Interbedded bentonitic mudstone (shale) and sandstone	160 (net) 430 (gross)	48[3]
20 storey building (single-level basement)	17	35	Closely spaced pads	Grade 3, Mercia Mudstone (Grade 2 to 1 at depth)	210 (net)	55
Second Severn Crossing Bridge	6	27	Caissons	Grade 2, locally 4, Mercia Mudstone	600 (gross)	<25
Oldbury Nuclear Power Station	26 dia		Circular raft	Grade 1, Mercia Mudstone	1100 (net)	105 to 125[1]
4 silos, Bury St. Edmonds	22.9 dia		4 Circular rafts	Low-density chalk	350	45 to 75
South Abutment Thames Barrier	14.7	31.5	Pad	High-density chalk	480	20
4 storey building, Basingstoke	3.3	3.3	Pad	Low-density chalk	365	7
Bridge, Luton	8.5	20	Pad	Low-density chalk	300	9
Hinckley Point 'B' Nuclear Power Station	19 dia		Circular raft	Interbedded mudstone and limestone, Lower Lias	1200 (net)	Reactor 3,14 to 16[2] Reactor 4, 30 to 35[2]

Notes:
1. End of construction settlement, approximately 80 to 95 mm, subsequent time-dependent settlement (9 year period) described as creep.
2. End of construction settlement, approximately 7 to 8 mm (Reactor 3) and approximately 16 to 18 mm (Reactor 4). Time-dependent settlement took about 10 years to complete.
3. The 13.7 m-deep basement excavation caused 55 to 60 mm heave and swelling of mudstone. End of construction settlement about 35 mm.

Table 53.9 Weak rocks, observed shallow foundation settlement for various sites

Figure 53.20 provides a plot of applied stress versus settlement ratio (settlement divided by foundation width) for structures on chalk. For chalk, foundation settlement increases rapidly once bearing stresses exceed the yield stress, q_y; refer to section 53.7.4. Hence, it is prudent in order to limit settlement to keep the foundation bearing pressure below q_y. At stresses below q_y, an isolated 5 m wide foundation would be expected to settle less than 10 mm (CIRIA C574, Lord *et al.*, 2002), provided weathering grade Dm chalk is absent. For larger foundations or higher foundation bearing pressure, CIRIA C574 provides more detailed guidance on appropriate methods of settlement assessment.

An interesting feature of the case history data is that significant time-dependent settlement can develop once foundation bearing pressures become relatively high. At Oldbury, on unweathered Keuper Sandstone and Siltstone, under a net bearing pressure of 1100 kN/m^2 (compared with UCS values mainly in excess of 10 MN/m^2), post-construction settlement, after a period of about 10 years, had increased by about 50% to 60% compared with the settlement that occurred during construction. The time-dependent settlement was ascribed to creep in a 13 m-thick 'leached' zone (where gypsum nodules had gone into solution, and left a porous sandstone with numerous small cavities) (Meigh, 1976). At Hinkley Point, two large foundations were built on interbedded unweathered mudstone and limestone of the Lower Lias of Jurassic age. The net bearing pressure was about 2000 kN/m^2 for both foundations (less than 10% of the UCS for the moderately strong rock). For Reactor 4, there was negligible post-construction settlement, whereas for Reactor 3 settlement increased by about 50% over a 12 year period and the final settlement was about double that of Reactor 4, which was in similar rock. The much greater settlement of Reactor 3 was ascribed to ineffective dewatering of the high groundwater pressures, which caused joint opening in the fractured limestone and softening of the unloaded mudstones (Haydon and Hobbs, 1974). The settlement of Reactor 3 was about three times larger than that originally predicted.

53.6.7 Corrections for foundation rigidity and foundation depth

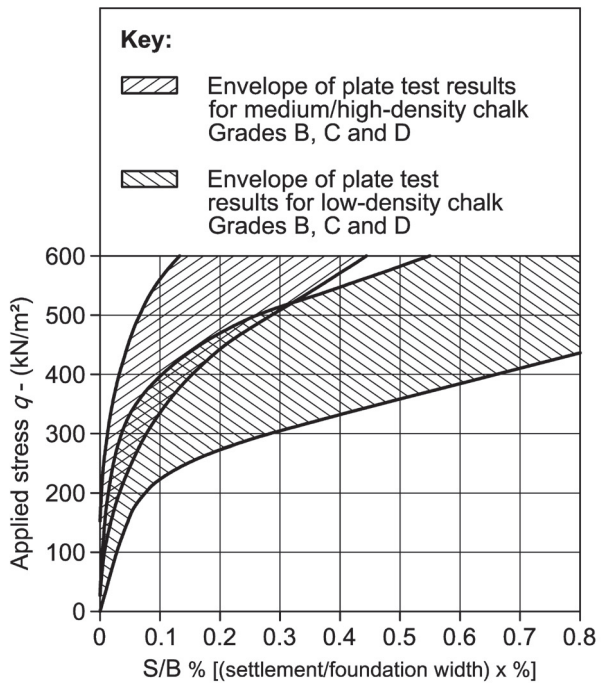
The classic one-dimensional and elasticity equations are based on a flexible foundation resting on the surface of a compressible medium. Hence, corrections are necessary for:

(i) the actual rigidity of the foundation;

(ii) the actual depth of the foundation below the surface of competent ground.

The settlement of a rigid footing is usually assumed to be 0.8 times that of a flexible footing. Horikoshi and Randolph (1997) provide a rigidity correction factor that can be applied to the full range of foundation rigidities (between perfectly rigid and flexible) and geometries (from circles, squares to strips).

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NB.
Observed settlements of foundations on chalk fall within above envelope, if appropriate shape correction factors applied, refer to CIRIA C574.

Figure 53.20 Chalk applied stress – settlement ratios
Reproduced from CIRIA C574, Lord *et al.* (2002), www.ciria.org

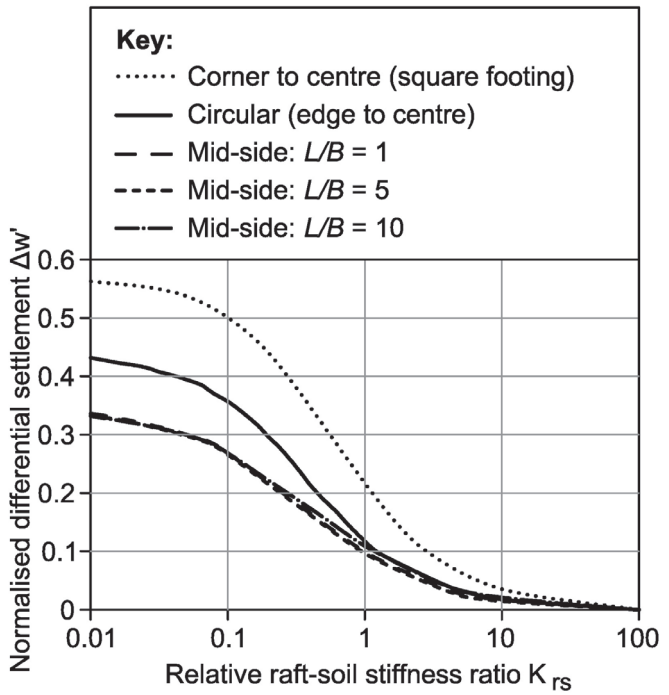


Figure 53.21 Normalised differential settlement as a function of raft stiffness ratio

Figure 53.21 gives a plot of the normalised differential settlement versus the relative raft–soil stiffness ratio K_{rs}:

$$K_{rs} = 5.57\,(E_f / E_{Sav})\,[(1 - \nu_s^2)/(1 - \nu_f^2)]\,(B/L)^{0.5}\,(t_f/L)^3 \quad (53.18)$$

$$\Delta S^* = (S_A - S_B) / S_{av} \quad (53.19)$$

where S_A is the settlement at the centre, S_B is the settlement at the middle of the short side of a footing or the settlement of a corner (**Figure 53.21**), S_{av} is the average settlement of a footing, E_f is Young's modulus for the footings or raft material (e.g. reinforced concrete), E_{Sav} is the representative Young's modulus for the ground beneath the foundation (i.e. the value of E_s at a depth equal to the radius of a circular footing or half the foundation width), t_f is the footing thickness, B is the footing width, L is the footing length, ν_s is Poisson's ratio for the soil and ν_f is Poisson's ratio for the footing material.

To use equations (53.18) and (53.19), it is most convenient to assume that the foundation is initially rigid (i.e. apply a correction factor of 0.8) and then modify the results in accordance with **Figure 53.21** and equations (53.18) and (53.19). It is apparent from the figure that foundation rigidity changes rapidly with changes in the ratio t_f/L. Hence, many foundations will either be practically flexible (K_{rs} <0.1) or practically rigid (K_{rs} >5).

The effect of foundation embedment on reducing settlement is often overestimated, in particular Fox's depth correction factor (given in many textbooks) significantly overpredicts the influence of embedment depth for shallow foundations. The correction factor given by Mayne and Poulos (1999) is more realistic.

53.6.8 Heave and swelling

When the ground is unloaded beneath an excavation, undrained heave will develop and additional upward movement can also develop due to time-dependent swelling. For sands and gravels, heave and swelling are assumed to be negligible; however, for clays and argillaceous rocks (particularly mudstone, shale, etc.) heave and swelling can be significant. Most of the published data are for undrained heave of medium- to high-plasticity overconsolidated clays. Vertical deformations due to heave and swelling of the order of 60 mm and 100 mm have been measured beneath 14 m and 20 m deep excavations, respectively, in bentonitic shale (e.g. Chan and Morgenstern, 1987 and De Jong and Morgenstern, 1973). Interbedded layers of water-bearing limestone facilitate rapid swelling of Frankfurt Clay and movements of up to 140 mm have been reported for 25-m-deep excavations (Breth and Amann, 1974).

The mechanisms associated with heave and swelling of overconsolidated fissured clays and mudstone or shales are more complex than those associated with settlement. Some shales and mudstones that contain active clay minerals (e.g. montmorillonite) can swell considerably following stress release due to water absorption by the clay minerals, together with exposure to air. Weathering of iron sulphide minerals (e.g. pyrite) accelerated by oxidising bacteria can also occur (often associated with lowering of the water table), leading to additional volumetric expansion. Terzaghi *et al*. (1996) provide a more detailed discussion of the above mechanisms, which can lead to a progressive weakening of the soil or rock mass. This can mean that if the ground mass at the base of an excavation is exposed for prolonged periods of time, then it is likely to deteriorate. The subsequent settlement of shallow foundations will then be much larger than would be expected, based on the 'undisturbed' ground properties. Hence, it is extremely important that the ground surface at foundation level is protected as much as practical, refer to section 53.3.4.

53.6.9 Simplified nonlinear methods and numerical modelling

Since the mid-1980s, there have been numerous research studies and published papers on the nonlinear stiffness characteristics of soils and weak rocks. Nevertheless, routine analytical methods and commonly used stress–strain models within commercially available finite element and finite difference software usually assume linear elastic, or linear elastic-perfectly plastic behaviour. Some sophisticated nonlinear models are used in industry, but these are very complex and difficult to use correctly. Highly trained specialists are needed to run the software and a high level of expertise is needed to define the model inputs and review the output, in order to obtain reliable predictions. For foundation design, sophisticated modelling is rarely required; however, there are situations when engineers may need to consider the influence of nonlinear ground behaviour in more detail, as discussed below. Some simplified nonlinear methods have been outlined in the literature, and these are summarised in **Table 53.10**. Each method has its own intrinsic advantages and disadvantages, although in general there is still very limited experience in their use across the industry. Hence, if these techniques are used, they need to be used with some caution and under the guidance of a suitably experienced specialist. One of the main benefits of these methods is that the variation in settlement with changes in net bearing pressure can be assessed. Nonlinear methods may be useful when the following are being assessed:

(i) the interaction between adjacent structures;

(ii) differential settlement adjacent to, or beneath, a shallow foundation;

Method	Type of analysis	Nonlinear parameters	Foundation geometry	Soil layering	Comments
Stroud (1990)	Elasticity, plus bearing capacity	Normalised Young's modulus versus degree of loading	Isolated strip, circular pad or footing	No. A single 'representative' Young's Modulus is used	Bearing-capacity theory used to assess 'degree of loading' (inverse of factor of safety), then empirically derived correlation with normalised Young's modulus (E'/N or E'/S_u). Conventional elastic equations used to assess foundation settlement. Drained (total) settlement of clays or sands can be calculated
Osman and Bolton (2005)	Plasticity theory	Mobilised undrained strength versus shear strain (linked to applied load and foundation settlement via scaling factor)	Isolated circular footing on clay	No. A single 'representative' stress–strain curve is used	Footing load-settlement behaviour is linked to triaxial stress–strain curve, via bearing-capacity theory and scaling factors to link triaxial shear strain to footing settlement. Can also be used to assess horizontal and rotational deformation of foundation under horizontal and moment loads. Undrained deformation of clays only
Atkinson (2000)	Elasticity theory	Normalised Young's modulus (E_s/E_0) versus axial strain (linked to applied load and foundation settlement via scaling factor)	Isolated circular, strip or pad footings	No. A single 'representative' stress–strain curve is used	Footing load-settlement behaviour is linked to triaxial stress–strain curve, via elasticity theory and scaling factors to link triaxial strain to footing settlement. Can also be used to assess horizontal and rotational deformation of foundation under horizontal and moment loads. Undrained and total settlement of clays; total settlement of sands
O'Brien and Sharp (2001)	Modified one-dimensional method	Constrained modulus versus axial (vertical) strain	Any geometry, both isolated and multiple closely spaced footings	Yes. Multiple layering and variations of stiffness with depth	Elastic theory used to calculate vertical stress changes. Constrained moduli at various stress and strain amplitudes can be assessed from oedometer, triaxial and geophysics data, or empirically. Input data also needed for in situ mean effective stress and variation of stiffness parameters with depth. Total settlement of clays only

Table 53.10 Simplified nonlinear methods for foundation settlement

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Issues	Comments
1. What is the key objective of numerical modelling, e.g. vertical or horizontal movements, structural forces or stresses?	A numerical model can produce a plethora of different outputs. Intrinsic limitations of most constitutive models mean that different outputs from a particular analysis will be of variable reliability.
2. What relevant case histories are available for the calibration of the model?	The use of uncalibrated models is poor practice and can provide highly misleading results.
3. Is the ground investigation adequate enough to supply appropriate parameters?	There is no value in carrying out sophisticated modelling in the absence of a good quality ground investigation. Specific testing may be needed to provide appropriate input parameters.
4. How will key input parameters be checked, e.g. strength and compressibility (which is more important as the stress–strain model becomes more complex)?	It is important to run computer simulations of 'element' behaviour under relevant drainage conditions and stress paths, e.g. undrained or drained strength in triaxial compression or extension, compression and swelling of oedometer; and to compare against laboratory data. Influence of sampling disturbance? Differences between field and lab data?
5. Will groundwater flow or seepage influence behaviour ('undrained' analyses may be unrealistic)?	Below the water table, in permeable horizons, local drainage and consolidation may have a marked effect on behaviour, necessitating coupled analyses.
6. What is the construction sequence, are there other miscellaneous effects?	The construction sequence can significantly influence many ground–structure interaction problems; hence realistic sequences need to be developed. Construction effects, such as vibration, may be important in sandy soils.

Table 53.11 Numerical modelling: some questions to consider

(iii) the potential for a more economic foundation design is being considered, e.g. factors of safety against bearing-capacity failure of less than three;

(iv) settlement and differential settlement of sensitive structures (say, a limiting settlement of less than 25 mm);

(v) negative skin friction and displacement of 'floating' piles in stiff clays (refer to Chapter 57 *Global ground movements and effects on piles*);

(vi) heave-induced tension, and displacement, of piles beneath excavations in stiff clay (refer to Chapter 57 *Global ground movements and effects on piles*).

One of the important practical consequences of soil nonlinearity is that subsurface settlements and settlements adjacent to a foundation become concentrated much closer to the foundation than indicated by linear elastic predictions (Jardine *et al.*, 1986). A practical application of the modified one-dimensional method is outlined in a short case history in section 53.8 below, which facilitated foundation re-use for a new settlement-sensitive structure. When numerical modelling is used, the questions in **Table 53.11** need to be considered very carefully.

53.6.10 Unsaturated soils and non-engineered fills

Deep deposits of unsaturated natural soils are commonly encountered across many parts of the world (such as Central Asia, South Africa, the Americas, etc.), although, in the context of foundation design, they are rarely a serious concern in temperate climate zones such as the UK. For shallow foundations, the principal concern is the metastable volume-change potential of unsaturated soils; see **Figure 53.22**. Below the threshold stress, if moisture content (or pore water pressure) increases then heave can develop, whereas above the threshold stress, large collapse-type settlement will occur if moisture content (or pore water pressure) increases. These movements can develop quite rapidly, as the moisture content increases, and have caused substantial structural damage. The assessment of unsaturated soil behaviour is a specialist task. Fredlund and Rahardjo (1993) provide detailed guidance on the theoretical and practical issues.

In the UK, the practical foundation problems associated with unsaturated ground are usually associated with non-engineered fills (discussed in detail in Chapters 34 *Non-engineered fills* and 58 *Building on fills*) or with clays that have been desiccated by vegetation (discussed in section 53.2 above).

53.7 Information requirements and parameter selection

53.7.1 Ground profile and site history

Many of the serious problems that occur with shallow foundations are due to a misinterpretation of the site's history or geology, which means that soft compressible soils are found at foundation level rather than competent low-compressibility soils. Two simple examples are given below:

(i) A low-rise building was to be founded on shallow strip footings. The local geology was Bagshot Beds (interbedded stiff clay and dense sands) over stiff London Clay. Boreholes had been sunk in the vicinity of the building. During initial excavations for the strip footings very soft to soft black sandy clay was encountered, rather than the stiff clays and dense sands originally anticipated.

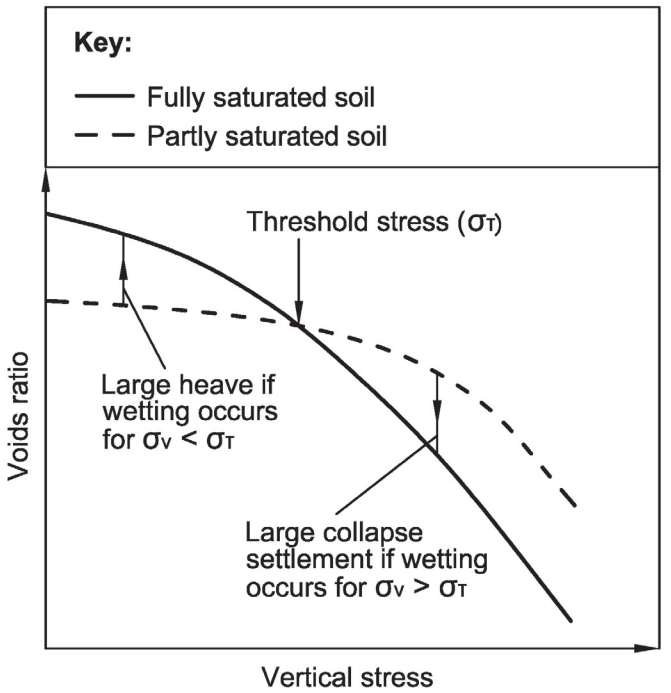


Figure 53.22 Metastable settlement and heave behaviour for partly saturated soils

Excavations down to 5 m depth did not find any improvement in strength. The original shallow foundation design had to be abandoned and replaced by piled foundations, with substantial cost increases and a large delay to overall completion of the project. Subsequent investigations revealed that the area had been used as a brick pit; the natural competent soils had been excavated and stockpiled. Once brick-making had finished the natural soils were end-tipped loosely into the pit. The borehole logs had proven to be completely unreliable.

(ii) Several low-rise buildings were successfully designed and constructed, with shallow strip footings, and an additional building was then commissioned close to the site perimeter. The location for the proposed building was being used for agricultural purposes, and intrusive ground investigations could not be carried out due to access and land ownership problems. No problems were anticipated due to the positive experience from the previous works. Once access into the area was achieved, construction commenced straight away. Unfortunately excavations for the strip footings only found very soft clays. Piled foundations were eventually used, instead of the original strip footings. Aerial photographs of the area indicated the existence of an old pond, which had subsequently been infilled.

With the benefit of hindsight it may seem surprising that these problems occurred, especially since experienced designers had been involved in both the sites. Desk studies and ground investigations must be implemented with great care and thoroughly checked. Even if old investigation data are available for a site, it is prudent to carry out some additional investigations to check the reliability of the old data. It is important that investigations are fully supervised by experienced staff.

The scope and scale of ground investigations will vary depending upon the size and complexity of the proposed structures, local experience and the nature of the geomorphology, geology and hydrogeology in the vicinity of the site. A comprehensive desk study is always required. The objectives of the subsequent investigations will include:

- confirmation of the conceptual ground and groundwater models derived from the desk study;
- assessment of the local variability of the ground properties;
- derivation of simple strength and compressibility parameters (often from empirical correlations with penetration tests or basic laboratory testing);
- assessment of likely construction problems (especially when groundwater related).

For simple (Category 1) projects (with shallow pad or strip footings) in an area with well-known geology, it may be sufficient to carry out trial pits, dynamic probing or static cone penetration tests. For Category 2 and 3 projects a broader range of more sophisticated *in situ* and laboratory tests will be necessary. In general, for sites underlain by clays, the emphasis for carrying out laboratory testing of strength and compressibility should be a small number of tests on high-quality samples (push-in thin wall tubes, triple-tube rotary coring with foam flush for stiff clays or push-in piston tubes for soft clays), rather than carrying out a mass of routine triaxial and oedometer tests on relatively poor-quality driven thick-walled U100 tubes. The case history in section 53.8 highlights the advantages of this approach.

For sands, if the foundation requirements are particularly onerous, the sands have unusual mineralogy (micaceous or calcareous) or contain significant clay or organic content then laboratory testing of reconstituted samples for strength and compressibility should be considered. As noted below (section 53.7.3) empirical correlations between penetration resistance and sand compressibility are of limited reliability. More sophisticated testing, including plate tests or pressure-meter tests, may need to be considered for complex situations.

The assessment of groundwater conditions will usually require piezometers to be installed in boreholes. The groundwater levels should be monitored over several months (preferably including a winter and spring period). *In situ* permeability tests should also be carried out in more permeable layers that may be the source of groundwater problems during construction.

For all projects, irrespective of their scale and complexity, the ground investigation should include both *in situ* testing, sampling and laboratory testing. They are complementary

techniques, with their own particular advantages. *In situ* testing is intrinsically better suited to assessing ground variability, whereas laboratory testing is better suited to assessing fundamental strength and compressibility characteristics provided the sample quality is sufficiently high. It is always important to sample and inspect the soil visually and describe it in a comprehensive and consistent manner using engineering descriptions consistent with appropriate codes (Rowe, 1972).

A major deficiency with conventional practice is that sampling is usually carried out at discrete depth intervals that are too large, typically 1.5 m depth intervals. Most foundation settlement will be due to settlement occurring within a depth of about two-thirds of the foundation width. Hence, *continuous* sampling of boreholes is recommended, down to a depth of about 1.0*B*, where *B* is the proposed foundation width. Laboratory testing and penetration tests should be particularly intensive within a depth of about two-thirds of the proposed foundation width.

Much of the UK is underlain by glacial soils; these soils are often competent and suitable for shallow foundations, indeed piling can often be far more problematic; refer to CIRIA C504 (Trenter, 1999); however, investigations can be challenging due to the potential presence of:

- significant spatial variability in soil type (exhibited by large changes in strength, overconsolidation ratio, compressibility and consolidation properties);
- large variations in depth to rockhead and associated variations in the thickness of overlying compressible soils;
- water-bearing layers of silt, sand, gravel of variable thickness and continuity;
- artesian groundwater pressures;
- cobbles and large boulders.

Figure 53.23 shows a profile through glacial deposits in northeast England, where a weaker, lightly overconsolidated, glaciolacustrine layer underlies a very stiff heavily overconsolidated lodgement till. In these ground conditions the full range of ground investigation methods may be necessary in order to adequately characterise the ground conditions.

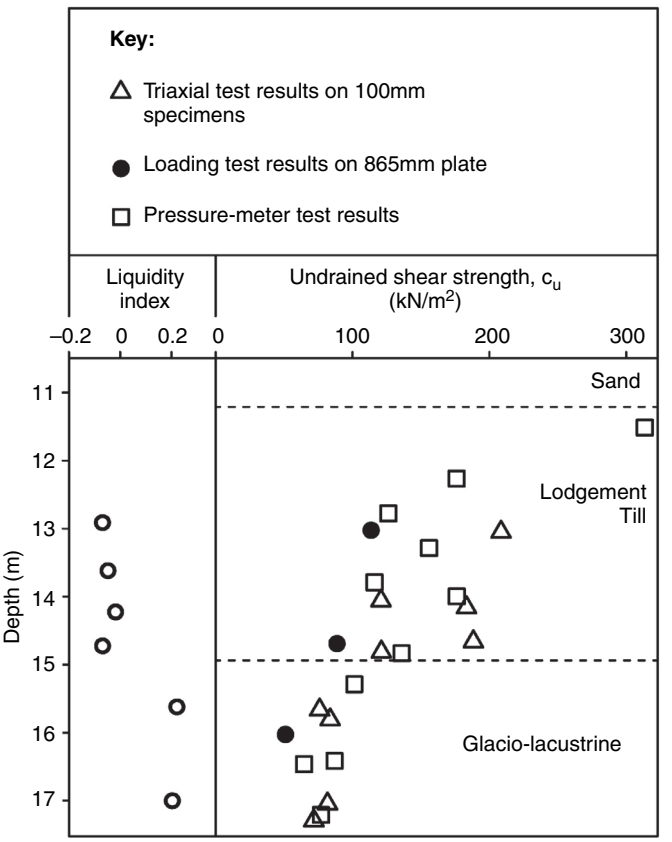


Figure 53.23 Glacial deposits, weaker layer beneath stiff clay
Reproduced from CIRIA C504, Trenter (1999), www.ciria.org

53.7.2 Strength and compressibility of clays

Classification: As noted in section 53.6.4 (**Tables 53.6** and **53.7** and **Figure 53.17**) the observed settlement of shallow foundations is closely linked to the plasticity index (PI) and degree of overconsolidation (OCR) of the underlying clays. The shear strength of clays are similarly dependent on PI and OCR, and are also significantly affected by their macrofabric and structure (i.e. the presence of silt and sand lenses, fissures and bonding). Classification test data can be used to derive void indices (Burland, 1990) and compare these against intrinsic and sedimentation compression lines (ICL and SCL, respectively); refer to Chapter 52 *Foundation types and conceptual design principles* and **Figure 52.21**. Void index plots can provide a robust framework for assessing settlement behaviour, especially for soft clays.

Reference should be made to Chapter 17 *Strength and deformation behaviour of soils* for an introduction to the strength and deformation behaviour of clays.

Heavily overconsolidated clays – undrained strength (Su): The common method in the UK for measuring strength is to sample with driven thick-walled tubes (U100) and carry out quick undrained triaxial (QUT) tests (time to failure of about 2–5 min). This is a very crude method and data usually exhibit significant scatter. **Figure 53.24** gives examples of the differences between different test data. For Lodgement Till, the plate tests indicate an increase of strength with depth, whereas the U100 QUT data indicate a reduction of strength with depth. The misidentification of soft layers can be a common problem, especially in clays with silt or sand lenses. This is due to the clay softening after sampling as water migrates from the silt and sand to the adjacent clay. *In situ* tests (such as CPT or SPT) can be a valuable means of identifying more realistic strength profiles. SPT is a simple and robust means of assessing undrained strength for a wide range of stiff clays. Stroud (1990) provides further guidance; with Su ~ *fi* (N), where N is the SPT blow count and *fi* is a correlation factor, which varies

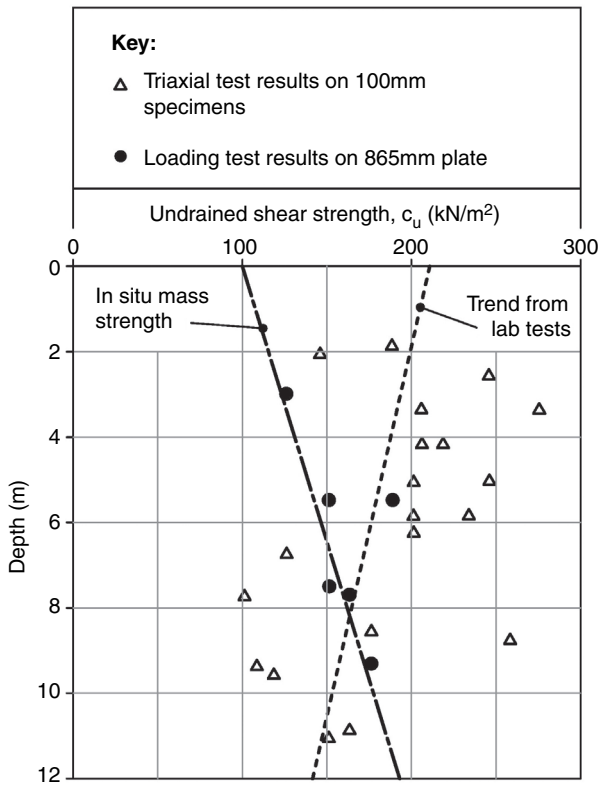


Figure 53.24 Undrained shear strength data for a stiff clay

from about 4 to 4.5 for high-plasticity overconsolidated clays to about 5.5 to 6 in low-plasticity overconsolidated clays. Lunne *et al*. (1997) give guidance on deriving the undrained strength from CPT's in stiff clays, where the correlation is dependent mainly on fissure spacing.

If the project is relatively simple and a high factor of safety (say, 3.0 or more) against bearing-capacity failure is used, then the potential errors introduced by routine sampling and testing will usually be of little practical consequence. However, if a more economical design is being developed with a lower factor of safety (e.g. Eurocode 7 allows a factor of safety on undrained strength of 1.4 to be used), then the potential errors can become critical and high-quality sampling together with more sophisticated testing is strongly recommended.

Heavily overconsolidated clays – compressibility: When carrying out preliminary estimates of total settlement and using linear elastic analytical methods, then (based on Stroud, 1990):

$$E'_v = 200S_{um} \text{ for high-plasticity clays} \quad (53.20)$$

$$E'_v = 250S_{um} \text{ for low-plasticity clays} \quad (53.21)$$

where $E'v$ is the drained Young's modulus for vertical loading and S_{um} is a cautious estimate of the mean undrained strength profile.

It is now well known that the deformation modulus (or compressibility) of soils is highly nonlinear, and may change by more than an order of magnitude between very small strain and large strain amplitudes. Hence, the confusing range of soil stiffness correlations quoted in various technical publications. Research has shown that the degradation of deformation modulus, with increases in strain amplitude or shear stress, forms a consistent pattern; **Figure 53.25** plots data for a wide range of soil types under undrained and drained loading. Atkinson (2000) and Clayton and Heymann (2001) give additional information on this topic. **Figure 53.26** plots drained Young's moduli from oedometer and geophysical tests (i.e. at large and very small strain, respectively) for a site in central London (QE2 Conference Centre, Burland and Kalra, 1986). Also shown are mobilised Young's moduli back analysed from observations of foundation settlement. It can be seen that at shallow depths the mobilised stiffness tends towards large strain values (from oedometer tests), but rapidly increases with depth towards very small strain values (from geophysical tests). Also shown is the Young's modulus profile based on equation (53.20) above, which underestimates the rate of increase of mobilised stiffness with depth.

53.7.3 Strength and compressibility of sand

Classification: To assess the strength and compressibility of sands the following basic information will be required:

- geological age (e.g. post-glacial, glacial or pre-glacial) and likely OCR (overconsolidated or normally consolidated);
- particle grading and particle angularity;

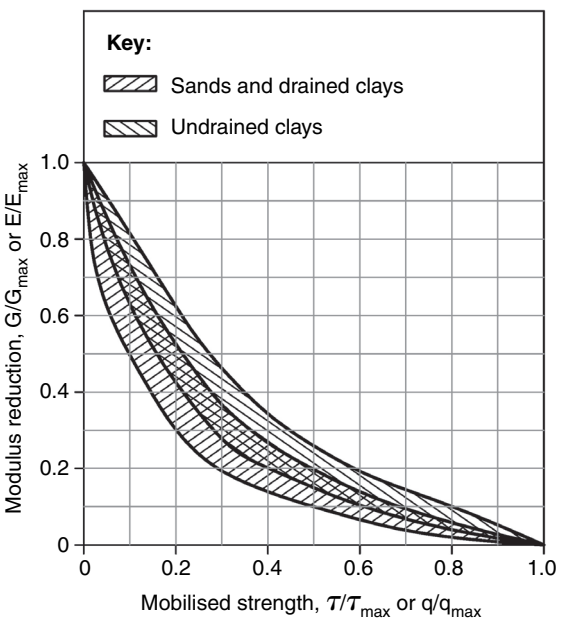
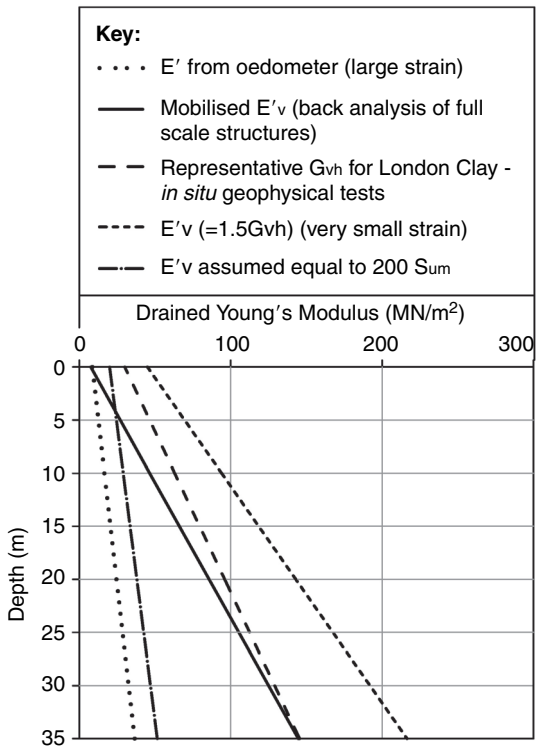


Figure 53.25 Modulus reduction versus mobilised strength for clays and sands
Modified from Mayne (2007)

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NB.
Mobilised E′v and oedometer E′v (after Burland and Karla, 1986).
Representative Gvh for London Clay (derived from Hight *et al.* 2003).
Gvh = vertical shear modulus;
Sum = mean undrained strength.

Figure 53.26 Drained Young's moduli versus depth for a stiff plastic clay

- mineralogy (silica or non-silica);
- relative density.

Clayton (1995) and Lunne *et al.* (1997) provide guidance on the derivation of relative density from SPT 'N' and CPT q_c values, respectively.

Chapter 17 *Strength and deformation behaviour of soils* gives an introduction to the strength and deformation behaviour of sands.

Strength: A simple and coherent approach to assessing the strength of sands has been described by Bolton (1986):

$$\text{Peak angle of friction, } \Phi'_p = \Phi'_{cv} + \psi_d \quad (53.22)$$

$$\text{For triaxial conditions, } \psi_d = \Phi'_p - \Phi'_{cv} = 3I_R \quad (53.23)$$

$$\text{and } I_R = I_D\,(Q - \ln p') - 1 \quad (53.24)$$

where Φ'_{cv} is the critical state angle of friction, which is mainly dependent on particle shape, grading and mineralogy (refer to **Table 53.12**) and ψ is the dilatancy of the sand, which is mainly dependent on the relative density and the stress level at failure. I_R is a relative density index, with values between zero and 4. I_D is the relative density, p' is the mean effective stress at failure (for $p' \geq 150$ kN/m^2) and Q depends on the strength or 'crushability' of the grains (refer to **Table 53.13**).

Dilatancy as a function of relative density and mean effective stress is given in **Figure 53.27**, for quartz and feldspar sand. For micaceous and calcareous sands, dilatancy will be much reduced, especially at high stress, since Q is lower (i.e. the particles are more easily crushed).

In general, making direct estimates of peak angles of friction from penetration tests is NOT recommended. An assessment of the critical state angle of friction should be made based on: the sand mineralogy (e.g. from geological memoirs), visual description of particle angularity and particle grading and by reference to **Table 53.12**. The dilatancy component of the strength can be assessed from equations (53.23) and (53.24) above (or **Figure 53.27**). An appropriate use of penetration test data is to assess the dilatancy component of the strength, from an assessment of relative density (i.e. I_D in equation (53.24) and **Figure 53.27**). If necessary, appropriate laboratory tests can be carried out (Bolton, 1986). A cautious estimate of dilatancy is required. However, for dense sands it would be overconservative to ignore dilatancy completely.

Compressibility: It is recommended that the original paper by Burland and Burbidge is reviewed together with the subsequent published discussion (Proceedings of ICE, December, 1986), which provide insights into the various issues that can affect the settlement of sand. It is important for the engineer to appreciate the limitations of any method of settlement prediction that is based on the results of penetration tests. In particular it should be noted that the sand's stress history has a significant influence on its compressibility, but has a negligible influence on penetration resistance. In addition to any intrinsic problems with penetration tests the natural variability of sands tends to be high. **Figure 53.28** gives the results of a statistical study by Burland and Burbidge (1984) for 13 sites. This shows that the ratio of the maximum to the minimum settlement of practically identical footings (subject to the same loads) is large, and a ratio of maximum to average settlement of 1.6 is a reasonable bound to the data spread.

If it is necessary to derive Young's moduli from SPT 'N' values, then **Table 53.14** provides some guidance, and Lunne *et al.* (1997) also provide guidance for interpretation of CPT data (for E' mobilised at strain amplitudes of about 0.1%, and the variation of Young's moduli with degree of loading, for various CPT q_c values). Many of the old published correlations for relating penetration test values to Young's moduli are unreliable.

53.7.4 Strength and compressibility of rocks

Classification: Assessing the strength and compressibility of rocks is predominantly concerned with determining the discontinuity characteristics and degree of weathering of the rock. If the rockhead is close to the ground surface, then trial pits or trial trenches will be valuable for assessing the rock mass characteristics; for deeper deposits, large diameter (P

Sand type	Particle size[1]	Mineralogy[2]	Particle shape[3]	D50 (mm)	D10 (mm)	Unif coeff.	emax	emin	Φ'_{cv}
Ottawa	c	q	well rnd	0.75	0.65	1.2	0.8	0.49	29.5
Ottawa	m	q	rnd	0.53	0.35	1.7	0.79	0.49	30.0
Chattahoochee	m	q	s ang	0.37	0.17	2.5	1.10	0.61	32.5
Mol	f–m	q	s rnd	0.19	0.14	1.5	0.89	0.56	32.5
Ticino	c	q	s rnd	0.53	0.36	1.6	0.89	0.6	31.0
Sacramento	f–m	q+f	s ang/s rnd	0.22	0.15	1.5	1.03	0.61	33.3
Reid Bedford	f–m	q+sf	s ang	0.24	0.16	1.6	0.87	0.55	32.0
Hokksund	c	q+f	s ang	0.39	0.21	2.0	0.91	0.55	32.0
Toyoura	f	q	s ang	0.16	0.11	1.5	0.98	0.61	32.0
Mersey	f–m	q	s ang/s rnd		0.1	2.0	0.82	0.49	32.0
Milton Mines	f–m	q+f	ang	0.2	0.11	2.0	1.05	0.62	35.0
Southport	f–m		ang	0.2	0.12	1.8	0.88	0.53	35.0
Crushed quartz	f	q	v ang	0.12	0.07	2.0	1.15	0.55	36.4
Crushed feldspar	f	f	v ang	0.12	0.07	2.0	1.21	0.49	38.7
River sand and gravel	37 mm to f sand	f+q	s rnd/s ang	4.8	0.6	8			35.0
Glacial outwash sand	f–c		s ang	0.75	0.15	6	0.84	0.41	37.0
San Francisco	50 mm to fines	basalt	ang						38.0
Furnas Dam	10 mm to fines	quartzite							39.0
Granite gneiss	37 to 4 mm		ang						40.8

Notes:
1. Particle size; f= fine, m=medium, c=coarse.
2. Mineralogy: f=feldspar, q=quartz, s=some.
3. Particle shape: rnd=rounded, s ang=sub-angular, s rnd=sub-rounded, v ang=very angular.
Based on Stroud (1990) and Bolton (1986)

Table 53.12 Critical state angle of friction, Φ'_{cv}, for some sands and gravels

Sand mineralogy	Q
Quartz and feldspar	10
Limestone	8
Quartz and glauconite	7.8
Calcareous	7.5
Chalk	5.5

Notes:
1. From Bolton (1986) and Randolph *et al.* (2004).
2. *Q* can be assumed equal to the natural logarithm of the particle crushing strength in kN/m^2.

Table 53.13 Crushability Q for some sands

or S size core, i.e. 90 to 100 mm diameter) triple-tube coring is recommended, together with logging by an experienced geologist. A common error is to carry out rotary coring with too small a diameter, which can be highly misleading. The initial steps in assessing the rock mass properties typically comprise:

(i) Classify the rock mass with respect to its geological origin (i.e. igneous, metamorphic or sedimentary).

(ii) Assess the rock mass quality by assessing the nature, orientation and spacing of all discontinuities and joints, including if the joints are open or closed, the strength and compressibility of discontinuities and any infill (which may include soft clay) in the joints. Practical applications normally utilise a rock mass classification system (such as the Geological Strength Index, GSI, Hoek *et al.*, 1995).

(iii) Assess the weathering grade of the rock; a number of weathering schemes have been developed following Chandler (1969), although no weathering scheme can be universally applied. In complex interlayered sequences (say of interbedded mudstone and sandstone) more weathered material may underlie less weathered material.

A general problem with very weak and weak rocks is that poorer zones of rock are often not sampled. Hence, reliance on laboratory tests can lead to an unsafe bias in the data on strength and compressibility. Hence, *in situ* tests, such as weak rock self-boring pressure-meter and geophysics, can be valuable.

Strength: The discontinuity characteristics, orientation and spacing need to be understood. Investigation techniques are discussed by Wylie (1991).

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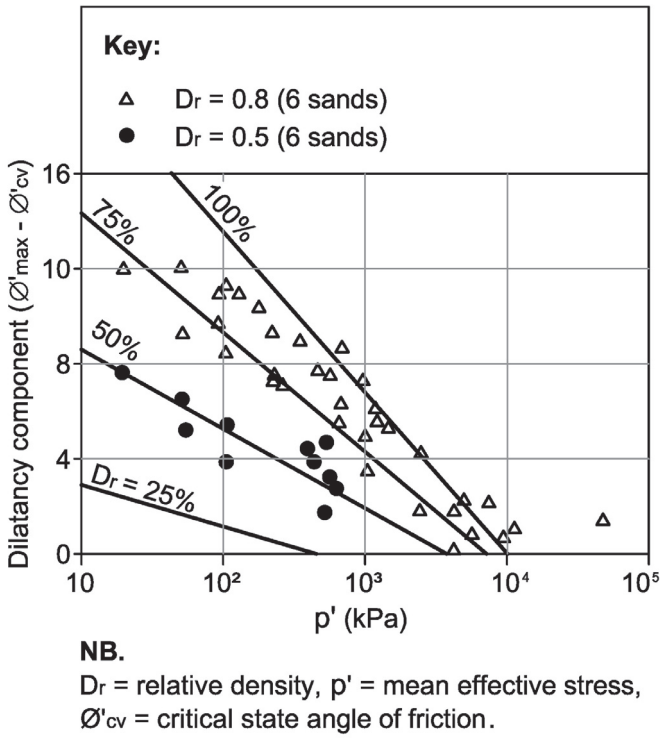


Figure 53.27 Variation of dilation (Φ′max – Φ′cv) versus relative density and mean effective stress
Data taken from Bolton (1986)

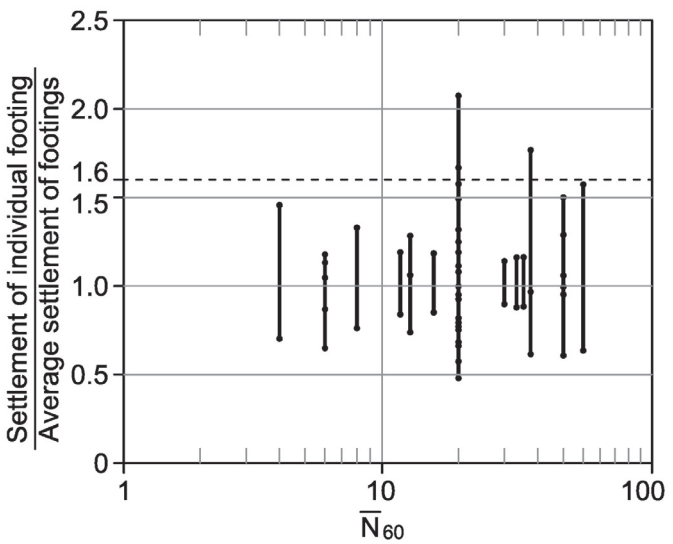


Figure 53.28 Variability of sand. Observed variations of footing settlement at a site
Reproduced from Burland and Burbidge (1984)

Standard penetration value (blows/300 mm), N	E′/N (MN/m²) at Lower limit (large strain)	Mean (small strain)	Upper limit (2) (very small strain)
< 4	0.4 to 0.6	1.6 to 2.4	3.5 to 5.3
10	0.7 to 1.1	2.2 to 3.4	4.6 to 7.0
30	1.5 to 2.2	3.7 to 5.6	6.6 to 10.0
> 60	2.3 to 3.5	4.6 to 7.0	8.9 to 13.5

Note:
1. Based on back analysis of Burland and Burbidge (1984) database and for initial guidance only
2. The upper-limit values should not be used in routine practice, site-specific verification required via geophysical or other independent tests or correlations
3. *E′* = 'N' and *E′* = 2'N' are often used in UK practice for normally and overconsolidated sands respectively.
4. For important projects, site-specific testing may be required.

Table 53.14 Correlation between SPT 'N' and Young's modulus
Data taken from Clayton (1995)

For a wide range of rocks the Hoek–Brown failure criterion (Hoek and Brown, 1997) provides a coherent and practical approach to assessing appropriate strength parameters. CIRIA reports C574 (Lord *et al.*, 2002) and C570 (Chandler and Forster, 2001) provide specific guidance on Chalk and Mercia Mudstone respectively.

Compressibility: As indicated by **Figure 53.29** the compressibility of a rock mass (due to the influence of discontinuities) can be far higher than the compressibility of the intact material. This figure compares the intact stiffness of chalk in the laboratory with the mass stiffness derived from surface-wave geophysics and large (1.8 m- diameter) plate tests. At comparable strain amplitudes, the intact lab stiffness is several times higher than the mass stiffness. Hobbs (1974) has related the rock mass quality via RQD, to a rock mass factor, j, which reduces the intact Young's modulus (typically measured in the laboratory) to a 'mass' modulus, from:

$$E_m = j\,(E_i) \tag{53.25}$$

where E_m is the mass modulus, j is the mass factor and E_i is the intact modulus. The relationship between rock quality and j is given in **Table 53.15**. Some care is needed when using RQD since it takes no account of joint opening or infill materials (which are assessed in more comprehensive rock mass classification systems such as the GSI or RMR systems). An open joint infilled with clay would lead to a much lower mass modulus than one with closed joints. Hoek and Diederichs (2006) outline a more sophisticated empirical approach based on GSI.

For preliminary settlement assessments and routine design of simple structures, it can be assumed that $E_i = Mq_{uc}$, where M is the ratio between the intact modulus E_i and the unconfined compressive strength q_{uc}. Values of M for different rock types

are given in **Table 53.15**. For some very weak rocks, such as Mercia Mudstone, it may not be practical to obtain samples of sufficient quality for reliable laboratory measurements of stiffness and compressibility, and then more reliance will need to be placed on *in situ* loading tests such as the pressure-meter. Some care is required, since a horizontal undrained modulus will be measured and a vertical drained stiffness will need to be derived for settlement estimates. Mass compressibility parameters for the design of shallow foundations in chalk are given in **Table 53.16**.

For Mercia Mudstone, profiles of rock mass Young's modulus versus depth, based on the back analysis of foundation settlement in unweathered mudstone, siltstone (typically Grades I and II) and sandstone, are given in **Figure 53.30**. Meigh (1976) suggested that for weathered mudstone/siltstone, typically mobilised drained Young's modulus, is about 4 MN/m^2 at the surface and increases at about 4 MN/m^2 per metre depth (i.e. similar to firm-stiff clays); hence weathering could change mobilised Young's modulus by more than an order of magnitude in weak rocks.

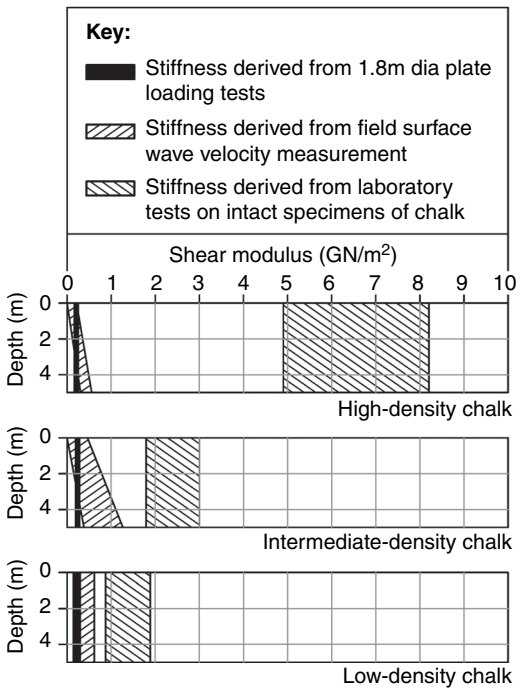


Figure 53.29 Comparison of initial modulus from 1.8-m-diameter plate tests with stiffness measured on intact chalk, and derived from surface-wave geophysics
Reproduced from Clayton *et al.* (1994)

53.8 Case history for a prestigious building on glacial tills

53.8.1 Introduction

A large public building was to be constructed in an urban area. An important architectural feature was a special glass frontage across one corner of the building. This meant that an onerous differential settlement criterion was imposed. The maximum net foundation bearing pressure was 250 kN/m^2.

A desk study and preliminary ground investigations indicated a surface layer of loose and variable made ground, about 2 to 3 m thick. Beneath this was a stiff to very stiff layer of glacial till. An existing old building covering the proposed site constrained use of the site, both for carrying out ground investigations and for construction of the new building (since demolition was on the overall project critical path).

The building was to be constructed under a 'design and build' contract, and there were very severe penalties under the contract if the building was completed late. The consultant responsible for the building design (including the foundations) was commissioned by the building contractor.

Quality classification[1]	RQD (%)	Fracture frequency per metre	Velocity index[2] (Vf/Vl)^2	Mass factor [j]
Very poor	0–25	15	0–0.2	0.2
Poor	25–50	15–8	0.2–0.4	0.2
Fair	50–75	8–5	0.4–0.6	0.2–0.5
Good	75–90	5–1	0.6–0.8	0.5–0.8
Excellent	90–100	1	0.8–1.0	0.8–1.0

	Rock type	Modulus ratio, Mr[3]
Group 3	Very marly limestones, poorly cemented sandstones, cemented mudstones and shales, slates and schists (steep cleavage and foliation)	150
Group 4	Uncemented mudstones and shales	75

Notes:
1. As BS 5930.
2. *Vf* is the wave velocity in the field and *Vl* is the wave velocity in the laboratory.
3. M_r is the modulus ratio, where $E_m = j\, M_r q_{uc}$; E_m is the modulus for a rock mass, j is the mass factor and q_{uc} is the unconfined compressive strength.
These approximate correlations are for preliminary settlement estimates only. Site-specific tests may be necessary depending on project requirements.

Table 53.15 Approximate correlations for rock mass stiffness

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Density	Grade	Es [1] (MPa)	q_y (kPa)	Ey[2] (MPa)	q_u[3] (MPa)
Medium/high	A	1500–3000	>1600–2400	?[4]	>16
Medium/high	B	1500–2000	300–500	35–80	4.0–7.7
Medium/high	C	300–1500	300–500	35–80	4.0–7.7
Low	B and C	200–700	250–500	15–35	1.5–2.0
	Dc	200[5]	225–500	20–30	-
	Dm	6	?	-	-

Notes:
1. E_s is the initial modulus, but standardised at a net applied stress of 200 kPa; q_y is the yield stress.
2. E_y is the post-yield modulus, assumed to be fully developed once ρ/D exceeds about 0.4%.
3. q_u is the ultimate stress, measured at ρ/D = 10–15% of the plate diameter.
4. Yield has not been observed for this grade of chalk.
5. Values given in this table are short term, based upon plate loading tests. In the long term, this figure reduces to 75 MPa.
The above values are for initial guidance only, site-specific tests may be necessary.

Table 53.16 Empirical mass compressibility parameters for the design of shallow foundations in chalk
Data taken from CIRIA C574, Lord *et al.* (2002), www.ciria.org

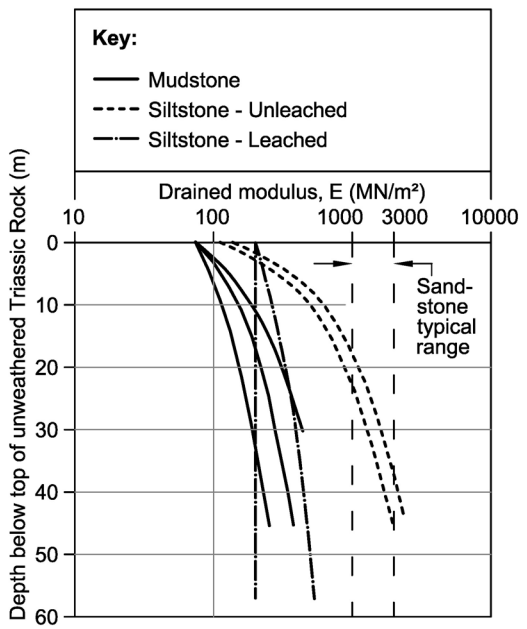


Figure 53.30 Measured secant drained Young's moduli in unweathered Triassic rocks
Data taken from Meigh (1976)

53.8.2 Ground conditions and site history

Desk studies had shown that neither mining nor quarrying had affected the site. A key observation from the ground investigation was the presence of numerous boulders within the glacial till. The description of the glacial till indicated it was consistently stiff or very stiff and its liquidity index was about –0.2 (±0.1) in the uppermost 5 m of the till. These were indicators of the heavily overconsolidated nature of the till and together with the plasticity index, which was typically about 15%, indicated that compressibility of the till would be very low. The above observations were consistent with the relevant technical literature and local experience, which was collated during the desk study.

53.8.3 Foundation options and risks

The initial design criterion was for foundation settlement to be less than 5 mm. As a consequence, bored piles were initially selected for the new building foundations. There were serious risks with this foundation type, because the numerous boulders would form significant obstructions to pile boring. Hence, there was a major risk of large delays to the overall construction programme. Past experience had shown that shallow foundations on stiff glacial tills usually exhibit small settlement under the envisaged foundation bearing pressures. The old building foundations were pad foundations and the decision was taken to supplement the existing shallow foundations, so mass concrete was used to broaden and lengthen the existing pads, which were then overlain by reinforced concrete pads. In addition, negotiations with the architectural finishes supplier enabled a less onerous differential settlement criterion to be agreed of 10 mm.

53.8.4 Additional ground investigations and geotechnical analysis

An additional ground investigation was carried out. This comprised three 4 m-deep trial pits to expose the glacial till and enable high-quality block samples to be taken. Advanced laboratory tests were then carried out to obtain realistic drained stiffness parameters for the till. Glacial till is often heterogeneous and, because of access constraints (due to the existing building), there were several areas across the site where there was no geotechnical data. To mitigate this risk, static cone tests (CPT's) were carried out as soon as the existing building was demolished and before foundation construction for the new building began. The CPT's could be carried out and interpreted

quickly. Cone penetration was limited, typically to about 7 or 8 m depth, due to boulders, but this was adequate given the limited depth of influence of the narrow, circa 2 m wide, pad footings. These tests confirmed the till strength was adequate across the whole site for the proposed design.

The undrained strength and compressibility data are plotted in **Figures 53.31** and **53.32**. The routine triaxial data (on U100 samples) obtained from the first investigation indicated relatively low undrained strengths, compared with the block samples taken from the trial pits. The subsequent CPT profiles indicated that the till was reasonably uniform and that strength increased with depth. The design profiles were based on the block samples and the CPT's.

To assess the till's compressibility across a wide range of strain amplitudes several different test techniques were specified: geophysical (bender element) tests for compressibility at very small strain; oedometer tests for large strain compressibility and triaxial tests with local instrumentation to measure compressibility at intermediate strain amplitudes. The data derived from some of these tests are shown in **Figure 53.32**. It is worth noting that the oedometer tests (using relatively large load increments/decrements of 100 kN/m^2 plus) on U100 samples (taken during the first ground investigation) gave an average coefficient of volume compressibility m_v of 0.1 m^2 MN, whereas oedometer tests on the block samples (with load-unload-reload loops and small load increments on reloading, typically 20 to 30 kN/m^2) gave m_v values that were between two and four times smaller.

Bearing-capacity checks showed that the factor of safety was greater than three. In view of the onerous different settlement criterion, a careful analysis of the foundation settlement was required. The main method of settlement analysis was a modified one-dimensional method, which allowed for the variation of deformation modulus with both strain amplitude and mean effective stress due to foundation loading (O'Brien and Sharp, 2001). The configuration of the pad and strip footings beneath the building footprint was quite complex; however, this was easily taken into account by allowing for superposition of vertical stress increments (calculated from Boussinesq theory).

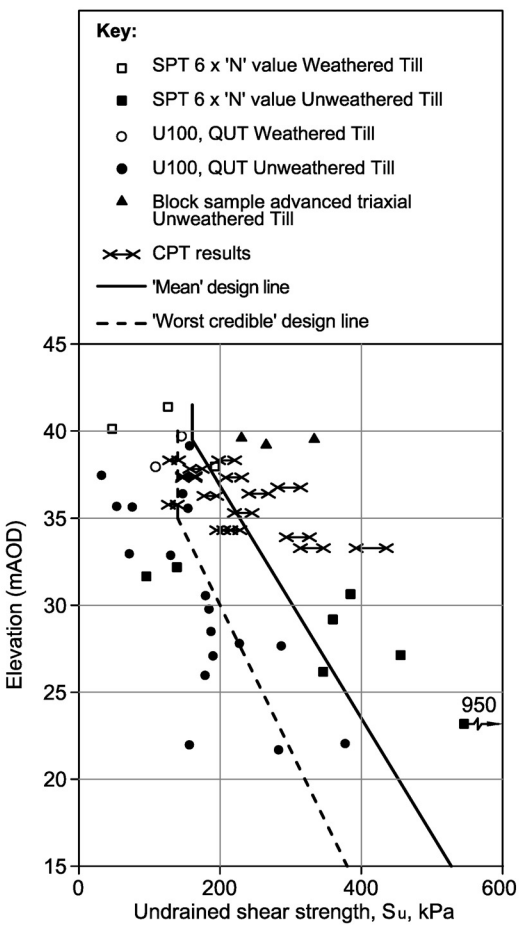


Figure 53.31 Case history, glacial till undrained shear strength data

53.8.5 Design verification and construction control

A critical risk was the potential for the till to soften rapidly if it was exposed to water (either groundwater or rainfall). During trial pitting, no problems were encountered with water seepage. Nevertheless, the construction specification required the till at foundation level to be carefully protected. In particular, a protective layer of till 0.6 m thick was to be left above the foundation level, and once final excavation was carried out the till was be covered with a structural-quality layer of blinding concrete 100 mm thick within the same shift.

The shallow foundations were constructed rapidly and without difficulties, and the project was completed ahead of schedule. Subsequent monitoring indicated foundation settlement of less than 10 mm, which was consistent with the predictions derived from the additional ground investigation data.

53.8.6 Conclusions

The building featured a special glass frontage with an onerous settlement criterion. It was built under a fast-track design and build form of contract. If piling had been attempted it is likely that severe delays would have adversely affected the project, due to large boulders within the glacial till. Local experience generally indicated that shallow foundations exhibited only small settlement when founded on the very stiff heavily overconsolidated till. The first ground investigation, using routine sampling and testing methods, indicated that shallow foundation settlement would be excessive. Hence, additional investigations were carried out using modern testing methods. This high-quality data was used together with a modified one-dimensional settlement analysis (section 53.6.4), which took account of nonlinear elastic behaviour, to demonstrate that shallow foundations would perform satisfactorily. The old building foundations were re-used and supplemented by new shallow foundations, and the new building was successfully completed.

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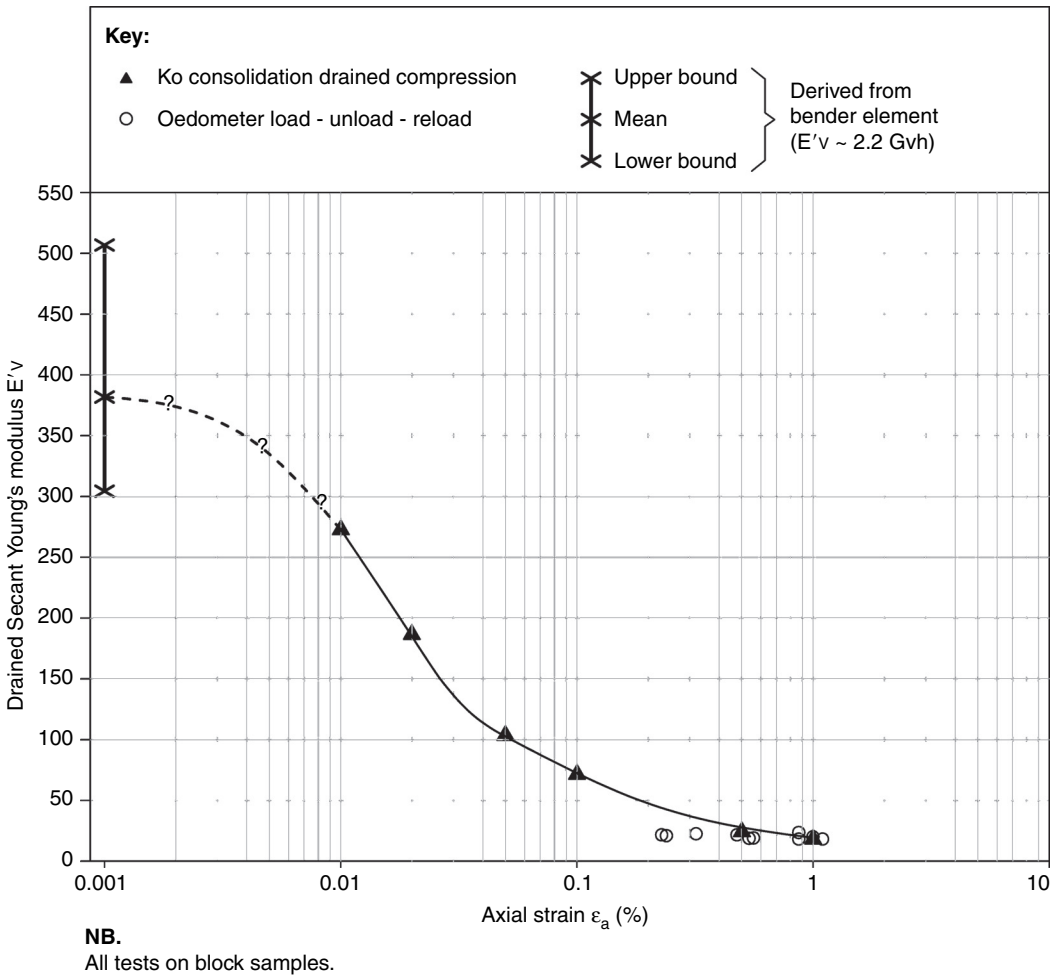


Figure 53.32 Case history, glacial till drained secant Young's modulus variation with strain amplitude

53.9 Overall conclusions

- There can be several causes of shallow foundation movements, which are independent of the applied bearing pressures imposed by a superstructure (**Table 53.2**). The global ground movements caused by these geohazards can lead to serviceability or ultimate limit state failure of structures. The seasonal shrink–swell movement of clay soils, induced by trees, is a common cause of excessive shallow foundation movement.
- The temporary stability of excavations and the potential for groundwater inflow require careful consideration, because of the safety and cost implications.
- The applied loads on a foundation usually comprise a variety of different types of load, including dead loads and transient loads (including wind, impact and accidental loads). It is important to understand the nature of the applied loads before carrying out bearing-capacity and settlement calculations. For example, when calculating the long-term settlement of clays, occasional transient loads would usually be ignored.
- When carrying out bearing-capacity and settlement checks, it is important to consider the behaviour of the whole structure, in addition to the individual foundation elements, within the context of the site, adjacent structures and topographical features. The risk of adverse interaction between adjacent structures tends to increase if foundations are subject to lateral loads.
- There have been significant developments in bearing-capacity theory beyond the original equations proposed by Terzaghi and Skempton. A wide range of situations, including increasing strength with depth, anisotropy, soil layering and allowances for soil compressibility and stress level can be analysed. Foundations subjected to combined vertical, horizontal and moment loads have conventionally been assessed by applying inclination factors to the bearing-capacity factors for vertical loading. However, the reliability of these correction factors reduces as the load inclination (*H*/*V* ratio) increases, and for angles beyond 18° to the vertical these correction factors are not recommended. More recent bearing-capacity theories based on a *V*/*H*/*M* failure surface (Chapter 21 *Bearing capacity theory*) are more appropriate.
- The bearing capacity and settlement of rock will be sensitive to the orientation, spacing and characteristics of discontinuities within the rock mass (**Figure 53.13**), the degree of weathering and the presence of voids.

- Vertical stress changes beneath a foundation predicted by homogeneous, isotropic, linear elastic theory are reasonably accurate for many practical situations, which may involve anisotropy, nonlinearity and increasing stiffness with depth. In contrast, horizontal stress changes predicted by elastic theory are often inaccurate for these situations.
- Foundation settlement can be assessed quickly by using displacement influence factors derived from elastic theory. Published solutions that assume the elastic material has constant Young's modulus with depth or is of infinite thickness are not recommended. Solutions that allow for finite thickness and with Young's modulus increasing with depth will generally provide more realistic predictions.
- For heavily overconsolidated clays the simple one-dimensional method can be used to calculate the *total* settlement, and this method is as accurate as (and often more accurate than) many more sophisticated methods. The major difficulty is reliable measurements of compressibility, which are appropriate for the relevant stress changes and strain amplitudes. The modified one-dimensional method, which uses the constrained modulus D, is helpful in this respect since D can be related to a variety of measureable parameters that are relevant for a wide range of strain amplitudes.
- The settlement of medium dense and dense silica sands is usually small (some exceptions are summarised in **Table 53.8**) and for simple structures the use of the empirical correlations in **Figure 53.18** will often be sufficient to estimate the likely bounds of settlement. A variety of more complex methods are available; however, these usually depend on relating penetration resistance (from SPT's or CPT's) to sand compressibility and will, therefore, be of limited reliability.
- Many of the serious problems that occur with shallow foundations are due to a lack of understanding of the site's history, geology or hydrogeology. Hence, comprehensive desk studies are always important and ground investigations must be implemented with great care and thoroughly checked.
- For shallow foundations on heavily overconsolidated clays, settlement is concentrated immediately beneath the foundation and more than half of the settlement will generally occur over a depth of about half the foundation width. Hence, much more emphasis needs to be placed on measuring soil properties in this area. The *mobilised* soil stiffness generally increases rapidly with depth (typically from 'large strain' values immediately beneath the foundations to 'very small strain' values remote from the foundations).
- For soft clays and bonded materials (such as chalk) perhaps the most critical parameter for foundation design is the assessment of the yield stress (commonly known as the pre-consolidation pressure for soft clays). The yield stress for chalk is reasonably well established (**Table 53.16**) and at net bearing pressures below the yield stress, settlement will be small. The compressibility of a rock mass will generally be far higher than the intact material because of the influence of discontinuities.

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It is recommended this chapter is read in conjunction with

- Chapter 19 *Settlement and stress distributions*;
- Chapter 21 *Bearing capacity theory*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 54
Single piles

Andrew Bell Cementation Skanska Ltd, Doncaster, UK
Christopher Robinson Cementation Skanska Ltd, Doncaster, UK

This chapter considers the design of single piles, from selection of appropriate pile type(s) through to pile load testing strategies. Methods which are commonly used for estimating axial pile capacities in three principal ground types, as well as layered soils, are discussed, covering both empirical and theoretical methods. Other aspects of single pile design covered by this chapter relate to factors of safety, pile settlements and pile response to lateral loading.

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54.1 Introduction

Piles are generally required to transfer load from a superstructure through weak or compressible strata, or through water, on to stiffer and less compressible soils and rock. Piles are also required to reduce both overall and differential settlements of the supported structure and may be required to enable construction processes such as top-down construction. They may also be required to resist uplift loads as well as compressive loads when used, for example, to support tall structures which are subject to overturning forces caused by wind. Where basements are to be constructed piles may also be subject to heave-induced tension. A more recent additional use of piles is within ground source heating and cooling systems as part of a development's renewable energy strategy.

Piles used for marine structures are often subject to lateral loads from ship impact and wave forces. Many land-based piles are also subject to lateral forces; for example, in bridge works there are earth pressure forces, expansion and contraction forces and braking and traction forces. Combinations of vertical and horizontal forces and bending moments are common and pile sections have then to be designed according to the relevant structural codes in addition to geotechnical considerations. Foundations beneath machinery may also be subject to cyclic and vibration forces.

Before embarking on the design of a piled foundation, it is essential to have a thorough understanding of all the relevant factors that might affect the performance of the pile being considered.

The design of piled foundations has traditionally been based on a combination of simple empirical methods and local experience, but over the past few decades there has been a gradual change towards more soundly based theoretical methods. Poulos (1989) provided a summary of the range of analysis and design methods available for piled foundations, ranging from simple empirical methods to the most sophisticated non-linear numerical analysis methods. When selecting the appropriate design approach, the following factors should be considered:

- the quality of the site investigation and scope of geotechnical data available;
- the design stage, e.g. conceptual design or detailed design;
- the budget and timescale for foundation design;
- the scale and sensitivity of the proposed structure, especially allowable total and differential settlements;
- the complexity of the ground conditions and loading regime.

54.2 Selection of pile type

Foundation selection is discussed in Chapters 9 *Foundation design decisions* and 52 *Foundation types and conceptual design principles*.

The key factors that usually affect the choice of pile type and ultimately the design of the pile itself are as follows:

1. *Ground conditions*
   - Strength or weakness of the overlying and founding strata
   - Ground variability
   - Potential for downdrag or heave loading, or lateral loading
   - Presence of obstructions
   - Aggressive ground conditions
2. *Construction constraints*
   - Location
   - Access
   - Working area
   - Sensitivity of adjacent assets
   - Underground utilities and structures
3. *Safety and environmental constraints*
   - Risk to operatives and the public

Risks to adjacent structures and underground utilities assets

Risk of contamination to surrounding environment

Effects of noise

Effects of vibration

4. *Nature and magnitude of loads*

   Direction of foundation loading (e.g. lateral, tensile, compression)

   Source and characteristics of loadings (e.g. heave, downdrag, cyclic, static)

5. *Economic considerations*

   What it will cost and whether there is a more economic solution

Prior to embarking on the design of a pile, it is essential that the designer first assesses the type of piles and construction methods that are most appropriate for the particular site under consideration. There is clearly no sense in designing a driven pile in an area where noise and vibration will be an issue. Conversely, designing a continuous flight auger pile 35 m deep when available plant will only reach 30 m is equally futile. Under current health and safety regulations (Health and Safety Executive, 2007), it is also essential for the designer to take into account the risks associated with the installation of the proposed pile solution during the design and specification process; to complete this satisfactorily the designer will need at least a general understanding of the various pile types available and the advantages, disadvantages and key safety considerations of each.

Before any design work is carried out it is essential that an assessment of pile buildability is carried out. Reference should be made to Chapters 52 *Foundation types and conceptual design principles*, 79 *Sequencing of geotechnical works* and 81 *Piling problems* in this respect.

It is important to recognise that each pile type and installation method disturbs the ground in different ways, and as such the design method and considerations may vary between the different systems. This disturbance can improve or reduce both skin friction and end-bearing, and as such will affect the bearing capacity of the complete pile.

Piles of varying type and length may be used on a site provided a suitable assessment of likely differential settlement is carried out. This should include:

- pile size (diameter and length), i.e. geotechnical axial load capacity in compression and tension of a single pile;
- displacement at working load of a single pile;
- differential settlement between adjacent foundations;
- influence of lateral loading, assessment of geotechnical and structural capacity, and lateral displacement;
- assessment of pile group behaviour under vertical and lateral loading (see Chapter 55 *Pile-group design*);
- objective of piles – if it is to reduce settlement/bending moments in a raft then consider piled raft (see Chapter 56 *Rafts and piled rafts*);
- influence of any 'global' ground movements both during construction and long term (see Chapter 57 *Global ground movements and their effects on piles*);
- testing strategy for checking integrity of pile;
- testing strategy for checking vertical and/or lateral load behaviour;
- potential for foundation reuse.

54.3 Axial load capacity (ultimate limit state)

The approach described in this chapter to assess the resistance of piles to compressive loads is the static 'soil mechanics' method for piles loaded axially. Dynamic methods of design and analysis are available but have not been included in this section and attention will be limited to problems involving static or quasi-static axial loading of single piles only.

The general simplified formula for the design of bearing piles typically takes the form of

$$Q_{ult} = Q_b + Q_s - W_p, \tag{54.1}$$

where Q_{ult} is ultimate pile capacity, Q_b is base capacity, Q_s is shaft capacity and W_p is pile self weight.

Furthermore, $Q_b = q_b A_b$ and $Q_s = q_s A_s$, where q_s is unit shaft resistance, q_b is unit base resistance, A_b is area of pile base and A_s is area of pile shaft.

The diagram in **Figure 54.1** shows this relationship, for circular piles.

The above approach, separating the evaluation of shaft and base capacity, forms the basis of all static pile capacity calculations. These components can be broken down further to resistance per unit shaft or base area, and the relative magnitude is dependent on the combination of pile geometry, pile installation method and ground conditions.

Typically, the shaft capacity of a pile is mobilised at much smaller displacements than the base capacity. Shaft capacity can be fully mobilised at movements of only 0.5% to 2% of pile diameter, whereas displacements of around 10% to 20% of pile diameter may be required to mobilise the base resistance, even more if the soil at the base of the pile is disturbed as part of the installation process. It is essential that these differences between the load deformation characteristics of the pile shaft and base are considered early during the pile design process; **Figure 54.2** shows this as an idealised load displacement response curve. Generally, long slender piles are more efficient than short fat piles with regard to settlement control but limitations on the installation technique may often dictate the pile geometry.

There can be concerns about the axial pile capacity under tension loads. De Nicola and Randolph (1993) showed that shaft capacity under tension load is between about 70% and

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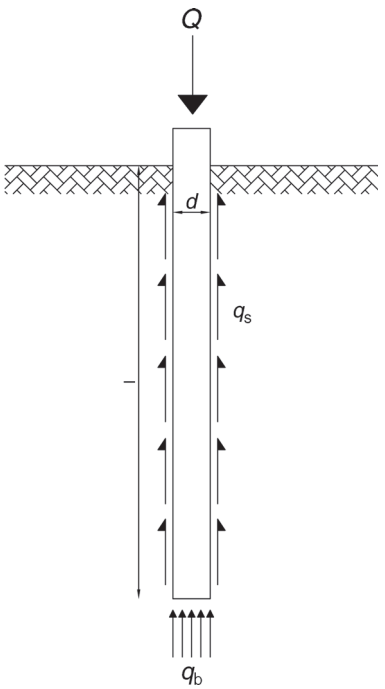


Figure 54.1 Pile under vertical load
Courtesy of Cementation Skanska

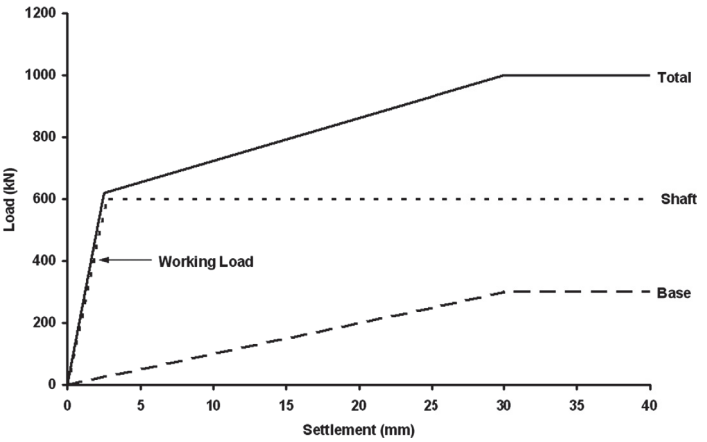


Figure 54.2 Idealised load displacement response
Courtesy of Cementation Skanska

90% of the shaft capacity under compression loads (depending on the relative soil/pile compressibility). However, most codes require higher factors of safety for piles subject to tension loads than those for piles subject to compression. If these higher factors of safety are used, then there is no need to reduce the shaft capacity calculated by routine methods, since the potential adverse effects are taken into account by the use of a larger factor of safety. For piles subject to tension, the base resistance is ignored.

54.3.1 Fine-grained soils

54.3.1.1 Total stress

End-bearing

Whilst the long-term 'effective stress' drained end-bearing capacity of a pile in clay will be significantly larger than the undrained capacity, unfortunately the movements required to mobilise such capacity would be unacceptable for most structures. Furthermore, the pile will require sufficient capacity in the short term to provide the initial load-carrying capability required, and the rate of build-up of load over time will depend on the speed of construction, something which continues to increase as construction technology advances. For these reasons, the end-bearing of piles in fine-grained soils is generally considered using total stress methods, in terms of the undrained shear strength of the clay, S_u, and a bearing capacity factor, N_c, where

$$q_b = N_c S_u. \tag{54.2}$$

The bearing capacity factor relevant for the depth of most piles is usually taken as 9 after Skempton (1951). Allowance should be made for the potential of a softened or disturbed base in some circumstances, particularly when considering underreamed piles, with enlarged bases. In such cases a reduction in S_u should be applied. A reduced value of N_c should also be considered when the pile tip only just penetrates the founding fine-grained strata. A value of $N_c = 6$ is recommended for founding in the top of such founding strata, increasing linearly to 9 after 3 pile diameters' penetration into the stratum.

Shaft friction

Piles in fine-grained soils generally derive most of their capacity in shaft friction, with the base resistance often not mobilised under working loads, particularly in the short term.

Shaft friction, in terms of total stress, is generally calculated using an empirical factor α, where

$$q_s = \alpha S_u. \tag{54.3}$$

The value of α adopted for bored piles typically ranges from 0.3 to 0.6 for straight-shafted piles; higher values have been proven for ribbed or irregular pile shafts. Values of 0.45 to 0.6 are generally adopted for bored piles in stiff overconsolidated clays but can be as high as 0.7 for driven displacement piles. The value of α depends on a combination of the pile type and installation method and the nature of the fine-grained soil. In the UK, the LDSA London District Surveyors Association's *Guidance Note 1* (2009) gives clear recommendations on appropriate values for α for bored and CFA piles founding in London Clay. It is also important for more complex and variable fine-grained soils, such as glacial till, that reference is made to specific guidance such as Weltman and Healy (1978) and Trenter (1999). **Figure 54.3** details the variation of α with

undrained shear strength in glacial till, after Weltman and Healy (1978).

Poulos (1989) summarised some of the most commonly used methods in calculating α for driven piles, an extract of which is included in **Table 54.1**.

It is generally considered good practice to limit the average skin friction in fine-grained soils to 100 kN/m^2 unless proven higher by pile load testing or by suitable experience of pile performance in similar ground condition.

The S_u value in equation (54.3) is really an empirical index value, closely linked to historical sampling/lab test methods together with back analysis of pile test data, rather than a 'fundamental' soil parameter. In the UK, the method is mainly based on experience in London Clay, with the results of Quick Undrained Triaxial (QUT) tests on U100 samples being compared with the results of maintained load tests (refer to Patel, 1992 for further background). Shaft failure requires shearing to take place through a large volume of ground; hence, the S_u value selected for design is a 'mean' value (where the mean is based on representative data, excluding spurious high or low values).

54.3.1.2 Effective stress (fine-grained soils)

Historically, the use of effective stress calculations for the design of piles in fine-grained soils has been limited, but recent research has provided more accurate methods of using this approach. The Imperial College design methods (Jardine *et al.*, 2005) provide recommendations on appropriate methods of calculation for both end-bearing and shaft resistance for driven steel piles in terms of effective stress. Chapter 22 *Behaviour of single piles under vertical loads* provides guidance on effective stress design to assess the capacity of single piles subjected to vertical loading. It is important with such methods to limit the calculated values of end-bearing and shaft friction to those achievable in the field for the pile type and pile construction methods being employed.

An important application of effective stress methods is for piles below or adjacent to deep excavations (say in excess of 4 m deep), when total stress methods may be unsafe.

End-bearing

The use of effective stress methods to calculate the end-bearing of piles in clay soils is, in general, not recommended. It would only be appropriate to adopt such an approach based on the results of suitable pile load testing or appropriate experience in similar ground conditions.

Shaft friction

Assuming loading takes place under fully drained conditions, where remoulding and drainage at the pile–soil interface

Alpha factors	Reference
$\alpha = 1.0$ ($S_u \le 25$ kN/m^2)	API (1984)
$\alpha = 0.5$ ($S_u \ge 70$ kN/m^2)	
Linear variation in between	
$\alpha = 1.0$ ($S_u \le 35$ kN/m^2)	Semple and Bigden (1984)
$\alpha = 0.5$ ($S_u \ge 80$ kN/m^2)	
Linear variation in between	
Length factor applies for $L/d > 5$	
$\alpha = \left(\frac{c_u}{\sigma'_v}\right)_{nc}^{0.5}\left(\frac{c_u}{\sigma'_v}\right)^{-0.5}$ for $\left(\frac{c_u}{\sigma'_v} \le 1\right)$	Fleming *et al.* (1985)
$\alpha = \left(\frac{c_u}{\sigma'_v}\right)_{nc}^{0.5}\left(\frac{c_u}{\sigma'_v}\right)^{-0.25}$ for $\left(\frac{c_u}{\sigma'_v} \ge 1\right)$	

Table 54.1 Summary of commonly used α factors for driven piles in fine-grained soils

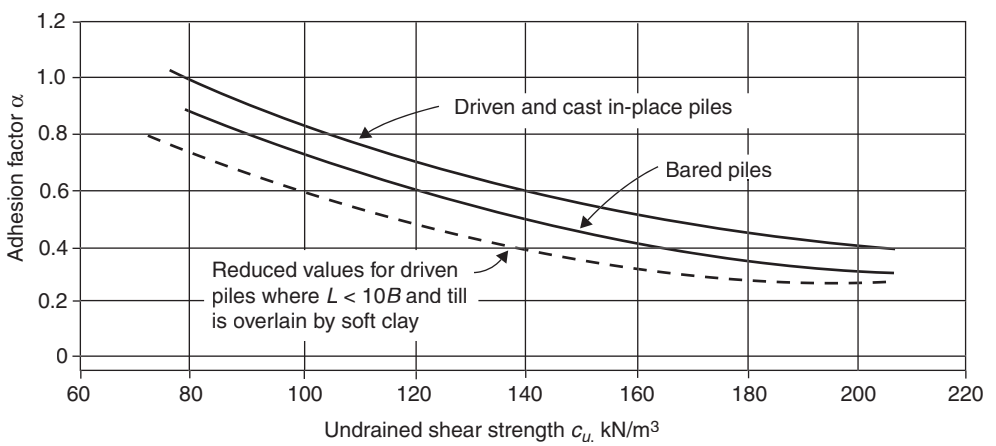


Figure 54.3 Adhesion factors, α, for piles in glacial till
Reproduced from CIRIA PG5, Weltman and Healy (1978), www.ciria.org

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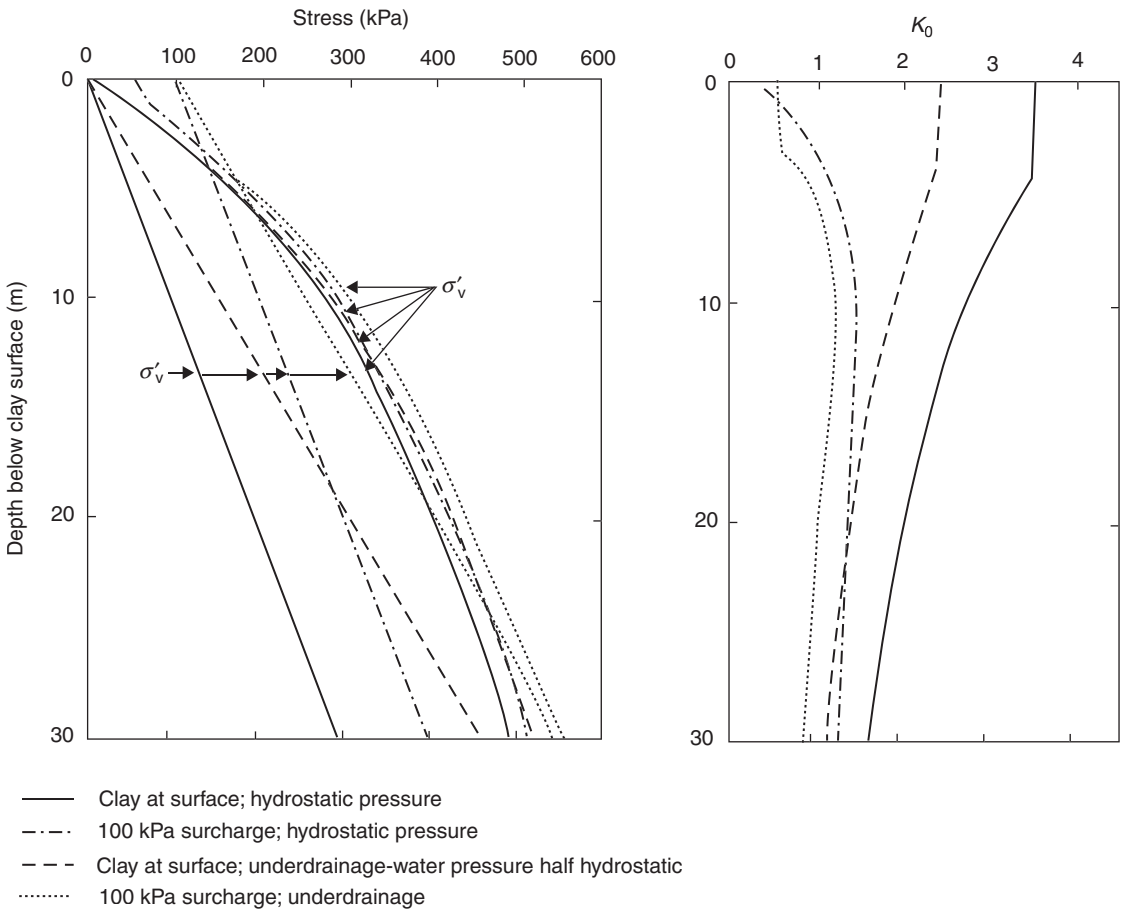


Figure 54.4 Influence of stress history on K_0 and σ'_h in overconsolidated clay
Reproduced from Burland *et al.* (1979)

destroys all cohesion so that $c' = 0$, then the unit shaft friction can be calculated using

$$q_s = \sigma'_h \tan \delta, \tag{54.4}$$

where σ'_h is horizontal effective stress following pile installation and δ is effective interface friction angle.

For bored piles δ would usually be assumed to equal the critical state angle of friction, φ'_{cv}, whereas for driven piles it would be assumed to equal the residual angle of friction, φ'_{res}.

A simplification can be made if σ'_h is assumed equal to $\sigma'_v K$, in which case $q_s = \sigma'_v K \tan \delta$, further simplified, where $K \tan \delta = \beta$, to $q_s = \beta \sigma'_v$.v Values of β can vary significantly depending on consolidation history (i.e. normally consolidated or overconsolidated) and method of pile installation (driven or bored). Examples of typical values of β and K are given in **Tables 54.2** and **54.3**.

The main challenge in applying equation (54.4) above is to assess the appropriate values of σ'_h or K. As shown in **Figure 54.4** the value of K is very sensitive to the site's stress history and also depends on stress changes during pile installation. Further discussion can be found in Bown and O'Brien (2008).

The β approach is particularly useful for normally consolidated or lightly overconsolidated soils where K_0 can be assumed to be close to $1 - \sin \varphi'$. Then, for example, in the case of a driven pile, if the angle of effective friction is taken as, say, 21°, the ultimate friction becomes $0.25\sigma'_v$. For a very wide range of normally consolidated soils, the value of β is likely to be between 0.2 and 0.3. This value is often used as the basis for calculation of downdrag or 'negative friction'.

54.3.2 Coarse-grained soils

Approximate design methods for piles in coarse-grained soils are well documented and reference should be made to Fleming *et al.* (2008) and Tomlinson (2004) for a wider understanding of the range of design methods available. Recent research has focused on steel-driven piles, aimed predominantly at the offshore, industry, and reference should be made to the work by Randolph and White (2008) and Jardine *et al.* (2005).

K value	Reference
K is the smaller of K_0 and $0.5(1 + K_0)$	Fleming *et al.* (1985)
$K/K_0 = 2/3$ to 1; K_0 is a function of OCR	Stas and Kulhawy (1984)

Table 54.2 Typical *K* values for bored piles in fine-grained soils

β value	Reference
$\beta = (1 - \sin\varphi')\tan\varphi'(\text{OCR})^{0.5}$	Burland (1973)
Meyerhof's value	Meyerhof (1975)

Table 54.3 Typical β values for driven piles in fine-grained soils

54.3.2.1 End-bearing

The end-bearing capacity of piles in coarse-grained soils can be calculated by the formula

$$q_b = \sigma'_{vB} N_q, \tag{54.5}$$

where σ'_{vB} is effective vertical stress at the pile toe and N_q is bearing capacity factor.

N_q is often derived based on a correlation with φ. A commonly used method is that by Berezantzev *et al.* (1961) (**Figure 54.5**). The main problem with this approach is that it is unduly sensitive to small changes in φ', and does not allow for the reduction in mobilised φ' as effective stress levels increase. It also implies that end-bearing pressure will continue to increase proportionally with depth. An alternative approach is that given in Fleming *et al.* (2008), which relates N_q to relative density, effective stress and the critical state angle of friction φ'_{cv}, shown graphically in **Figure 54.6** for $\varphi'_{cv} = 30°$. φ'_{cv} values for a range of sands are summarised by Stroud (1989) and Bolton (1986). Relative density can be assessed from standard penetration test (SPT) 'N' or cone penetration test (CPT) 'q_c' values (section 54.3.4).

Routinely used methods that can be used to calculate end-bearing are summarised in **Table 54.4**.

With any design approach, it is very important to limit the calculated end-bearing to achievable values to ensure serviceability limits are achieved; this is discussed further in section 54.3.5 below. It is possible with theoretical bearing capacity based methods to significantly overestimate the end-bearing resistance, particularly for piling methods that may disturb the soil at the pile toe or for short piles that are highly dependent on end-bearing.

The end-bearing capacity of a pile in a uniform layer of coarse-grained soils will not continue to increase proportionately with depth. Research has shown that the end-bearing pressure soon reaches a limiting value (Vesic, 1977), typically for pile lengths in excess of about $20D$ (where D is the pile diameter). For this reason end-bearing is often limited

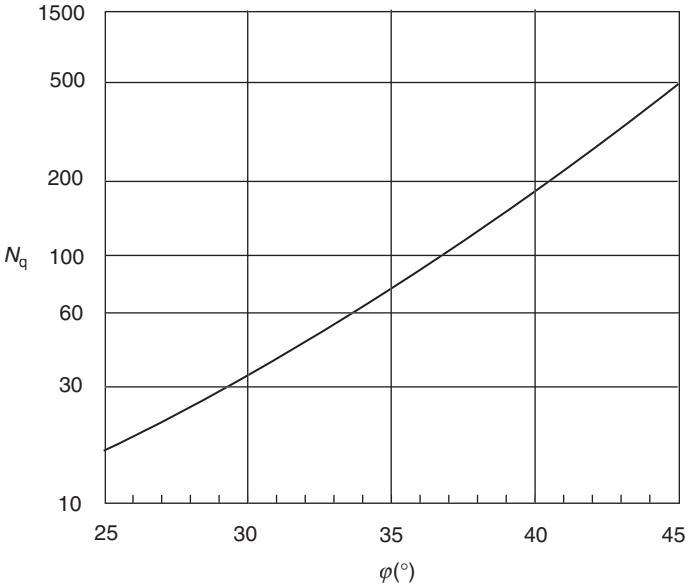


Figure 54.5 Variation of N_q with φ
Reproduced from Berezantsev *et al.* (1961)

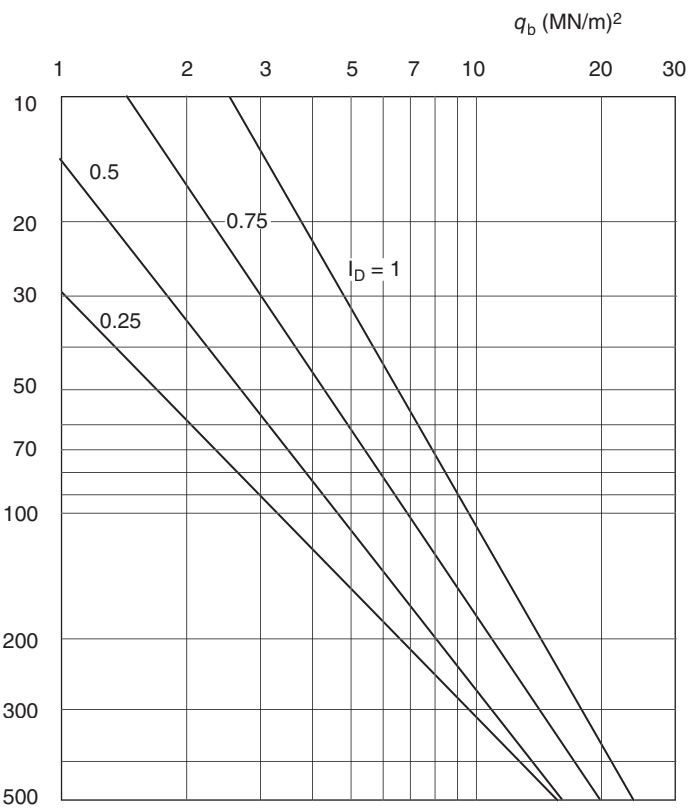


Figure 54.6 End-bearing capacity, coarse-grained soils, with $\varphi'_{cv} = 30°$
Reproduced from Fleming *et al.* (2008), Taylor & Francis Group

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Material	N_q Value	Reference
Silica sand	$N_q = 40$	API (1984)
	N_q plotted against φ'	Berezantzev *et al.* (1961)
	N_q related to φ', relative density and mean effective stress	Fleming *et al.* (2008)
	N_q from cavity expansion theory, as a function of φ' and volume compressibility	Vesic (1972)
Uncemented calcareous sand	$N_q = 20$	Datta *et al.* (1980)
	Typical range of $N_q = 8$–20	Poulos (1988)
	N_q determined for reduced value of φ'	Dutt and Ingram (1984)

Table 54.4 Methods of calculating end-bearing

conservatively to between 10 and 15 MN/m^2 for dense sands or gravels, depending on the pile installation method, and to about half these values for medium-dense sands or gravels. Higher values can be adopted based on the results of suitable pile load testing or appropriate experience in similar ground conditions. For loose coarse-grained soils very little end-bearing resistance may be mobilised at displacements which are small enough for normal foundation requirements.

54.3.2.2 Shaft friction

The ultimate shaft friction of piles founding in sands or gravels is generally calculated using

$$q_s = K_s \sigma'_v \tan \delta, \tag{54.6}$$

where K_s is earth pressure coefficient after pile installation, σ'_v is effective vertical overburden pressure and δ is angle of skin friction of the pile.

δ is generally derived from φ'_{cv}, with the correlation dependent on the roughness of the pile–soil interface, resulting from the combination of pile installation method and soil characteristics, generally ranging from 0.75 to 1.0. For most bored piles the angle of skin friction may be taken as approximately equal to the critical state angle of friction of the soil. φ'_{cv} values are typically between 30° and 33° for uniformly graded sands; 34° and 37° for well-graded sands and gravels; lower values are relevant for coarse-grained soils with rounded particles, and higher values if particles are angular; the above assume clay contents are low, say less than 5–10%. Reference should be made to Fleming *et al.* (2008) for further details. For full displacement driven piles, assuming δ equals $0.75^{-1}\,\varphi'_{cv}$ would usually be appropriate.

For most bored piles, $K_s = 0.7$ is appropriate, but for CFA piles the value of K_s is more sensitive to the soil characteristics and appears to be dependent on technique. The following values are recommended for the design of CFA piles:

Clean medium/coarse sand	$K_s = 0.9$
Fine sand	$K_s = 0.7$–0.8
Silty sand	$K_s = 0.6$–0.7
Interlayered silt and sand	$K_s = 0.5$–0.6

For full displacement driven piles K_s can be estimated as $N_q/50$; this relationship gives a typical value of $K_s = 1.2$ for such piles, and limiting values between 0.7 and 2.0 are suggested. For partial displacement piles K_s values will be lower than for full displacement, and the K_s value is usually reduced by 20%. For driven cast *in situ* piles some reduction in horizontal stress will occur when the casing is extracted, and normally a K_s value of 1.0 is assumed (if wet concrete is placed).

For auger or rotary displacement piles, there are examples of K_s values derived from pile load testing well in excess of 1.2 (Bell, 2010). Such installation methods can produce significant improvements in shaft friction over conventional bored or driven piles but the use of such high values must be used with caution and in conjunction with suitable pile load testing. The shaft friction of such piles is also highly dependent on the shape of the drilling tool, of which there are numerous variants in the marketplace.

The basic formula for calculating q_s can be further simplified to $q_s = \beta\sigma'_v$ and a range of typical β values are included in **Tables 54.5** and **54.6**.

It is important, particularly with long piles in coarse-grained soils, to limit the calculated skin friction to achievable values: a sensible limit to average shaft friction of 110 kN/m^2 should be applied. Values well in excess of this can be achieved for various installation techniques, but should ideally be proven by suitable pile load testing or relevant experience in similar ground conditions.

The ultimate shaft friction of driven piles in coarse-grained soils will approach a limiting value for pile lengths in excess of 20 pile diameters, a similar phenomenon to that for end-bearing. The limiting value is dependent on several factors (including installation methods, due to friction fatigue effects). White and Lehane (2004) provides a useful reference for this in addition to the work carried out by Jardine *et al.* (2005).

It is also important, particularly for bored and CFA piles in sands, especially silty sands, not to place too much reliance on the base capacity of the pile at working loads. It is common practice to design such piles with a factor of safety greater than 1.0 on the shaft capacity alone to limit settlements at working

Material	β value	Reference
Silica sand	β = 0.15–0.35 (compression)	McClelland (1974)
	β = 0.10–0.24 (tension)	Meyerhof (1976)
	β = 0.44 for φ' = 28°	
	β = 0.75 for φ' = 35°	
	β = 1.2 for φ' = 37°	
Uncemented calcareous sand	β = 0.05–0.1	Poulos (1988)

Table 54.5 β values for driven piles (where $q_s = \beta\sigma'_v$)

Material	β value	Reference
Silica sand	β = 0.1 for φ' = 33°	Meyerhof (1976)
	β = 0.2 for φ' = 35°	Kraft and Lyons (1974)
	β = 0.35 for φ' = 37°	
	$\beta = F \tan(\varphi' - 5°)$	
	where F = 0.7 (compression) & F = 0.5 (tension)	
Uncemented calcareous sand	β = 0.5–0.8	Poulos (1988)
	limiting shaft resistance = 60 to 100 kN/m²	

Table 54.6 β values for bored piles (where $q_s = \beta\sigma'_v$)

loads to acceptable levels as determined through specified serviceability limits. If base resistance needs to be mobilised at working loads (even for driven piles), then a separate serviceability check must be carried out to assess if the working load can be mobilised at tolerable settlements.

54.3.3 Layered soils

From sections 54.3.1 and 54.3.2 above it should be clear that piles in coarse-grained soils may mobilise relatively high end-bearing capacity compared with fine-grained soils. Hence, when designing piles in interbedded sand and clays the location of the pile toe is of fundamental importance. A common practical situation when designing driven piles would be to attempt to take advantage of the potentially high end-bearing resistance of coarse-grained layers. However, if the pile toe is close to the interface with adjacent fine-grained layers then the pile will *not* develop its full end-bearing capacity (i.e. the end-bearing resistance which would be calculated by the methods outlined in 54.3.2 above).

Meyerhof (1976) first considered this important issue, and suggested that end-bearing capacity would be reduced if the pile toe was within 10B (where B is pile diameter) of the adjacent weaker layer (**Figure 54.7**). Subsequent work by Meyerhof and Sastry (1978) and Matsui (1993) indicates that Meyerhof's original guidance is, in general, overconservative; nevertheless it remains a useful guide for preliminary design.

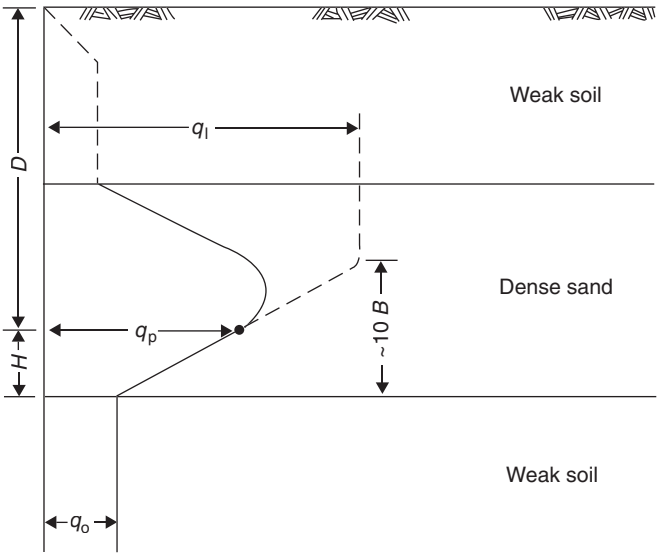


Figure 54.7 Relation between ultimate point resistance of pile and depth in thin sand overlying weak soil
Reproduced from Meyerhof (1976), with kind permission of ASCE

When the pile toe is within the zone of influence of the adjacent weaker layer, the end-bearing capacity can be assessed from **Figure 54.7** by interpolating between the capacity for the coarse-grained founding layer and the adjacent weaker layer.

A key practical issue for pile design in layered deposits is the reliable identification of the level, thickness and continuity of the proposed coarse-grained founding layer. Hence, a large number of boreholes/CPTs may be required to verify that the founding layer does not thin out, and is not in the form of isolated lenses.

If there is significant uncertainty, which can be a particular issue in glacial deposits, then it may be appropriate to base pile design on the worst credible stratification unless proven otherwise by full-scale testing. A common approach is to seek a factor of safety on shaft capacity alone in excess of 1.0, based upon the worst-case stratigraphy in terms of shaft capacity, and to place a limit on end-bearing, e.g. assuming a clay soil is present at the pile toe.

54.3.4 Correlations with SPT/CPT

In general, it is possible to use either 'indirect' or 'direct' correlations. In the 'indirect' approach the SPT or CPT is used to correlate the measured *in situ* test value with laboratory test parameters or other indices of soil behaviour (such as relative density for sands/gravels). The methods outlined in section 54.3.1 or 54.3.2 are then used. In the 'direct' approach, the measured *in situ* test value is directly correlated with shaft friction or end-bearing capacity. Both approaches can be of value, especially when there is a high level of uncertainty associated with predicting the ultimate pile capacity, e.g. driven piles in coarse-grained soils.

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Many of the most recently developed driven pile design methods directly use CPT data to derive the pile capacity, and the available evidence indicates that these methods are more reliable than the older empirical methods. When driven piles are proposed it is strongly recommended that CPTs are used to the maximum practical extent.

54.3.5 SPT

54.3.5.1 Fine-grained soils

A number of correlations between SPT results ('N' value) and *in situ* undrained strength, S_u, of silts and clays have been proposed. The most commonly adopted correlation within the UK for heavily overconsolidated clays is that proposed by Stroud and Butler (1975): $S_u = 4$ to $6 \times$ 'N' kN/m².

Other published correlations are available, but these can be unreliable. Some of the differences in correlations may result from variations in the methodology adopted in the field whilst undertaking SPTs, or the types of, or interpretive methods used for, pile tests rather than any significant differences in the soils being tested. It is to be noted that any correlations should be compared to all other available data, e.g. results from triaxial tests, field shear vane test results, etc.

There are direct methods of calculating single pile capacities from the results of SPTs undertaken in fine-grained soils, although there is limited experience with their use for UK conditions, and they should therefore be used cautiously.

Poulos (1989) has proposed the relationship

$$q_s = \alpha + \beta N, \tag{54.7}$$

where q_s is the unit ultimate shaft friction, N is the SPT penetration resistance, and α and β are constants depending upon soil and pile type.

The relationship between SPT N value and unit ultimate end bearing proposed by Poulos is

$$q_b = KN, \tag{54.8}$$

where K is a constant depending upon soil and pile type and N is the SPT penetration resistance.

The values of α, β, and K are highly dependent on the type of pile and method of pile construction. CIRIA Report 143 and Poulos (1989) give suggested values of α, β and K.

54.3.5.2 Coarse-grained soils

Correlations between SPT 'N' and peak friction angle are available, and the correlation by Peck *et al.* (1967) is often used and is usually conservative. This correlation is given in **Table 54.7**, and although fairly crude (it ignores many fundamental factors which govern mobilised friction in coarse-grained soils), it is useful for the preliminary assessment of the end-bearing resistance of small diameter (say less than 0.5 m) driven piles of moderate length (say 10–15 m long). The SPT 'N' values are uncorrected.

It is usually appropriate to also use SPT 'N' values to assess relative density, with:

- overburden correction, $N_1 = C_N N$;
- overburden correction factor, $C_N = 2/[1 + (\sigma'_v/100)]$;
- relative density, $D_r = [(N_1)_{60}/60]^{0.5}$.

Here, N is measured SPT 'N', σ'_v is vertical effective stress, and $(N_1)_{60}$ is the SPT blow count normalised to a vertical effective stress of 100 kN/m² and corrected to 60% of free-fall energy. The above relationships are for uncemented, normally consolidated sands (with a median particle size of less than 0.5 mm). For soils of large particle size (i.e. gravels/cobbles) the above correlations are unconservative. CIRIA report 143 provides further guidance.

Meyerhof (1976) proposed a set of direct correlations between SPT 'N' values and end-bearing resistance/shaft friction as follows:

$$q_b = 0.4\ \text{'N'}\ (\text{MN/m}^2) \text{ for sands, when } P > 6D, \tag{54.9}$$

$$q_b = 0.3\ \text{'N'}\ (\text{MN/m}^2) \text{ for silts, when } P > 6D. \tag{54.10}$$

Note: P is penetration of pile toe beneath surface of coarse grained soil. The 'N' value is averaged over about 3 pile diameters above/below the pile toe. For shaft friction, Meyerhof recommends the following:

$$q_s = 2\ \text{'N'}\ (\text{MN/m}^2) \text{ for driven piles,} \tag{54.11}$$

$$q_s = 1\ \text{'N'}\ (\text{MN/m}^2) \text{ for bored piles.}$$

54.3.6 CPT

54.3.6.1 Indirect methods – fine-grained soils

As with the SPT a number of correlations between *in situ* undrained shear strength, c_u and CPT cone resistance, q_c, have been proposed.

SPT 'N'	φ' (degrees)
< 5	28
5	29
10	30
15	31.5
20	33
25	34.5
30	36
35	37
40	38
> 40	38

N.B. use $\varphi' > 38°$ with caution

Table 54.7 SPT 'N' correlation to φ'

The general relationship is given by

$$c_u = q_c/N_k + p_0, \quad (54.12)$$

where Nk is a cone factor, q_c is the unit cone end resistance (kN/m²) and p_0 is the total overburden pressure (kN/m²).

It is important to note that some correlations include the effect of overburden pressure, p_0, within the cone factor, N_k. Some correlations are specifically for use with the piezone.

The cone factor, N_k, is a variable which is affected by many factors, including the rate of penetration, strength anisotropy, soil fabric (e.g. fissuring, laminations), the degree of overconsolidation of the clay deposit and the reference tests which the CPT is compared against. Overconsolidated clays generally tend to exhibit significantly higher values of N_k (especially if fissured) than normally consolidated clays.

Comprehensive guidance for the selection of appropriate values of N_k can be found in Lunne *et al*. (1997).

54.3.6.2 Direct methods – fine-grained soils

Fully mobilised unit end-bearing, q_{bu}, of a pile can be directly correlated to the unit cone end resistance, q_c, determined by CPT testing. Effects of scale and rate of testing mean that the ratio q_{bu}/q_c is usually less than one (Poulos *et al*., 2000).

The Imperial College Pile (ICP) design method (Jardine *et al*., 2005) provides methods for calculating base capacity for both closed and open-ended driven piles directly from CPT tests. The ICP design method does not, however, provide a direct correlation to pile shaft friction.

Estimates of pile shaft friction for driven piles can be made from CPT sleeve friction, e.g. $q_s = f_c$, but a high degree of caution is required, since there may be large differences in the degree of consolidation of the soil around the pile when compared to the CPT test. Results of pile tests have indicated that long-term shaft friction values can be as much as 500% greater than those measured during driven pile installation (Fleming *et al*., 2008).

Correlations between unit cone end resistance, q_c, and unit shaft friction, q_s, generally yield more reliable results than those derived from CPT sleeve friction. These correlations themselves have a reasonably wide degree of variation, e.g. $q_s = q_c/10$ (Fleming and Thorburn, 1983) and $q_s = q_c/40$ (Thorburn and McVicar, 1971).

A more reliable method for assessing pile shaft friction is usually attained from the methods outlined in section 54.3.1.

54.3.6.3 Coarse-grained soils

As noted in Chapter 22 *Behaviour of single piles under vertical loads*, where a pile toe is located within a granular stratum, the CPT cone tip can be regarded as a model of the pile toe, and so the base capacity, q_b, will correlate with the cone resistance, q_c. Care needs to be taken to suitably derive an appropriate value of q_c given the typical variability of the magnitude of q_c. White and Bolton (2005) have carried out a careful comparison of ultimate base capacities measured in pile tests against measured values of CPT q_c. They found that

$$q_{bu} = 0.9\, q_{cm}, \quad (54.13)$$

where q_{bu} is ultimate, or 'plunging failure', capacity (refer to section 54.8 below) and q_{cm} is the weighted average of q_c values within the zone of influence of the pile toe (up to 8D above and 4D below the toe, where D is pile diameter), based upon the 'Dutch' method.

Lehane *et al.* (2005) carried out further analysis on a relatively large pile test database and found that

$$q_{b0.1} = 0.6\, q_{cm}, \quad (54.14)$$

where $q_{b0.1}$ is bearing capacity mobilised at a pile settlement equal to 10% of the pile diameter and q_{cm} is as given above in equation (54.13).

In both the above studies, the important influence of adjacent weaker layers was emphasised, and this is readily taken into account by the selection of q_{cm} based upon the Dutch method.

Shaft resistance is correlated to q_c in both the ICP design and UWA design methods (Lehane *et al*., 2005).

54.3.7 Rock

The consideration of pile design in weak rocks differs widely for replacement piles (bored and CFA) and displacement piles (driven or bored). This is predominantly because it is possible to install significant rock sockets with replacement techniques, particularly when using modern high-torque piling rigs, whereas displacement systems typically meet refusal at the top of the weak rock strata or at a very short penetration into the rock. Generally strong rocks underlying a development site would not require bearing piles to penetrate them by any significant distance. There are of course occasional exceptions to this general rule, for example where adjacent railway running tunnels are required to be isolated from bearing pile loads with slip liners.

54.3.7.1 Shaft capacity, i.e. rock sockets (generally applicable to replacement piles only)

Transfer of pile loads by shear transfer along the pile-shaft-to-rock interface is relatively complex and depends on a number of factors including:

- rock strength;
- rock socket roughness;
- rock–concrete bond;
- degree of 'contamination' of the rock socket with overlying superficial deposits;
- potential degradation (smearing/polishing) of rock socket through the boring/drilling process.

Comprehensive details of load transfer mechanisms can be found in CIRIA report R181 (Gannon *et al*., 1999).

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Driven piles will generally not penetrate significant distances into rock of moderate strength. For driven piles in weak to moderately strong rock, skin friction can be calculated from the equation

$$q_s = 0.5\, K_s\, \sigma'_{vo} \tan \delta, \qquad (54.15)$$

where K_s is an earth pressure coefficient, σ'_{vo} is the effective overburden pressure and δ is the angle of friction between the rock and pile shaft.

In general, pile driving into mudstone will create a 'skin' of remoulded material around the shaft, and mobilised shaft friction will only be a small fraction of that mobilised around bored (replacement) piles.

Skin friction in weathered argillaceous rocks, particularly weathered mudstones/siltstones, can be calculated using methods described in section 54.3.1 – pile capacity in fine-grained deposits – although care is needed to suitably account for remoulding of material at the pile–ground interface.

For bored cast-in-place piles (replacement piles) a number of correlations between the unconfined compressive strength of the rock and rock socket skin friction have been proposed. The most common correlations are those proposed by Horvath (1978), Rosenberg and Journeaux (1976), Williams and Pells (1981) and Rowe and Armitage (1987).

Whitworth and Turner (1989) summarised the various proposed formulae for calculating ultimate unit shaft friction (see **Table 54.8**):

Pile design method	Ultimate unit shaft friction, q_s (kN/m²)
Horvath (1978)	$0.33(q_{uc})^{0.5}$
Horvath and Kenney (1979)	$0.20–0.25(q_{uc})^{0.5}$
Meigh and Wolski (1979)	$0.22(q_{uc})^{0.6}$
Rowe and Armitage (1987)	$0.45(q_{uc})^{0.5}$
Rosenberg and Journeaux (1976)	$0.375(q_{uc})^{0.515}$
Williams and Pells (1981)	$\alpha\beta(q_{uc})$

Table 54.8 Typical correlations between q_{wc} and ultimate shaft friction

here, q_{uc} is unconfined compression strength, α is a reduction (adhesion) factor relating to unconfined compression strength and β is a correction (rock socket side friction) factor related to discontinuity spacing within the rock mass.

Clearly, the above correlations can yield a wide range of values for ultimate shaft friction. Some of these differences may be due to the construction methods adopted for test piles, different back analysis methods to derive the correlations, or difference of the rock materials and/or rock masses in which back analysed test piles were constructed. A common approach is to take an average value from each of the correlations to gain a starting position in terms of design values and use additional information from historical pile testing in a given geographical area or rock formation for similarly constructed piles to those under consideration.

It must be noted that particular care needs to be taken when calculating design values for rock socket friction for mudstones. Experience has shown that the calculated values for unit shaft friction based on mudstone rock strengths are rarely achieved in practice, largely due to softening/polishing effects during rock socket construction.

The most recent published guidance for pile shaft friction in chalk is based on an effective stress approach. Reference should be made to CIRIA report PR86 (Lord *et al.*, 2003) for specific guidance.

54.3.7.2 Base capacity (applicable to both displacement and replacement piling techniques)

Where piles are driven to practical refusal on strong rock the limiting factor on pile capacity is usually the maximum allowable direct stress on the pile cross-section. In the UK this is typically taken as $0.25 \times f_{cu}$. Care should be taken when driving pre-cast concrete piles to ensure that the piles are not overdriven, as this could result in damage to the pile section. When driving steel piles, care should be taken not to overdrive the pile, as this could cause shattering of the bearing stratum, resulting in a reduced pile capacity. Care should also be taken to ensure that the design assumptions relating to rock material (e.g. strength) are compatible with the rock mass (e.g. joint infilling, joint orientation, joint aperture, solution features, weaker underlying strata).

Bored piles can also be constructed to practical refusal on strong rock. The definition of practical refusal should be agreed in advance with the engineer where it is anticipated this will occur. In certain circumstances the definition of practical refusal may be qualified subject to undertaking satisfactory maintained load tests.

For both driven and bored piles constructed within rock, the ultimate unit end-bearing capacity is often taken as the design UCS value for the rock stratum, i.e. allowable unit end-bearing $q_{b\,all}$ = UCS/FoS. The factor of safety is usually a reasonably high value, typically 2.0–2.5, which will ensure that the rock present at the toe of the pile will not become overstressed, even for predominantly end-bearing piles, again subject to maintaining compatibility between rock material and rock mass.

Where piles derive a significant proportion of their capacity from shaft friction, there may be less concern over consideration of the end-bearing component of capacity in terms of the pile load–settlement behaviour, particularly if the pile has a factor of safety on shaft capacity alone of 1.5 or greater.

Where a significant proportion of the pile capacity is being derived from the end-bearing component, care must be taken to ensure that the pile toe remains in intimate contact with the rock. Factors such as soil heave (from overburden removal or driving further displacement piles) and disturbance due to breaking down the piles can lead to lifting of the pile toe,

which will result in greater settlements upon loading than would otherwise have been experienced. For driven piles it is often considered to be good practice to re-drive piles (subject to programme and access constraints) to ensure the toe of the pile is in good contact with the bearing stratum.

Sloping bedrock can pose particular risks to pile construction. Driven and bored piles may be deflected by steeply bedded rock surfaces. Driven piles may be installed to a refusal criterion yet upon loading cause instability of the inclined rock mass due to downward sliding of blocks of rock. Infilled quarries can pose a particular risk to piling: the position of the highwall is critical in assessing the required pile lengths and suitability of particular construction techniques. It may not be possible or practicable to construct piles in the zone where highwalls are located.

When constructing bored piles within rock, care should be taken to ensure that the base of the pile bore is thoroughly cleaned, so that the design end-bearing capacity can be achieved without adversely affecting the piles' load–settlement behaviour. This can only be realistically achieved when large diameter bored piling techniques are adopted, facilitating the use of cleaning buckets. Typically, large diameter bored piling ranges from 600 mm to 3 m diameter.

Reference should be made to CIRIA report PR86 (Lord *et al.*, 2003) for specific guidance on constructing piles within chalk.

54.4 Factors of safety

54.4.1 Global safety factors

Historically, the design of piles in the UK has adopted global factors of safety where the safe working load, SWL, is given as

$$\mathrm{SWL} = Q_b/F_b + Q_s/F_s. \tag{54.16}$$

Although overall factors of safety are not necessarily the best way to govern pile performance, the numbers traditionally adopted are:

- 2.0 overall minimum – usually with load test verification on expendable piles;
- 2.5 overall or 1.5 (on shaft) with 3.0 (on base) – usually with working pile load tests;
- 3.0 overall – may not require any load testing.

According to BS8004, for spread foundations a factor of safety of 3 is usually adopted, and for piles a value of between 2 and 3 is normally used, the lower value representing conditions where there is reliable information on the ground and pile behaviour. The above factors of safety should only be used as a guide and different values and combinations can be used in conjunction with suitable experience.

There are a number of ground conditions where different factors are used, placing more reliance on either end-bearing or shaft friction. Examples include piles in chalk, where large factors on end-bearing are adopted, as high as 10, to allow for the potential for soft spots and dissolution features beyond the pile toe. Similarly the design of bored or CFA piles in weak rocks may often be dictated by ensuring a factor of safety as high as 1.5 on the shaft capacity alone to allow for the potential for machining/polishing or softening/remoulding of the pile shaft, or disturbance of the pile base, and to limit settlements at working load.

If a pile relies predominantly on end-bearing, then great care will be required both for design and construction, and more specialist techniques and expertise will normally be required. Site-specific preliminary pile tests may be needed unless there is significant previous experience in similar ground conditions and using similar piling techniques.

54.4.2 Partial safety factors

Partial factors are commonly used in structural design and these are applied to the particular parameters and loading conditions to reflect more accurately where the uncertainties lie. Most modern design codes, in particular the Eurocodes such as EC7 (BS EN1997 Part 7), recommend pile design based on the use of partial safety factors. This code does allow reduced factors of safety where there are several pile tests on a site. Such an approach encourages the use of preliminary pile testing.

The factor of safety, whether partial or a lump factor, is a factor on strength. However, as reported by Atkinson (2000), the key parameter that governs pile behaviour is stiffness of both the ground and the pile. The factor of safety is only of relevance in determining the likelihood of satisfactory performance beyond working load. For example, an end-bearing pile can fail to meet the required performance at working load because of low stiffness and still have the prescribed factor of safety. Conversely it can perform at working load and still not meet the required factor of safety. In either case it is necessary to determine whether the behaviour is of sufficient concern to be detrimental to the structure.

The main difficulty is in selecting criteria for settlement and factors of safety that are compatible and allow for the likely variability of the ground. This can only be done by making a prediction of pile behaviour based on the best estimate of the soil parameters and allowing a reasonable margin in the permitted test criteria to allow for natural variability in the ground.

54.5 Pile settlement

Structures do not generally suffer due to total settlement *per se*. It is differential settlement of, and the vulnerability of services connected to, structures that give rise to most concern. Refer to Chapter 52 *Foundation types and conceptual design principles*.

One of the reasons for introducing a lumped factor of safety when considering allowable pile capacity (such lumped safety factors applied to calculated ultimate pile capacities are most often in the range 2.0–3.0) is to ensure that settlement of single piles under working load (i.e. at serviceability limit state) is within tolerable limits for most structures.

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A specified factor of safety does not guarantee a specific settlement and it is important to review the risk of excessive settlement as part of the pile design process. For most piling techniques in a range of ground conditions, except those that rely predominantly on end-bearing, it is good practice to ensure a suitable factor on the shaft capacity alone to limit settlements under working loads, i.e. to ensure suitable serviceability performance. It is recommended that the settlement of end-bearing piles is always assessed. Group settlements will need to be subject to separate consideration to ensure that pile group settlement is tolerable for the structure (refer to Chapter 55 *Pile-group design*).

It is widely acknowledged that more settlement is required to mobilise end-bearing than shaft friction. It is also generally to be expected that piles which rely significantly on end-bearing (carrying a low proportion of the total applied load in shaft friction), will settle more under load than piles which are predominantly frictional piles (i.e. piles which carry the majority of applied load by shaft friction). Patel (1992) has summarised pile test data for friction piles in London Clay (**Figure 54.8**), which indicates that at typical working loads (factor of safety of 2 or more) the single pile settlement would be expected to be less than about 1.0% of the pile diameter. Similar behaviour would be expected for comparable stiff overconsolidated fissured clays (Gault Clay, Weald Clay, etc.).

An additional factor which must be considered in any assessment of pile settlement is the magnitude of pile head displacement caused by elastic shortening of the pile under load. This can be a significant proportion of the total pile head settlement under load, but becomes increasingly significant when considering long slender piles constructed through deep weak deposits (i.e. transmitting a significant proportion of applied load to competent strata).

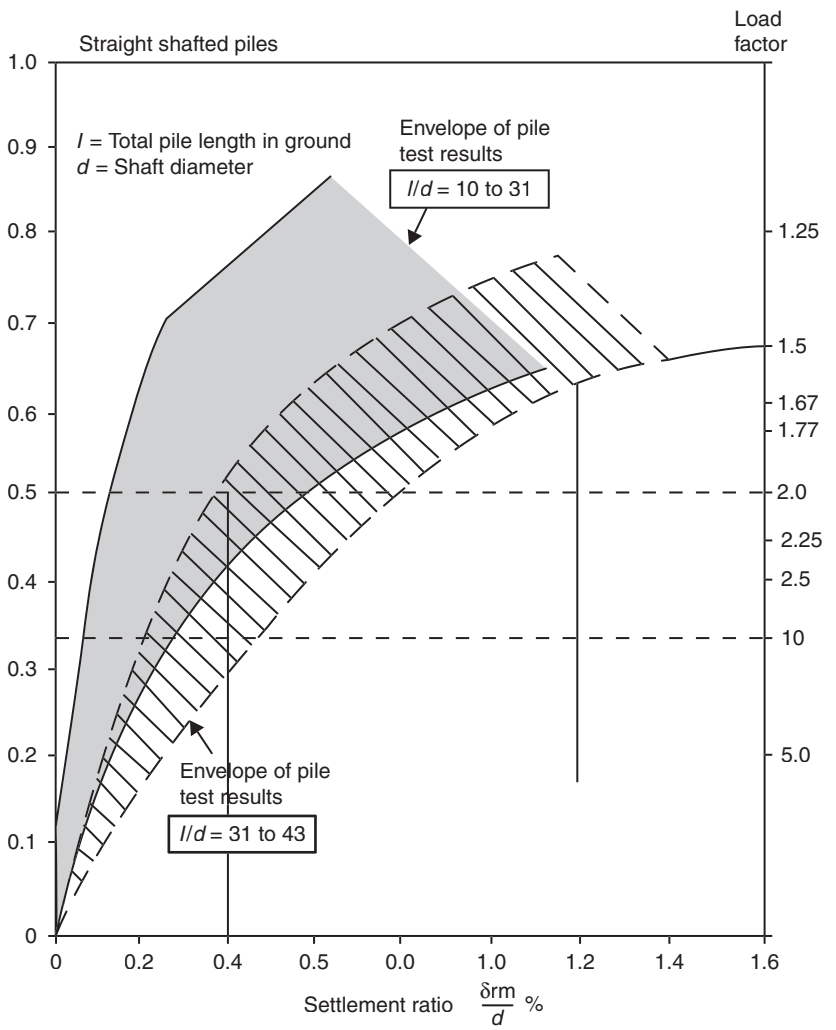


Figure 54.8 Results of loading tests on straight shafted bored piles in London Clay – normalised load settlement chart
Reproduced from Patel (1992)

A number of different methods have been proposed over recent years: elastic approaches (e.g. Poulos and Davis, 1968; Tomlinson, 2004), simplified empirical methods (e.g. Burland *et al*., 1966), finite element/boundary element analyses, and hyperbolic analytic methods (e.g. Fleming, 1992).

Burland's proposed simplified empirical approach is specifically for the assessment of settlement of end-bearing under-reamed piles in London Clay (Burland *et al*., 1966). Burland showed that where plate load tests are taken to failure, the normalised load settlement curve can be plotted, and provided the base factor of safety exceeds 3, the settlement of the pile base can be obtained for any diameter of pile base. Full details of this empirical method are provided in Burland *et al*.'s paper. Alternatively reference may be made to Tomlinson (2004).

In his paper, Fleming (1992) noted that sophisticated analytical techniques require sophisticated input data (e.g. certain geotechnical parameters) which are not usually available from routine site investigations. The method described by Fleming combines hyperbolic functions describing both the shaft and base components of pile capacity, together with the component of pile settlement derived from elastic shortening of the pile. The method takes advantage of the fact that the hyperbolic function adopted requires definition only of its origin and either its initial slope or a single point. The shaft flexibility parameter, M_s, does not vary significantly, with M_s values typically between 0.0010 and 0.0025 for many pile types and ground conditions. However, as expected the base stiffness parameter, E_b, varies across a wide range for different pile and soil/rock types, and is particularly sensitive to pile construction methods and workmanship. E_b, should not be confused with other 'elastic' soil parameters which are quoted in the literature: it is essentially an empirical parameter derived from back analysis of pile tests and is as sensitive to pile construction method as to the fundamental soil properties. Guidance on appropriate parameters is given in Fleming (1992). The definition of axial capacity adopted within Fleming's method is the plunging-failure (or asymptotic) definition, i.e. that load at which the soil resistance is fully mobilised and where settlement continues indefinitely for the load magnitude reached. Corke *et al*. (2001) provide interesting practical examples of Fleming's method, in the context of developing pile testing performance criteria.

Fleming's method is a powerful and simple method for single pile settlement analysis; it is especially useful for large diameter piles (say in excess of 1.0 m diameter) and for end-bearing piles.

Fleming's method has a significant additional benefit, in that it can be used in reverse to back analyse the results of maintained load tests to determine shaft friction, end-bearing and elastic shortening of the pile, subject to the pile test having moved the pile sufficiently to mobilise end-bearing sufficiently, typically more than about 5% of the pile diameter.

54.6 Pile behaviour under lateral load

54.6.1 Introduction

Piles can be subjected to lateral loads arising from different sources. These sources typically fall into two main categories: *active* loading (where external forces are applied to the pile) and *passive* loading (where movement of the soil immediately adjacent to the pile imposes bending stresses on to the pile). A more refined classification of the sources of lateral load may involve the following (after Elson, 1984):

- Static: e.g. structural reactions, lateral earth pressure.
- Dynamic – cyclic: e.g. from operating reciprocating equipment, earthquake, cyclic wind loading (wind turbines), wave action.
- Dynamic – transient: e.g. general wind loading, berthing, vehicular braking (impact) loads.
- Other: e.g. undrained soil movements, soil consolidation adjacent to piles, thermal movements, soil creep, soil shrinkage/expansion (soil moisture changes).

Geotechnical capacity is rarely an issue even in weak/soft soils. The issues are usually either ones of serviceability (i.e. pile deflection/deformation) or structural capacity of the pile. Passive loading is a particularly high risk in soft soils, where large global ground movements may readily develop (refer to Chapter 57 *Global ground movements and effects on piles*).

Piles subjected to cyclic loading require particularly close attention since cyclic loading will tend to modify soil behaviour and can lead to progressive failure. Consideration of dynamic cyclic and dynamic transient loads falls outside the scope of this chapter of the manual (refer to Chapter 60 *Foundations subject to cyclic and dynamic loads*).

54.6.2 Deformation/failure modes due to 'active' loading

Piles subjected to lateral loads will experience an increase in normal stress in front of the pile (in the direction of the lateral load), and a decrease behind the pile. Pile deformation is usually limited to approximately 10 pile diameters below ground level; the concept of a critical pile length (which is dependent upon the ratio of soil/pile stiffness) is particularly useful in this context (reference should be made to Fleming *et al*., 2008).

There are two principal failure modes for unrestrained (or free headed) single piles subjected to 'active' lateral loading (**Figure 54.9**). Unrestrained short rigid piles will fail by rotation when the passive resistance of the soil (in front of the pile above the point of rotation, and behind the pile below the point of rotation) are exceeded. Long piles will tend to fracture at some depth down the pile shaft. For the purposes of analysis a plastic hinge is assumed to develop which is capable of transmitting shear. The upper portion of the pile shaft above the plastic hinge will deform whilst the lower pile shaft, below the hinge, will not move significantly.

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For restrained (or fixed headed) piles (**Figure 54.10**) there are three possible failure modes:

(a) translation;

(b) 1-hinge failure (at underside of pile cap/restraint);

(c) 2-hinge failure (at underside of pile cap/restraint and at some depth).

54.6.3 Fixed and free headed piles

Fixed headed piles are restrained at head level, typically by a pile cap (pile groups of three or more) or ground beams (pairs of piles or single piles) or potentially by a structural slab (if so designed). Piles need to be restrained in two orthogonal directions to be considered truly fixed headed.

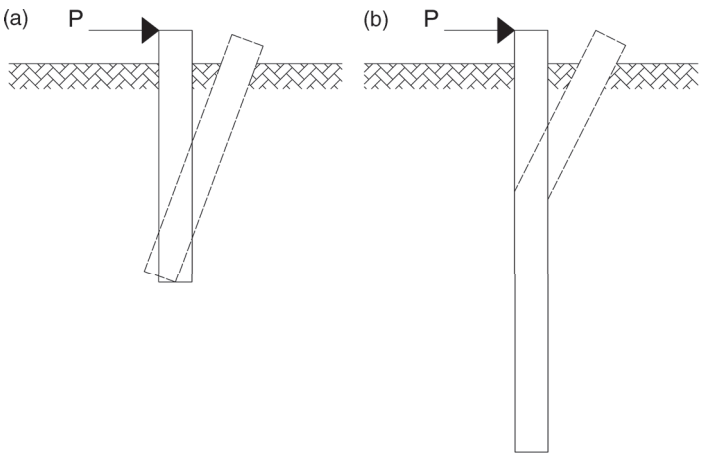


Figure 54.9 Possible failure modes for free headed piles: (a) rotation; (b) hinge failure (Broms, 1964a,b)
Courtesy of Cementation Skanska

Free headed piles are free to rotate and are not subject to orthogonal restraint.

In free headed piles the bending moment in the pile at ground level is positive and acts in the same direction as the applied lateral load.

In fixed headed piles the bending moment in the pile at ground surface is negative and acts in the opposite direction to the applied lateral load.

54.6.4 Lateral resistance of single piles

The ultimate lateral resistance of single piles can readily be estimated from the solution proposed by Broms (1964a, 1964b). Whilst Broms's solution may be considered to be on the conservative side, it is often recommended for routine design given the relative simplicity of the solution and the fact that the governing criterion is rarely the ultimate lateral soil resistance.

Other authors have provided more detailed/refined solutions. Full details of these solutions can be found in CIRIA Report 103 (Elson, 1984), Appendix A.

Once the bending moment and shear force envelopes have been derived adopting appropriate analysis methods, the pile can be designed in accordance with relevant structural design codes of practice (e.g. BS EN1997 Part 2).

54.6.5 Lateral deformation

When considering the lateral deflection of laterally loaded piles, the near-surface soil parameters merit particular attention. This can often cause some problems since the very near surface deposits (generally the upper 2–3 m) are often those about which very limited information is obtained during the ground investigation.

Lateral pile loading tests are occasionally specified to be undertaken on piles. It is the authors' experience that these are often specified without full cognizance being taken of the head

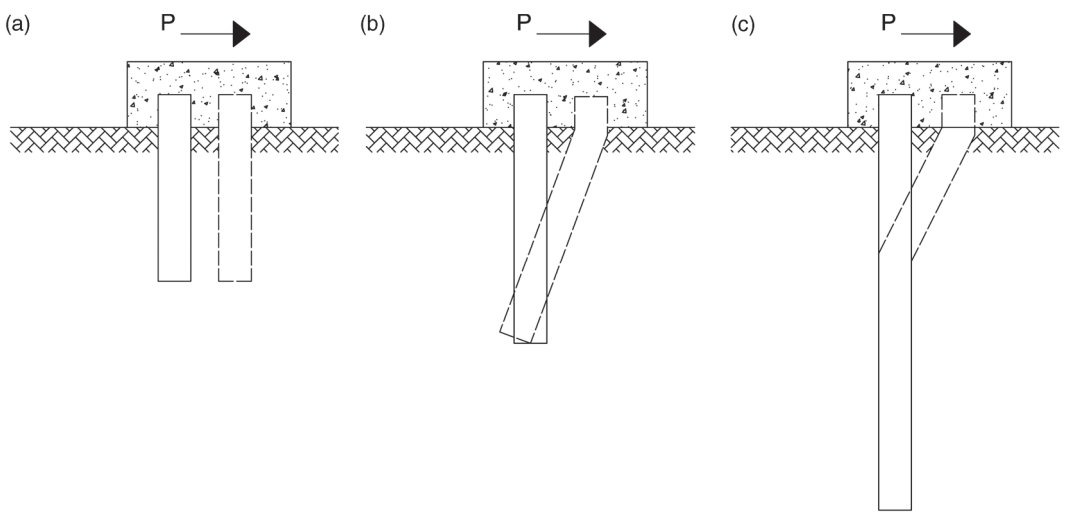


Figure 54.10 Possible failure modes for fixed headed piles: (a) translation; (b) 1-hinge failure; (c) 2-hinge failure (Broms, 1964a,b)
Courtesy of Cementation Skanska

fixity of the piles in the final construction. The lateral load–deformation behaviour of a fixed head pile is very different from that of a free headed pile, and this must be taken into account when considering how the results of a lateral pile test (usually free headed) will be analysed and used for the design of permanent piles (usually fixed headed). The analysis method used for lateral pile test interpretation and permanent works design must be sophisticated enough to allow for:

(i) potential changes in pile head fixity;

(ii) potential differences in load eccentricity between pile test and permanent works;

(iii) potential changes in near-surface ground conditions between pile test and permanent works.

If there is a coherent testing and analysis strategy then lateral load testing of piles, particularly those whose load deflection behaviour is deemed to be of significant importance, or where particularly high lateral loads will be applied to the pile (relative to the ground conditions and the pile geometry), can provide a very useful check on performance of piles and the analysis techniques adopted for pile design.

The two analysis approaches commonly adopted for calculating the lateral displacement of single piles subjected to lateral loads are the sub-grade reaction approach and the elastic continuum approach. Lateral load–deformation behaviour is strongly nonlinear even under moderate lateral loads, and linear elastic methods are usually inappropriate for piles subject to significant lateral loads. Ignoring nonlinear behaviour can lead to grossly inaccurate and unconservative predictions of lateral deformation.

The sub-grade reaction approach models the ground as a series of discrete springs along the length of the pile shaft, i.e. as a Winkler idealisation of the soil mass. A refinement of the early models allows the spring stiffness to vary along the length of the pile (Reese and Matlock, 1956) and a further refinement is the p–y form of the Winkler soil model (Matlock, 1970; Reese *et al*., 1974, 1975). The disadvantage of the p–y form of the model is that selection of reliable p–y curves requires careful judgement, although typical forms of the curves have been presented by Reese *et al*. (1974) in sand and by Matlock (1970) in clays, and are summarised in the API code (American Petroleum Institute, 2000).

Both Poulos (1971) and Randolph (1981) have published solutions relating to the behaviour of piles under lateral loads based on elastic continuum models. One principal advantage of the elastic continuum models over sub-grade reaction models is that the method can be extended to pile groups, since interaction between adjacent piles can be modelled. Where large lateral loads are required to be resisted by piled foundations, for example marine/offshore structures or complex/unusual superstructures, it will be more appropriate to assess lateral displacements by adopting nonlinear elastic plastic methods. A useful practical procedure for estimating lateral load–deformation behaviour, proposed by Brettmann and Duncan (1996), is the 'characteristic load' method. Parameters are provided for sands and clays, and for free and fixed headed piles.

54.7 Pile load testing strategy

Strategies for pile load testing often lack clear objectives. The requirements for testing are frequently driven by a desire to comply with regulations and to follow common practice. Testing is rarely seen as a part of a value engineering process, and therefore opportunities to optimise the foundation solution are often missed.

It is important that load testing of piles is considered as early as possible and included in the initial cost plan and programme stage. The programme must allow sufficient time for suitable tests to be carried out and for an objective appraisal of the test results and subsequent design revisions or value engineering to be carried out.

Without such an early review to set clear load testing objectives, combined with poorly specified requirements, avoidable problems will arise. Common problems include:

- inappropriate test method specified and lack of flexibility in the testing regime;
- final working conditions not modelled by the load tests;
- piles not tested to failure or sufficient load to allow suitable evaluation;
- insufficient time to carry out tests and to evaluate the results;
- unrealistic/unachievable performance criteria specified;
- no provision for value engineering;
- unrealistic expectations of the possible benefits of load testing.

54.7.1 Objectives

The strategy for pile testing needs to be established at the time the piles are designed. For most projects the main purpose of pile testing is to validate the design before construction and to check compliance with the specification during construction. However, in many cases there are considerable benefits in using testing for design development in order to identify the optimum solution. Testing strategies can therefore be divided into four main categories:

- design development;
- design validation;
- quality control;
- research.

The scope of testing will depend on the complexity of the foundation solution, the nature of the site and the risk of piles not meeting the specified requirements. The designer therefore needs to assess the risks and develop the testing regime accordingly. So what are the main considerations when assessing the risks?

- Site investigation data – are they sufficient?

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- Previous knowledge – are there similar piles that have been tested on adjacent sites?
- Time – is there enough time to verify the design? If not, a conservative approach is needed.
- Cost – can the testing provide cost savings? What are the cost and/or construction programme implications of tests failing?

For simple structures on a site where the ground conditions are well understood and there are test data from adjacent sites which have used similar piling solutions then the risks are low and the testing can usually be restricted to routine checks for compliance with the specification. For situations where the ground conditions or structural requirements are complex and there is little experience of similar piling work, then careful evaluation of the piling proposals is essential prior to embarking on the main piling works. The testing regime therefore needs to be considered in two phases, comprising preliminary pile testing before the main piling works, and testing of working piles.

54.7.2 Preliminary pile tests

Preliminary testing is carried out in advance of the main piling works to verify that the design assumptions and construction method will achieve the required performance. The testing strategy for preliminary pile testing should address a specific set of stated objectives, which should include the following:

- minimise risk – uncertainties about ground conditions, contractor's experience, novel techniques;
- optimise the design – pile diameters, lengths and factors of safety;
- confirm pile installation criteria – founding strata identification, pile set, refusal criteria;
- assess buildability – site variability, pile integrity, pile uplift and displacement, soil remoulding;
- check the pile performance meets the structural requirements – load/settlement behaviour, compression, tension and lateral loading;
- confirm safety procedures – piling platform, working method, inspection, testing methods;
- assess environmental impact – noise, vibration, pollution.

Where piles are required to carry very heavy loads, it may be impractical to carry out full-scale load tests. In such circumstances, consideration should be given to carrying out tests on smaller diameter piles using the same method of construction, provided the results of the tests can be extrapolated with some degree of confidence to predict the load settlement characteristics of the larger piles. The test piles should be founded at the same level and in the same soil as the works piles.

For preliminary pile tests it is essential to have a borehole and/or CPT close to the test pile so that the test results can be reliably evaluated.

54.7.3 Working pile tests

Working pile tests are carried out during the main piling works to verify that the workmanship and materials meet the specified requirements. Testing on the working piles comprises a range of quality control checks. These checks will normally include load tests, integrity tests and materials testing.

The strategy for testing working piles should address the following issues:

- load/settlement behaviour – if settlement criteria are specified then load tests should be carried out;
- risk of integrity problems – the number of tests should reflect that certain ground conditions and pile systems may have increased risk;
- installation criteria – use pile records, sets and dynamic testing where appropriate to confirm the installation criteria are being met.

The testing strategy for working piles should relate to the level of risk and the characteristics of the piling project. Test results and pile records should be continually reviewed during the works to reassess the risk level.

54.7.4 Number of tests

The need for preliminary testing in advance of the working piles, or in advance of new areas of piling on large projects, should be weighed against their cost and the time taken for installation, testing and analysis of results. It may be more economical and more logical to adopt a more conservative approach to design, for example by using larger or longer piles or reduced pile loads. Also, it may be more beneficial to undertake further site investigation to obtain information which is key to the success of the installation and hence achieve the necessary confidence in the piling. The value of preliminary testing undertaken to prove the design approach or pile performance is wasted unless the ground conditions are sufficiently well defined to allow the design to be applied to piles elsewhere on site.

The level of testing of working piles should consider the variability of the ground conditions from the point of view of the demands these place on the control of pile construction, area by area. Ensuring experienced supervision and monitoring, plus good pile records, can be preferable to increasing the level of control testing above the typical 1% of all piles installed.

54.7.5 Methods of load tests

The various available methods of testing piles are best characterised by the duration that the force is applied to the pile and the strain induced in the pile. Tests involving large forces applied for several minutes or hours, such as static load tests (also known as maintained load tests), are used to assess pile load capacity, and small-energy low-strain tests are used to assess pile integrity. In dynamic and kinetic tests, although the force is comparable in magnitude to a static test, it is applied over a much shorter period than in a static load test. Careful consideration is therefore needed in the interpretation of the dynamic

effects in order to derive static load capacities. The various test methods and their applications are detailed in Chapter 98 *Pile capacity testing* of this manual. There are a number of published documents dedicated solely to the subject of pile testing, such as Weltman (1980) and Handley *et al.* (2006), and these should be referred to for more detailed information on the subject of pile load testing. Chapter 98 *Pile capacity testing* of this manual focuses specifically on pile load testing.

54.8 Definition of pile failure

The definition of axial load capacity (or pile 'failure') needs to be clearly understood, since this can be the cause of confusion. Unambiguous definition becomes very important for piles which rely mainly on end-bearing resistance. Two definitions are commonly used:

(i) Ultimate capacity, 'plunging failure' – failure is defined as the load at which settlement continues indefinitely, shown as P_u in **Figure 54.11**. This is often a theoretical concept rather than a practical reality as most pile tests are not taken to sufficient load to reach this value. Calculation methods based on bearing capacity theory will usually derive a 'plunging failure' load.

(ii) Deformation controlled capacity, '10% failure load' – because the 'plunging failure' load cannot often be practically reached and measured during a pile test. Most codes/standards define failure as the load which is measured at a settlement equal to 10% of the pile base diameter, shown as $P_{F0.1}$ in **Figure 54.11**.

For friction piles, there may be little practical difference between the above definitions. However, for end-bearing piles there can be a considerable difference between the capacities derived by (i) and (ii) above. Referring to **Figure 54.11** Pile A and B both have the same 'ultimate' capacity, but in terms of a deformation controlled capacity (consistent with most code definitions of failure) Pile A has a higher capacity than Pile B. For end-bearing piles, the deformation controlled capacity will be a function of the mobilised deformation modulus of the ground immediately beneath the pile toe.

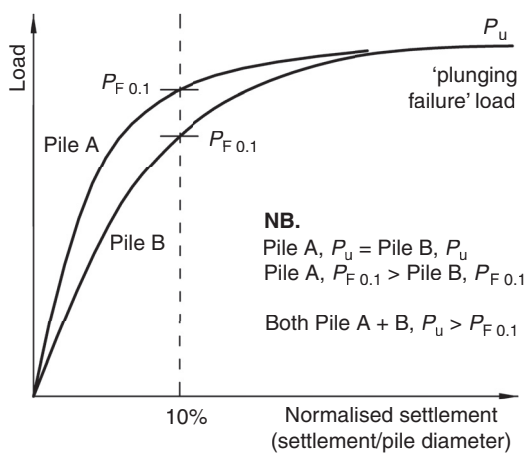


Figure 54.11 Definitions of pile failure

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It is recommended this chapter is read in conjunction with

- Chapter 19 *Settlement and stress distributions*
- Chapter 22 *Behaviour of single piles under vertical loads*
- Chapter 82 *Piling problems*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 55
Pile-group design

Anthony S. O'Brien Mott MacDonald, Croydon, UK

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Pile design often focuses on ultimate geotechnical capacity. When piles are located within a pile group a wide range of additional factors need to be considered, including the deformation of the pile group and the structural forces induced in the piles and pile cap. Interaction between adjacent piles depends on several factors, including the direction of applied loads. When large horizontal loads are applied, a complex range of interaction effects may need to be considered. Computer-based analytical methods are commonly used; nevertheless, it is important to use simplified methods of analysis both for preliminary design and to judge the reliability of output from computer-based methods. Pile-group deformation is usually more critical than group capacity, and, hence, the selection of appropriate deformation moduli is particularly important. Case history data and predictions from nonlinear modelling are compared with linear elastic methods. The delegation of design responsibility for pile-group design requires careful judgement and is also discussed.

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55.1 Introduction

The design of single piles is discussed in Chapter 54 *Single piles*, where the main focus is the assessment and verification of the ultimate geotechnical capacity of the pile. For pile groups a wider range of factors will need to be considered. The design must consider the overall performance of the whole foundation system (**Figure 55.1**). It is often the case, especially for larger pile groups, that pile-group deformation and the structural forces (bending moments and shear forces) induced in the piles are the main design issues, rather than ultimate geotechnical capacity. A pile group will stress a much larger zone of soil than a single pile (**Figure 55.2**) and this will be extremely important when considering how to develop an appropriate analytical model and input parameters, and assessing potential failure and deformation mechanisms. For example, if there is a soft compressible layer at depth below the pile toe, it may have little effect on single-pile behaviour, but could cause excessive pile-group deformation or even collapse. The overall behaviour of pile groups is affected by their geometry. An important parameter is the pile-group aspect ratio R (Randolph and Clancy, 1993) defined as $R = (ns/L)^{0.5}$, were n is the number of piles in the group and s and L are the pile spacing and length, respectively (both in metres). When the pile-group aspect ratio is large (**Figure 55.3(a)**), then most of the applied load is likely to be shed into the ground via compression of the deposits below the pile-toe level. In contrast, if R is small, **Figure 55.3(b)**, then most of the applied load is likely to be shed into the ground via shear around the pile-group perimeter.

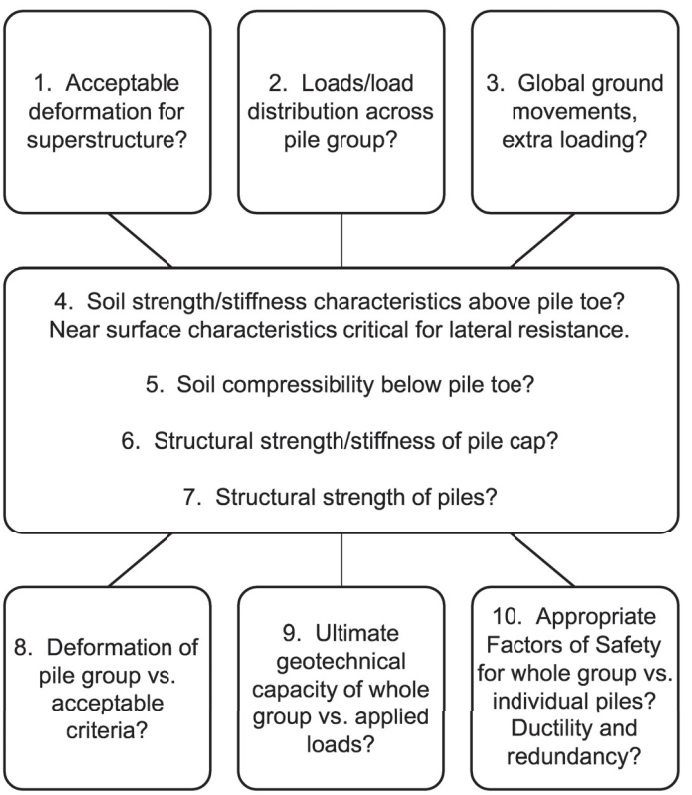


Figure 55.1 Design considerations for pile groups
Modified from O'Brien and Bown (2010)

In modern engineering practice, the analysis of pile-group behaviour is usually based on commercially available computer software. However, most software has a number of limitations, namely:

(i) soil behaviour is assumed to be linear elastic;

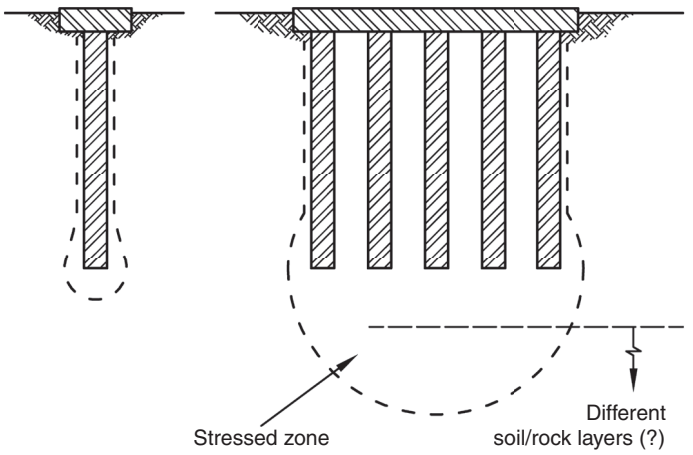


Figure 55.2 Comparison of influence zones beneath single piles and pile group

(ii) the influence of soil layering below the pile-toe level may be poorly simulated.

Analytical methods for pile-group analysis are often complex, therefore, the main intent of this chapter is to highlight:

(i) the key factors that will affect pile-group behaviour;

(ii) several simplified analytical methods, which can be useful either for preliminary design or to check computer output;

(iii) when linear elastic methods can give output that is appropriate for practical design, and when more complex methods are necessary;

(iv) how selected output, in particular axial load distributions, can be interpreted to facilitate more economic design.

55.2 Pile-group capacity

Independent calculations should be made of ultimate capacity based upon:

(i) the sum of the individual pile capacities (based on capacity of isolated piles, see Chapter 54 *Single piles*), **Figure 55.4(a)**;

(ii) failure of the 'block' that encloses the perimeter of the pile group, **Figure 55.4(b)**;

(iii) failure of rows of piles, **Figure 55.4(c)**.

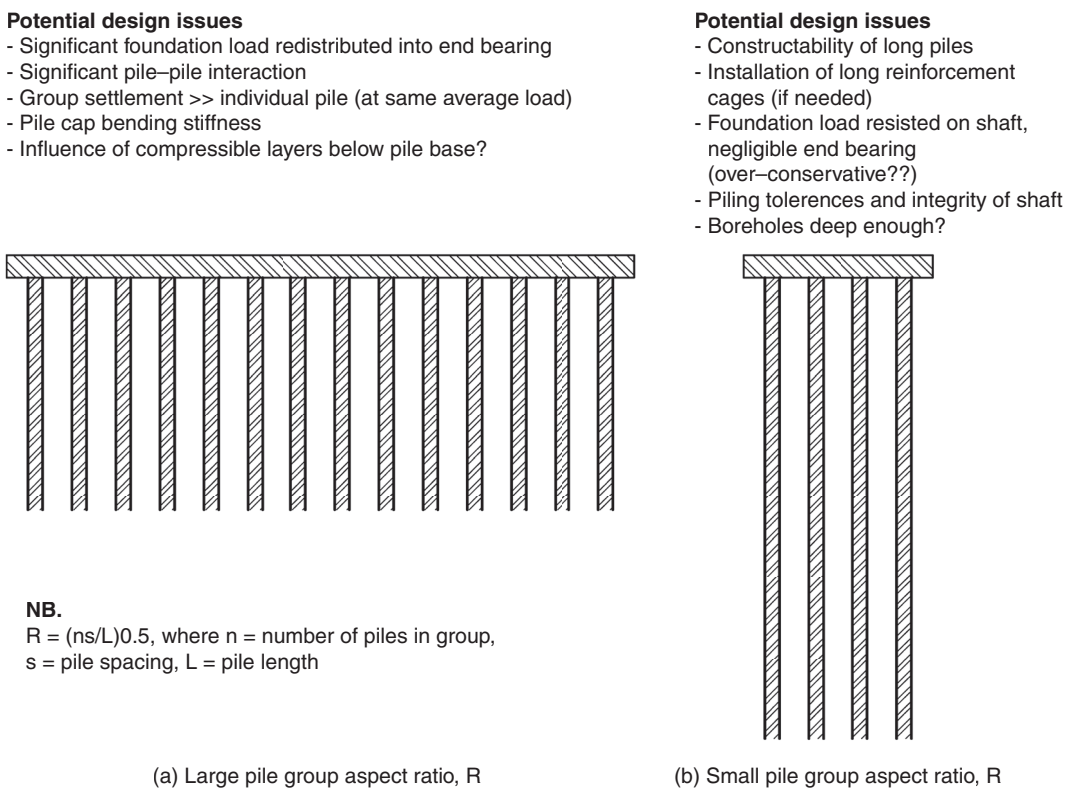


Figure 55.3 Pile-group aspect ratio, *R*, and potential design issues

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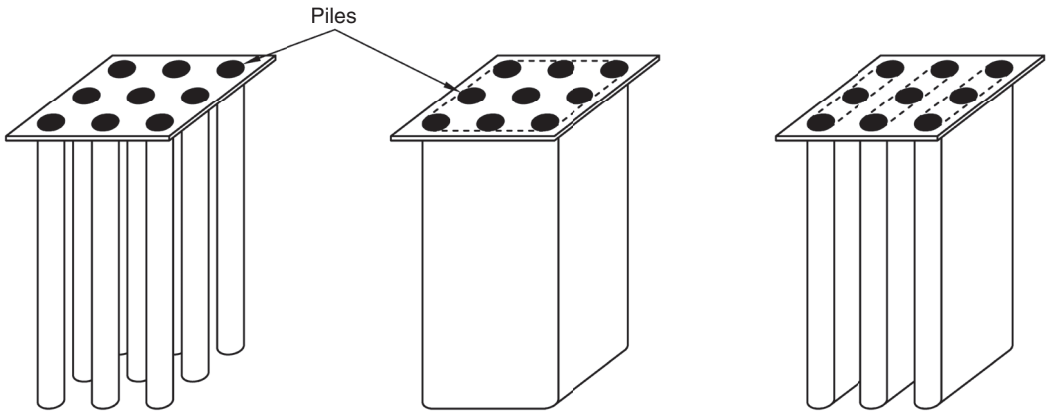


Figure 55.4 Failure modes for pile groups
Reproduced from Fleming *et al.* (2009)

For vertically loaded pile groups, usually only (i) and (ii) will need to be checked; failure mode (iii) becomes more important if the spacing between pile rows is variable or if the horizontal or moment loading is relatively large.

The failure of pile groups is rare; however, the following situations require particular care:

(i) When large global horizontal ground deformations develop around the pile group, usually due to out-of-balance loading on a soft clay or peat layer (**Figure 55.5(a)**), either due to placing fill or excavating adjacent to the pile group.

(ii) When a layer of soft clay underlies a thin competent layer, and end-bearing piles are toed into the competent layer (**Figure 55.5(b)**).

(iii) When end-bearing piles are toed in rock, which contains either dipping clay-filled discontinuities or voids (**Figure 55.5(c)**). Voids may arise in a rock mass either due to natural solution features, or due to historic mining activity.

In the author's experience, (i) above is the most common cause of pile-group failure (usually due to inadequate structural pile strength). This has occurred when the pile designers or constructors did not appreciate the potential impact of global ground movements and this failure mode was completely overlooked. Chapter 57 *Global ground movements and their effects on piles*, discusses the influence of global ground movements (both vertical and horizontal) on pile-group behaviour. This failure mode can often be avoided either by appropriate ground improvement (Chapter 59 *Design principles for ground improvement*) or by appropriate construction planning and sequencing. For example, installing the piles *after* the fill or excavation, adjacent to the pile group, has been completed.

The analytical methods used to assess ultimate pile-group capacity are based on conventional bearing-capacity theory (refer to Chapter 21 *Bearing capacity theory* and Chapter 53 *Shallow foundations*). Hence, for *vertical loading of a pile group in clay* the ultimate capacity of the block of soil encompassed by the group is:

$$Q = 2D\,(B + L)\,S_{\text{uave}} + S_{\text{ub}} f_s\, N_c\, BL \qquad (55.1)$$

where D is the pile length or embedded depth, B is the pile-group width, L is the pile-group length (i.e. the plan dimension), S_{uave} is the average undrained shear strength around the group perimeter, S_{ub} is the undrained shear strength beneath the group (the average between the pile-toe level and 0.5 B below the toe level for homogeneous conditions), f_s is the shape factor for the pile-group plan dimensions, B and L (**Figure 55.6(b)**), and N_c is the bearing-capacity factor allowing for appropriate depth and width (**Figure 55.6(a)**). **Figure 55.6** summarises the appropriate bearing-capacity and shape factors. Total stress methods are commonly used for clays. However, effective stress methods can be utilised (refer to Chapter 54 *Single piles*) and are recommended if deep excavations (say in excess of 4 m) are constructed above or adjacent to the pile group (since the excavation would lead to a reduction in the clay's mobilised strength in the long term).

The influence of an underlying compressible layer on the ultimate capacity of the group (acting as a block or equivalent pier, refer to section 55.5.4) can be assessed (based on studies by Matsui, 1993) as follows:

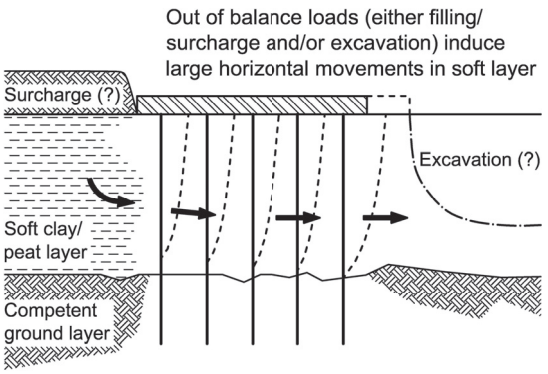
$$Q_b = Q_{u2} \qquad \text{for } Z_c/d_b \le 0.5 \qquad (55.2)$$

$$Q_b = Q_{u1} \qquad \text{for } Z_c/d_b \ge 3 \qquad (55.3)$$

and for Z_c/d_b between 0.5 and 3

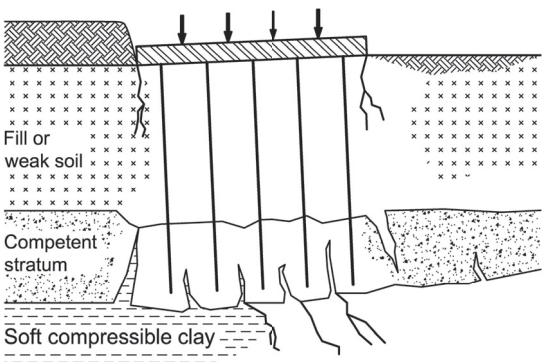
$$Q_b = Q_{u2} + (0.4\,Z_c/d_b - 0.2)\,(Q_{u1} - Q_{u2}) \qquad (55.4)$$

where Q_b is the ultimate bearing capacity of the base, Q_{u1} is the ultimate bearing capacity of the upper layer, Q_{u2} is the ultimate bearing capacity of the underlying weak compressible layer,

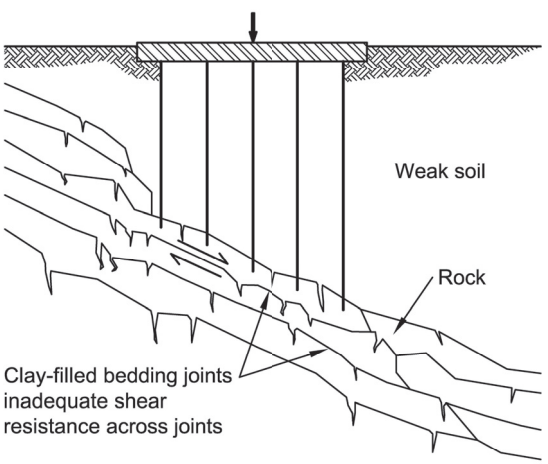


Ground deformation induces large bending moments/shear forces in piles

(a) Global ground movements induce extra loading and pile group deformation



(b) Load applied to group of piles causes shear failure of soft clay



(c) Pile group bearing on sloping rock surface, shear failure due to clay infill in rock joints

Figure 55.5 Pile-group failure, some high risk situations

Z_c is the depth of the underlying layer below the pile-group toe level and d_b is the equivalent pier diameter (**Figure 55.6(c)**).

If the upper layer is sand or gravel then Q_{u1} can be assessed by using conventional bearing-capacity theory based on effective stress parameters (refer to Chapter 21 *Bearing capacity theory* and Chapter 53 *Shallow foundations*) together with appropriate shape and depth correction factors. For weak rocks, the nature and spacing of discontinuities in the rock mass need to be considered, as outlined in Chapter 53 *Shallow foundations*. For clays, conventional bearing-capacity theory can be used, together with appropriate shape and depth correction factors, see (55.5) below.

$$\text{Ultimate bearing pressure, } q_b = f_s N_c S_{ub} \qquad (55.5)$$

where f_s, N_c and S_{ub} are defined above, for (55.1).

For pile groups subject to horizontal or moment loading and underlain by relatively weak layers, the potential for edge failure should be checked by assessing the bearing resistance beneath the outer (more heavily loaded) pile row. It is usually sufficient to calculate an average bearing pressure at the toe level beneath the row (based on the mobilised end bearing pressure). The factor of safety against local failure can then be checked against equations (55.2) to (55.5) above.

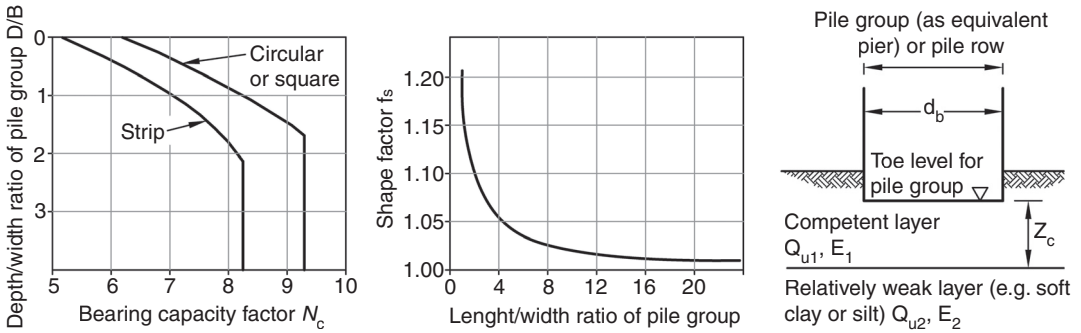
Historically, there was some concern about the capacity of a group of piles relative to the sum of individual pile capacities, and the term group 'efficiency' has been discussed in some textbooks. This concept has led to confusion. It should be noted that the efficiency factors quoted are based on a limited set of small-scale model tests, and general experience (together with full-scale tests, e.g. O'Neil *et al*., 1982) indicates that there is little merit in using efficiency factors. For piles driven into sand, the group capacity is likely to be much higher than the sum of individual capacities. However, settlement considerations will usually be critical and the actual ultimate capacity will not govern the design. For bored piles (in sands or clays), it could be argued (with some justification) that there would be an overall relaxation of horizontal ground stress around the pile group (thus reducing the pile-group capacity). However, this will be counterbalanced by the fact that shear failure would occur mainly through an undisturbed soil mass (rather than through the highly disturbed or remoulded ground adjacent to an individual pile shaft). In summary, therefore, the use of group efficiency factors is *not* recommended for calculations of ultimate capacity.

For pile groups subject to horizontal loading, the most likely geotechnical failure mode is shown in **Figure 55.4(c)**; commonly this will be more critical than the sum of individual pile capacities due to the 'shadowing' effect of adjacent piles in a row (**Figure 55.7**). The failure of pile rows under horizontal loading is discussed by Fleming *et al*. (2009).

For practical purposes the ultimate lateral capacity of a pile group can be estimated as the lesser of:

(i) the sum of the ultimate capacities of individual piles in the group (Chapter 54 *Single piles*);

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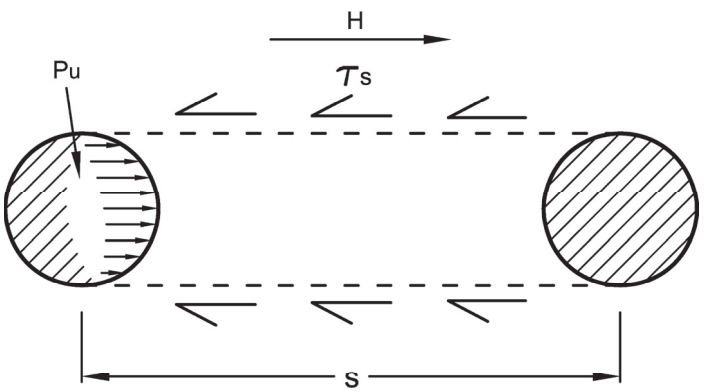


(a) Bearing capacity factor N_c (after Meyerhof)

(b) Shape factor for rectangular pile groups (Meyerhof-Skempton)

(c) Pile group (or row) founded on a competent layer over a weak layer (after Matsui, 1993)

Figure 55.6 Bearing-capacity and shape factors for pile groups
Modified after Tomlinson (1987)



NB.
Pu = ultimate lateral force, per unit length,
S = pile spacing, τs = ultimate shear stress,
H = horizontal force

Figure 55.7 Failure and deformation mechanism ('shadowing') for a pile row under a horizontal load

(ii) the ultimate capacity of an equivalent block containing the piles and soil;

(iii) the sum of the ultimate capacities of the pile rows.

For (ii) above, only the 'short-pile' case is relevant (Fleming *et al*. 2009), and failure mode (ii) is only likely to be critical when there is a large number of closely spaced and relatively short piles. For this scenario, the soil shear strength would only be ignored across a depth equal to 1.5 times the pile diameter.

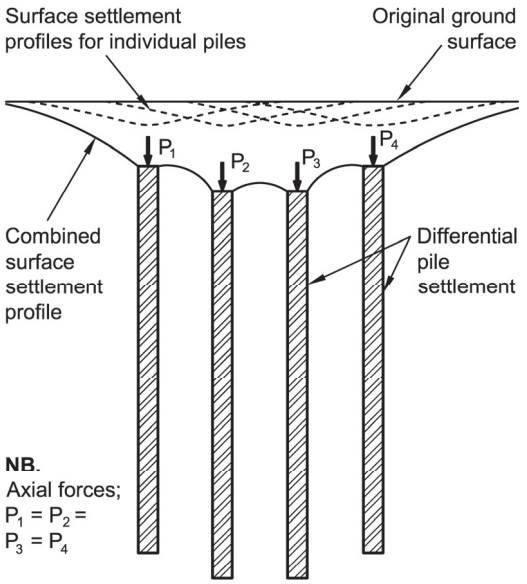
Pile-group failure under a lateral load will usually involve rotation as well as horizontal translation of the group. The ultimate geotechnical capacity will then be a function of both the axial and lateral capacities, with piles behind the axis of rotation failing in tension and piles in front of the axis of rotation failing in compression. In situations involving large horizontal or moment loading, the induced structural forces (bending moment, shear force, axial forces, tension or compression) and horizontal or rotational movements will usually be more critical than the ultimate geotechnical capacity. Pile-group deformation and induced structural forces will be discussed later in this chapter.

55.3 Pile-to-pile interaction: vertical loading

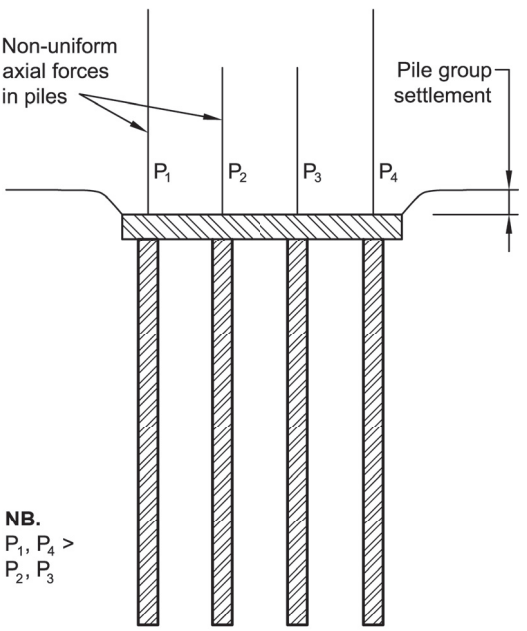
When piles are constructed close to one another, their subsequent load-deformation behaviour is affected by the presence of the neighbouring piles. **Figure 55.8(a)** provides a simple example of these 'pile-to-pile' interaction effects; it shows four piles under equal vertical loading (via a flexible pile cap). Each pile settles under the load and a zone of soil settlement develops around each pile. These soil settlement zones overlap and interact; hence, the middle piles settle more than the outer piles (and all the 'group' piles settle more than an individual isolated pile). If the pile group has a rigid pile cap, **Figure 55.8(b)**, each pile has to settle by an equal amount (because the pile cap is rigid) but the axial loads in the piles will now vary across the group with the outer piles developing higher axial loads than the middle piles (because the outer piles are stiffer than the inner piles and, hence, attract more load). Hence, the pile-to-pile interaction and how it affects both pile-group deformation and the redistribution of forces across piles (and into the pile cap) within a group is of fundamental importance. In practice, the pile-to-pile interaction depends on a large number of factors (**Table 55.1**).

In the context of assessing pile-group settlement, two parameters are useful:

$$\text{Settlement ratio, } R_s = \frac{\text{Average group settlement}}{\text{Settlement of single pile at same average load as a pile in the group}} \quad (55.6)$$



(a) Flexible pile cap, non-uniform pile settlement



(b) Rigid pile cap, non-uniform axial forces

Figure 55.8 Effects of pile-to-pile interaction, vertical loading

$$\text{Group reduction factor, } R_g = \frac{\text{Average group settlement}}{\text{Settlement of single pile at same total load as the group}} \quad (55.7)$$

R_g is conceptually difficult since it is physically meaningful only for an elastic soil when failure cannot occur. R_g is, however, useful when comparing the performance of different pile groups (of different geometries or pile numbers). R_g is simply related to R_s by $R_s = n\,R_g$, where n is the number of piles in the group. The value of R_g varies between $1/n$ and 1.0.

An important practical conclusion from various studies (e.g. Butterfield and Douglas, 1981) is that the precise layout of the piles has a negligible influence on the group settlement ratio R_S; for example a square pile group has the same value of R_s as a rectangular or circular group, at the same average pile spacing.

R_s is a convenient way of assessing the effects of interaction on pile-group behaviour. Two of the most important factors which influence R_s are:

(i) the variation of the ground stiffness with depth (which may be expressed in terms of the shear modulus G or Young's modulus E);

(ii) the influence of a rigid (or a relatively much stiffer or stronger) layer at some depth below the toe of the pile group.

The influence of these factors is illustrated in **Figures 55.9** and **55.10**. **Figure 55.9** compares the values of R_s for a pile group in a soil layer that has constant stiffness with depth ($\rho = 1$) against a group in a soil layer that has stiffness increasing linearly with depth ($\rho = 0.5$). In both cases the soil layer is infinitely deep. It can be seen that the degree of interaction (and the value of R_s) is much less when the stiffness increases linearly with depth (which occurs commonly in practice, in many geologies, due to the combined effects of increasing effective stress and reductions in weathering with depth). **Figure 55.10**(a) plots a correction factor, F_h, against the ratio of compressible layer thickness, h, to the pile length, L. F_h is defined by

$$F_h = \frac{R_s \text{ for finite layer of depth } h}{R_s \text{ for infinitely deep layer}} \quad (55.8)$$

This effect becomes more significant for larger pile groups, with F_h values of between 0.6 and 0.8 for a wide range of scenarios. The correction factor will increase as pile spacing increases (Poulos and Davies, 1980).

Figure 55.10(b) shows the potential difference in R_S value (indicated by the correction factor, F_b) for friction piles 'floating' in an infinitely deep homogeneous layer compared with end-bearing piles toed into a layer of varying stiffness, relative to the overlying strata (E_b/E_S). From **Figure 55.10(b)** it can be seen that R_s decreases (i.e. the pile-to-pile interaction decreases, and, similarly, the axial load distributions will become more uniform) as the relative stiffness of the bearing layer increases. The effect is most significant for short stiff piles. However, for slender piles (e.g. $L/d = 100$) the bearing layer has little effect, since little load is likely to reach the pile toe under normal working load conditions.

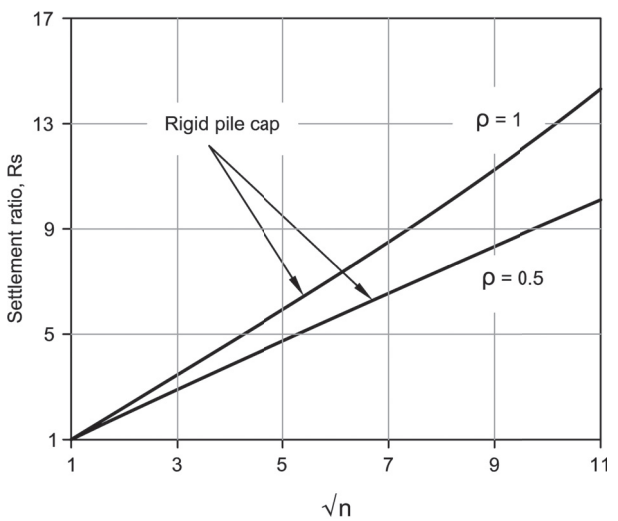
In practice, piles are often installed in geological sequences that exhibit marked variations in stiffness with depth, or 'layering'. The layers beneath the pile are either stiffer or softer than

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Factor	Vertical loading		Horizontal loading	
	Group settlement ratio	Non-uniformity of axial loads	Group deformation	Stresses in piles
Increase of ground stiffness with depth	–	–	– (if < L_c)	– (if < L_c)
Ground layering, relatively stiff layer at depth below pile toe	– –	– –	NE	NE
End- bearing pile (compared with friction pile)	–	–	NE	NE
Ground layering, relatively compressible layer at depth below pile toe	+++	+++	+	+
Ground, nonlinear stress–strain behaviour	–(1)	–(1)	++	++
Pile installation	– –	– –	+	+
Pile-group layout	NE	NE	+	+
Pile spacing (s > 3d)	–	–	–	–
(s < 3d)	+	+	+	+
Increasing displacement	–	–	++	++
Near-surface soils (<6d) (relatively weak layer)	NE	NE	+++	+++
Pile-cap stiffness reduces	NE	– –	NA	NA
Pile head fixity, fixed to free head condition	NA	NA	+++	Depends on pile location in group

Key
– Leads to a reduction in interaction effects, e.g. smaller group settlement, more uniform axial loads.
+ Leads to an increase in interaction effects, e.g. larger group deformation, more non-uniform axial loads and stresses in piles.
– – or ++ Indicates major effects, etc.
NE = negligible effect
NA = not applicable
L_c is the critical pile length for horizontal loading.
(1) Particularly important for large pile groups.

Table 55.1 Pile interaction, influential factors



NB.
ρ = 1, Young's (or shear) Modulus is constant with depth; ρ = 0.5, Young's (or shear) Modulus increases linearly with depth; n = number of piles in group; Rs = W/Ws, W = pile group settlement, Ws = single pile settlement (under same average load)

Figure 55.9 Influence of soil stiffness profile on pile-group settlement
Reproduced from Fleming *et al.* (2009)

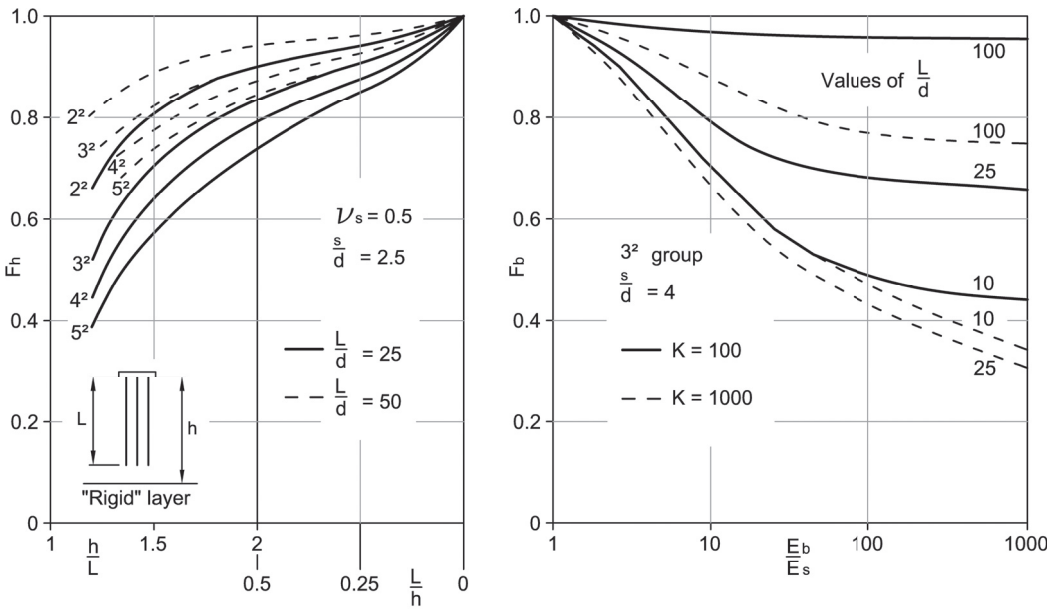
the layer at the pile-toe level. Although this is a common situation, computer software is often not able to accurately model the influence of these layers on pile-group behaviour. **Figure 55.11** illustrates two different situations. Firstly, **Figure 55.11(a)** shows the increase or decrease in pile-group settlement (for pile groups increasing in size up to 121 piles, pile spacing of 3D, and pile length of 25 m) due to underlying layers of lower or greater stiffness respectively. **Figure 55.11(b)** shows the increase in settlement due to an underlying layer of lower stiffness (for piles groups with up to 64 piles, pile spacing of 4D and pile length of 15 m). The following can be observed from **Figure 55.11**:

(i) the effect of the underlying layer is more significant for larger pile groups;

(ii) for large pile groups, settlement may be increased by a factor of three or more, when underlain by a more compressible layer;

(iii) if underlain by a relatively stiff layer, pile-group settlement may be more than halved.

Therefore, the potential exists either to seriously overestimate or underestimate pile-group settlement, if the presence of underlying layers are either ignored or not detected. Axial



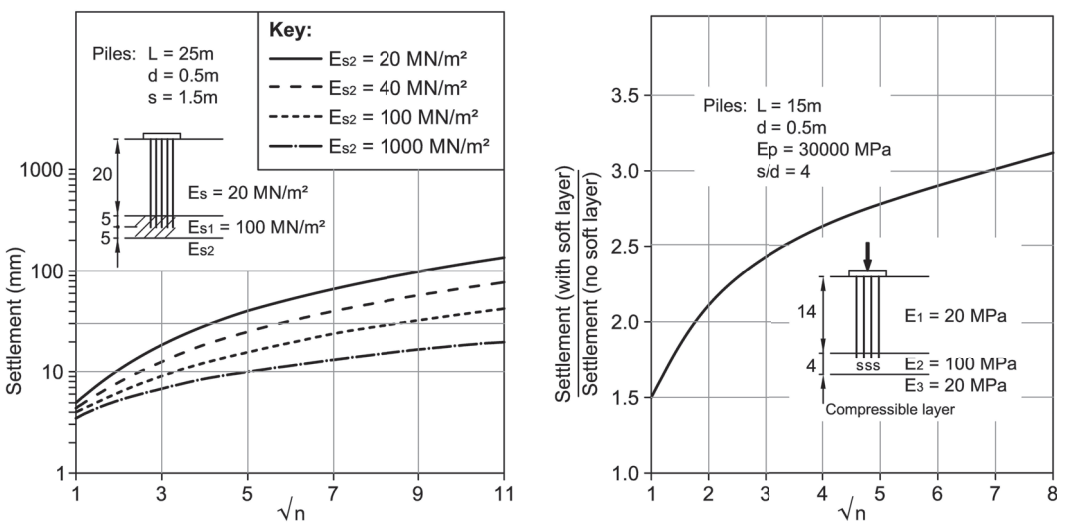
NB.
h = thickness of soil layer, L = pile length, d = pile diameter, s = pile spacing, 3² = 3x3 pile group

(a) Reduction coefficient F_h for effect of finite layer thickness, compared with infinite layer

NB.
K = relative soil/pile stiffness, K = (Ep RA)/Es, Ep = pile Young's Modulus, RA = area ratio (=1.0 for solid pile) = Ap/(πd²)/4; Ap = area of pile wall thickness

(b) Reduction coefficient Fb for effect of bearing stratum Young's Modulus, Eb, to overlying soil modulus, Es

Figure 55.10 Influence of finite layer thickness and bearing stratum stiffness on pile-group settlement
Reproduced from Poulos and Davis (1980)



NB.
n = number of piles in group

(a) Effect of underlying compressible layer Young's Modulus, E_{s2}, on group settlement

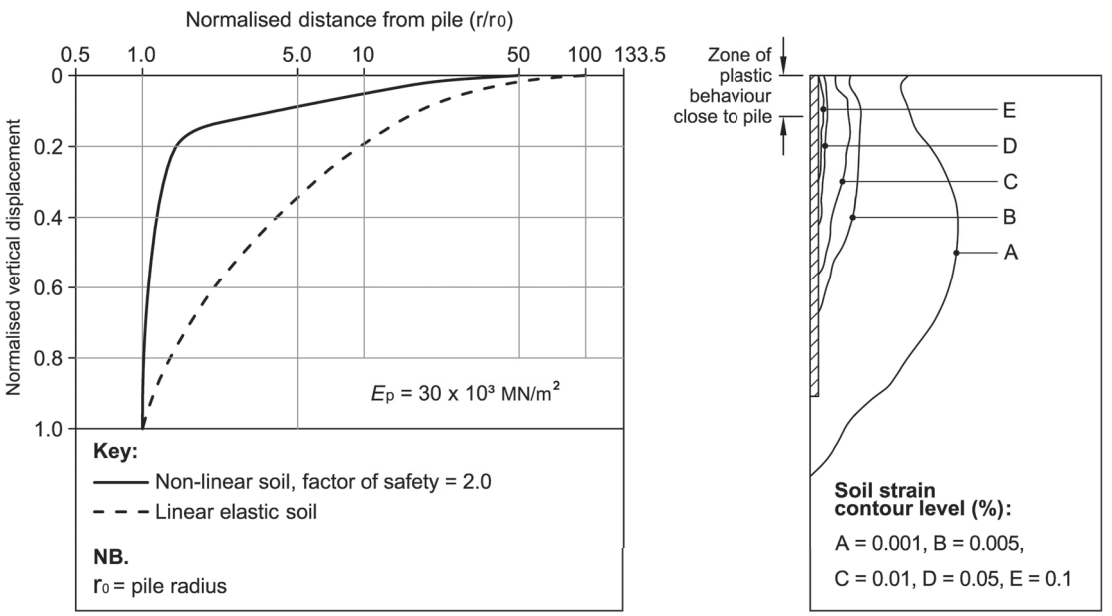
(b) Effect of underlying compressible layer on group settlement

Figure 55.11 Influence of soil layering on pile-group settlement
Reproduced from (a) Poulos (2005), with permission of ASCE; (b) Poulos *et al.* (2001)

load distributions across a pile group will be similarly affected: more compressible underlying layers will lead to more non-uniform axial loads across the group (higher axial loads in piles around the pile-group perimeter and lower loads in central piles) whereas stiffer underlying layers will lead to more uniform axial loads across the group.

Pile-group analyses often assume that the soil behaviour is linear elastic and that pile installation has no effect on soil properties. Both these assumptions lead to an overestimation of pile-to-pile interaction effects. **Figure 55.12** illustrates the influence of nonlinear elastic versus linear elastic assumptions for soil behaviour; **Figure 55.12(a)** indicates that modelling the soil behaviour as linear elastic will tend to overpredict the soil settlement adjacent to a pile. **Figure 55.12(b)** plots contours of soil strain around a pile (derived from a sophisticated nonlinear soil model), which indicates that most of the soil adjacent to the pile experiences strains of less than 0.01%, whereas close to the top of this relatively compressible pile there is a zone of soil that has become plastic. Hence, the mobilised soil stiffness close to the pile will usually be lower than that mobilised further away from the pile, at any given depth below the ground surface. This effect will be exacerbated by the influence of pile installation. For either bored piles or driven piles, there will be a zone of soil immediately adjacent to the pile that will be intensely disturbed and remoulded. **Figure 55.13** compares predictions from a range of different software with measured pile-group settlements (Poulos, 1989). **Figure 55.13(a)** indicates that although some software could accurately predict single-pile settlement, the group settlement was overpredicted, i.e. the interaction effects, or equivalent R_s value, were overpredicted. **Figure 55.13(b)** plots R_s versus the number of piles in the group. This indicates that the conventional linear elastic analysis overpredicts R_s and this overprediction becomes worse as the pile-group size increases. Relatively good predictions are made for a modified analysis in which the soil between the piles was assumed to be stiffer than that immediately adjacent to the piles. For the site in Texas, USA, used for **Figure 55.13(b)** (O'Neil *et al*., 1982), which was underlain by stiff overconsolidated low- to medium-plasticity clays, the modified analysis assumed a Young's modulus of $750S_U$ for the remoulded soil adjacent to the driven steel tubular piles, and a Young's modulus of about $2500S_U$ for the soil beyond the remoulded zone (this was equivalent to the modulus at very small strain derived from cross-hole geophysics data).

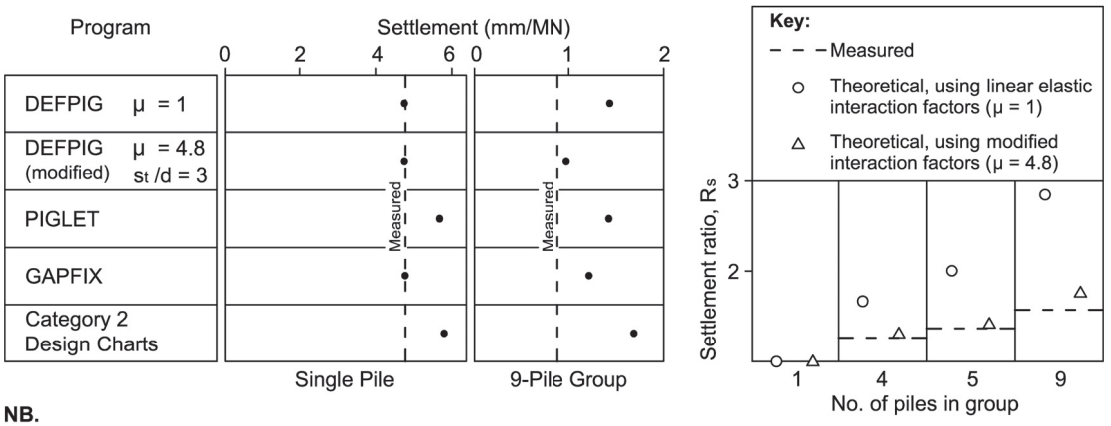
As noted above, pile-to-pile interaction leads to a redistribution of axial loads across a pile group. It is affected by the same factors as those which affect group settlement, with greater interaction causing higher axial loads in piles around the group perimeter and lower axial loads for central piles. End-bearing piles toed into a relatively stiff layer will have a more uniform distribution of axial loads than friction piles 'floating' in a deep layer with constant stiffness (**Figure 55.14**). Measurements of axial loads in pile groups are rare; however, Mandolini *et al*. (2005) have collated available data



(a) Profiles of surface settlement adjacent to a pile 30m long

(b) Contours of deviatoric strain around a pile 30m long when factor of safety = 2.0

Figure 55.12 Influence of soil nonlinearity on pile-to-pile interaction
Reproduced from Jardine *et al*. (1986)



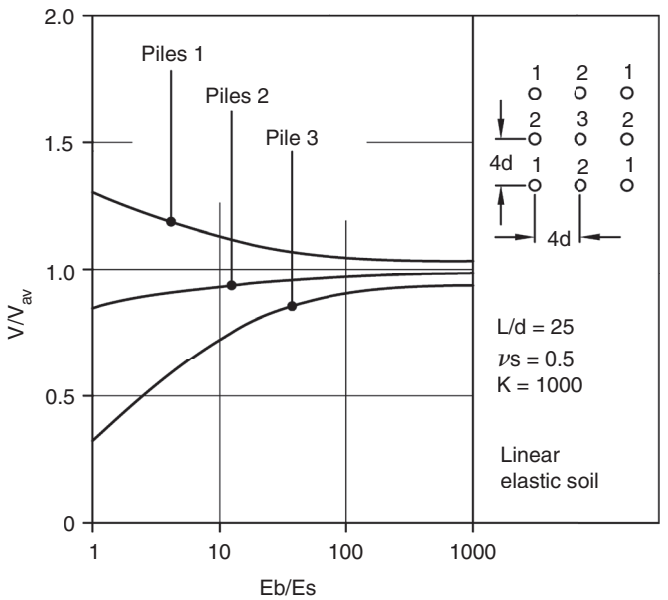
NB.

μ = Esm/Esr; Esm = Young's Modulus of soil beyond St; Esr = Young's Modulus of remoulded soil adjacent to pile
St = thickness of remoulded zone, Es varies linearly across St between Esr and Esm; d = pile diameter

(a) Effect of analysis method on single pile and group settlements

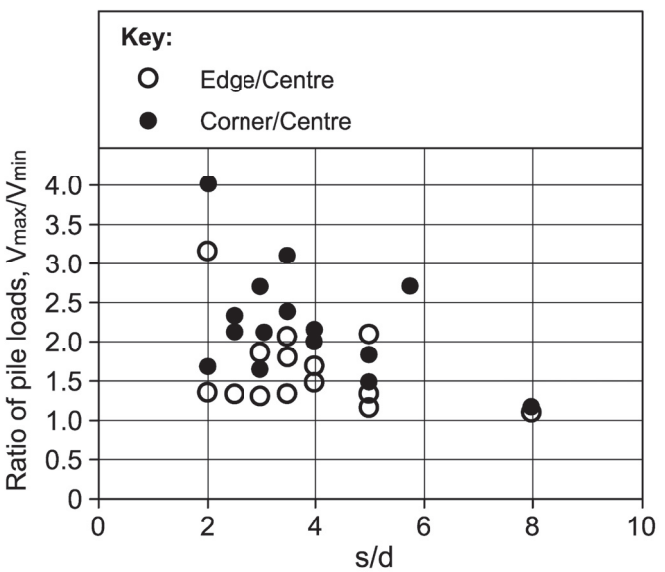
(b) Theoretical and measured group settlement behaviour (tests of O'Neill *et al*. 1982)

Figure 55.13 Influence of analysis method on single-pile and pile-group settlement
Reproduced from Poulos (1989)



NB.
Refer to Figure 55. 10 for definition of K, Eb, Es,
3 x 3 pile group

Figure 55.14 Influence of bearing stratum stiffness on axial load distribution
Reproduced from Poulos and Davis (1980)



NB.
Vmax = maximum axial load in a pile,
Vmin = minimum axial load in a pile,
measured axial loads, 22 case histories.
Wide range of pile types, geology, etc.
S = pile spacing, d = pile diameter

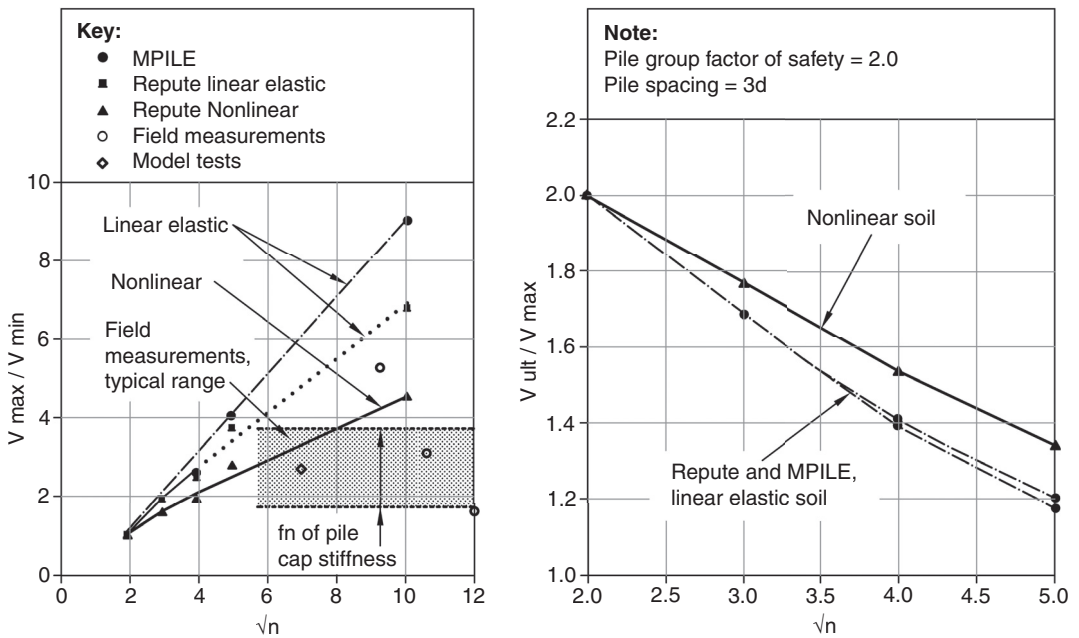
Figure 55.15 Measured axial load distributions, pile groups
Reproduced from Mandolini *et al.* (2005)

and this is plotted in **Figure 55.15**. The figure indicates that typically the ratio of corner to centre pile axial loads is between about 2.0 and 2.5 (and the full range is between about 1.5 and 3) for common pile spacings of about 2.5 to 4 times the pile diameter. It is apparent that pile axial loads become more uniform with increasing pile spacing, and that interaction effects appear to be negligible for pile spacings of more than 8 times the pile diameter. **Table 55.2** gives measured axial load ratios (V_{max}/V_{min}) for a few specific case histories. At Stonebridge Park, given the large pile-group aspect ratio, a much larger

Pile group	Ground profile	Pile type	Pile length (m)	Pile diameter (m)	Number of piles	Spacing (m)	Pile-group aspect ratio	Axial load ratios		Pile-cap stiffness
								C/I	E/I	
Stonebridge Park (Cooke *et al.*, 1981)	Deep layer of overconsolidated clay (London Clay)	Bored (friction)	13	0.45	351	1.61	6.6	2.2	1.7	Flexible
Cargliano Bridge (Mandolini *et al.* 2005)	Interbedded soft clays and silts over dense sand and gravel	Driven tubular (end bearing)	48	0.38	144	1.14	1.1	1.6	1.3	Rigid
Pile-group test (Koizumi and Ito, 1967)	Deep layer of overconsolidated clay	Driven closed-end (friction)	5.5	0.3	9	0.9	1.2	2.6	1.9	Rigid

Note :
C is the corner pile axial load, ***I*** is the internal pile axial load and ***E*** is the edge pile axial load.

Table 55.2 Examples of measured axial load distributions



NB.
V max = maximum axial load in a pile
V min = minimum axial load in a pile
n = number of piles in group

(a) Axial load ratio (Vmax/Vmin). Linear and non-linear soil models and field measurements

NB.
V ult = ultimate geotechnical capacity of a single pile
V max = calculated maximum axial load in a pile in the group

(b) Perimeter pile factor of safety (Vult/Vmax); linear and non-linear soil models, for overall pile group factor of Safety = 2.0

Figure 55.16 Axial load distribution for linear and nonlinear soil models versus pile-group size
Reproduced from O'Brien and Brown (2010)

value of V_{max}/V_{min} would have been expected. In this case, the pile cap was relatively flexible, which led to more uniform pile axial loads. For the Cargliano Bridge, the end-bearing piles led to a relatively low value of V_{max}/V_{min}. The pile-group test reported by Koizumi and Ito, with friction piles in a deep compressible layer and a rigid pile cap, led to relatively high values of V_{max}/V_{min}.

Figure 55.16(a) summarises the results of a study by O'Brien and Bown (2010), which plots the ratio of corner to centre pile axial loads plotted against pile-group size. V_{max} is the maximum predicted axial load, which for square pile groups occurs

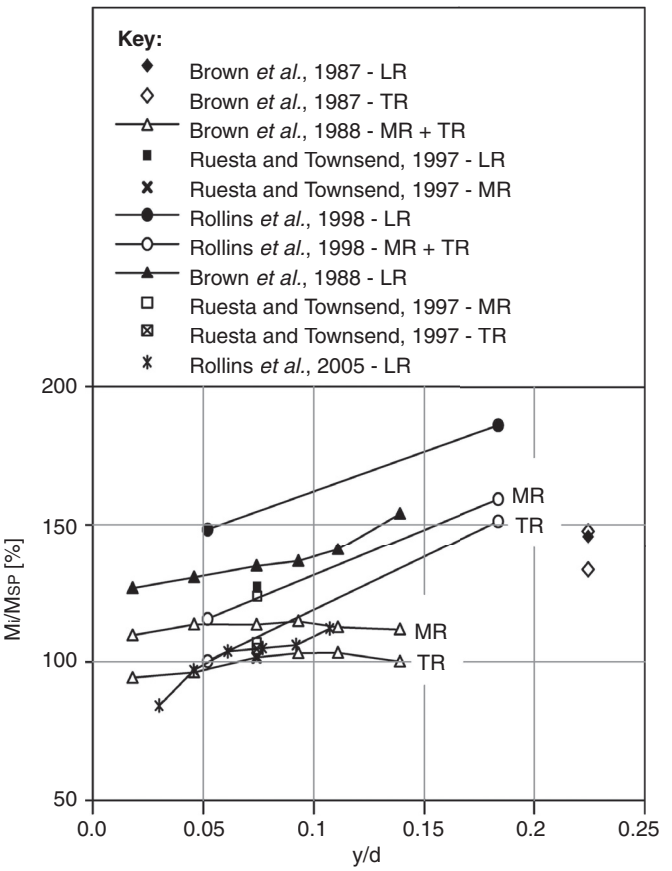
in the corner piles. V_{min} is the minimum predicted axial load, which occurs in the piles close to the centre of the pile group. This shows that linear elastic analyses overestimate the load redistribution across the group when compared with nonlinear analyses and case history data, and that this discrepancy increases with increasing pile-group size.

For large groups, if pile axial loads are predicted by linear elastic methods then it is inevitable that corner piles will have an apparently low geotechnical 'factor of safety' even when the overall group factor of safety is two or more (**Figure 55.16(b)**). This can be a problematic issue when considering how to apply code requirements. This is discussed in section 55.10.

55.4 Pile-to-pile interaction: horizontal loading

For piles subject to a horizontal load the main design issue is usually the stresses that develop in the piles relative to their structural strength.

Mandolini *et al*. (2005) summarise several studies on pile-group behaviour when horizontal loads are applied. Pile-to-pile interaction under a vertical load is fully developed at relatively small displacements, and nonlinearity at larger displacements is focussed in the thin layer of soil around the pile–soil interface (and does not amplify group interaction effects at larger displacements). In contrast, under horizontal loading, interaction between piles grows as the pile-group displacement increases. **Figure 55.17** provides some measured data from instrumented pile groups and centrifuge tests (Mandolini and Viggiani, 2005) for free head conditions. The ratio between the maximum bending moment in different piles in a group and in a single isolated pile is plotted against the pile-group displacement (the ratio is derived for the same average horizontal load per pile). The moments in the group piles are generally larger than the single pile, and the ratio increases with increasing displacement. Larger bending moments develop in the leading row (LR) piles compared to either the middle (MR) or trailing row (TR) piles (**Figure 55.17**); this phenomenon has been attributed to 'shielding' or 'shadowing' effects between adjacent piles in the same row (refer to **Figure 55.7**). This means that for horizontal loading, different group configurations, e.g. square vs rectangular vs circular, etc., will exhibit different group interaction effects even if the number of piles and average spacing is the same (this contrasts with group interaction effects under a vertical load, which are practically unaffected by the group configuration for a given number of piles and similar average spacing). Hence, for piles under horizontal loading it is potentially unsafe to assess the bending moments in piles within a group from an assessment of single-pile behaviour under horizontal loading; in addition the assumption of linear elastic behaviour can also be unsafe. The use of *p–y* curves is well established for predicting the behaviour of single piles under horizontal loads. However, the use of *p–y* multipliers to account for group interaction effects is of more doubtful validity because of the large number of complicating factors that can affect real pile-group behaviour under horizontal loading.



NB.
Mi = maximum bending moment in pile in a group; Msp = maximum bending moment in a single isolated pile; LR = piles in a leading row MR = piles in middle of a row; TR = trailing piles

Figure 55.17 Ratio of bending moments for piles in a group and single isolated pile vs displacement
Reproduced from Mandolini *et al.* (2005)

55.5 Simplified methods of analysis

55.5.1 General comments

There are four simplified methods that may be used to assess pile-group settlement:

(i) pile-group settlement ratio;

(ii) elastic interaction factors;

(iii) equivalent pier;

(iv) equivalent raft.

Each method has distinctive advantages and disadvantages (which are summarised in **Table 55.3**). When applied appropriately, the methods are helpful supplementary tools to

Simplified method	Advantages	Disadvantages
Empirical settlement ratio	Very quick and simple. Best suited for friction piles, in deposits with increasing strength and stiffness with depth	The influence of specific geological features, pile or ground properties cannot be assessed
	Useful as a 'sense' check of more complex methods	Potentially unsafe if weak or compressible strata underlies the bearing stratum. Potentially overconservative, if a relatively stiff layer underlies the bearing stratum
Elastic interaction factors	Influence of varying pile length, diameter and spacing can be quickly checked	Cannot directly check influence of underlying strong or weak layers (refer to **Figures 55.10** and **55.11**)
	Best suited for friction piles in deposits with increasing strength and stiffness with depth	Care needed in amplifying single-pile settlement for large groups, use initial tangent pile stiffness, rather than secant; otherwise can be overconservative
Equivalent pier	Well suited for pile groups with relatively small aspect ratio, $R < 3.0$	Not appropriate for large pile-group aspect ratios, $R > 3.0$
	If using elastic solutions for single pile, then is quick to use	Inappropriate if pile lengths in group vary significantly
	Flexible, method can also be used within sophisticated numerical models, in axisymmetric mode	
Equivalent raft	Most appropriate simplified method for checking influence of strong or weak layers at depth	Overconservative for pile groups with a small aspect ratio, $R < 3.0$
		Significant judgement needed to assess appropriate raft level and dimensions

Table 55.3 Pile-group settlement: Advantages and disadvantages of simplified methods

computer-based methods (which may also have serious limitations in certain situations) and can facilitate a quick check of more sophisticated methods (which can be more prone to error). It is probably fair to say that methods (i) to (iii) above are under-used, whereas method (iv) is over-used.

There are no simplified methods for checking axial load distributions across a group, but **Figures 55.15** and **55.16** indicate ranges of V_{max}/V_{min} that could be reasonably anticipated, and **Figure 55.14** and **Tables 55.1** and **55.2** indicate the factors that may influence axial load distribution.

It should be noted that the use of structural analysis software (which typically simulate pile behaviour as independent elastic springs) is not recommended, since pile-to-pile interaction is not modelled. Poulos and Davis (1980) give examples of some of the errors that can be produced.

55.5.2 Pile-group settlement ratio, R_{Se} (empirical correlations)

Mandolini *et al.* (2005) proposed an empirical correlation between the pile-group settlement ratio R_{Se} and the pile-group aspect ratio, based on an analysis of 63 case histories (for pile groups in varying geologies, and with varying pile length, type, diameter, etc.), which is shown in **Figure 55.18**. Equations have been developed for the upper-bound, best-estimate and lower-bound settlements:

$$\text{Best estimate, } R_{Se} = \frac{a}{(R)^b} = \frac{0.29}{(R)^{1.35}}(n) \qquad (55.9)$$

where n is the number of piles in the group, the pile-group aspect ratio $R = (ns/L)^{0.5}$ and R_{Se} is the empirical pile-group settlement ratio.

$$\text{Pile-group settlement, } W = R_{Se} W_S \qquad (55.10)$$

where W_s is the single-pile settlement under the average working load of piles within the group (Q/n) and Q is the total vertical load applied to the pile group.

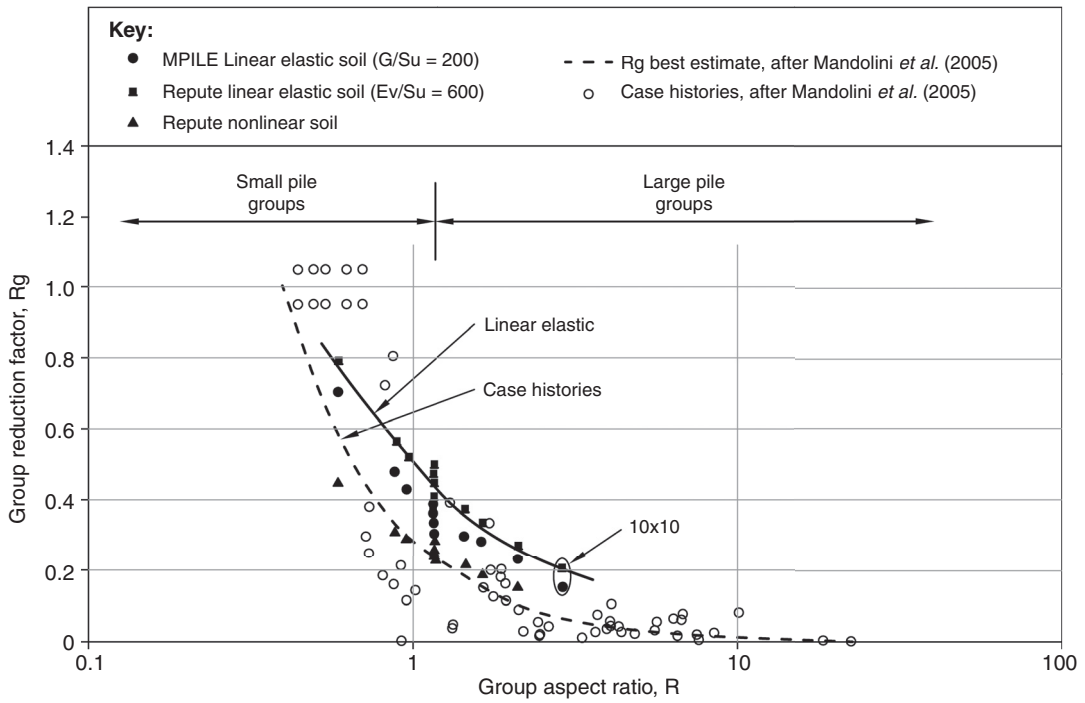
The single-pile settlement W can be calculated by using either relevant pile test data or Fleming's method for single-pile settlement (Fleming, 1992). The latter has been found to be reliable for a wide range of pile types and ground conditions (refer to Chapter 54 *Single piles*). The upper- and lower-bound values of R_{Se} are:

$$\text{Upper bound } R_{Se} = \frac{0.5}{R}\left[1 + \frac{1}{3R}\right]n \qquad (55.11)$$

$$\text{Lower bound } R_{Se} = \frac{0.17}{(R)^{1.35}}(n) \qquad (55.12)$$

Hence, application of the above equations enables a rapid estimate to be made of the likely range of the pile-group settlement.

Experience indicates that (55.9) is generally appropriate for friction piles with soils with strength and stiffness increasing gradually with depth; (55.11) applies if relatively deep and uniform soils underlie the pile group and (55.12) applies if



NB.
Settlement ratio, Rs = nRg, n = number of piles in group, Rg = group reduction factor. Group aspect ratio, R = (ns/L)0.5, s = pile spacing, L = pile length. G = soil shear modulus, Ev = soil Young's Modulus, Su = undrained shear strength

Figure 55.18 Group reduction factor *Rg* versus Group aspect ratio *R*. Field measurements and parametric study
Data taken from Mandolini *et al*. (2005); O'Brien and Bown (2010)

the soil or rock beneath the group exhibits a rapid increase in strength and stiffness.

55.5.3 Elastic interaction factors (design charts)

Elastic interaction factors have been widely published (for example Poulos and Davies, 1980). However, many of the early solutions relied on unrealistic assumptions (e.g. an infinite elastic medium with constant Young's modulus, rigid pile, etc.) and overestimated the interaction between the piles and, therefore, the pile-group settlements (and the non-uniformity of axial loads) were overpredicted. Pile installation effects and soil nonlinearity lead to reduced interaction between piles. Randolph (1994) has outlined a simple and practical approach to improve the reliability of elastic interaction factors, which is shown in **Figure 55.19**. The initial stiffness derived from the single-pile load-settlement response is given by the tangent line OA (and will be a function of the small strain stiffness of the soil). Plastic strain, which will develop at higher load levels, leads to the offset AB. This plastic strain will develop locally adjacent to the pile shaft. Group interaction effects under vertical load are predominantly due to elastic, rather than plastic, strains. Therefore, the pile-group settlement ratio (derived from linear elastic theory) R_{St} is only applied to the elastic component of the single-pile settlement. Hence, the elastic pile-group settlement is shown as OC in **Figure 55.19** where OC = R_{St} OA and the total pile-group settlement OD = OC + CD (with CD = AB). For most conventionally designed pile groups, which will have overall factors of safety well in excess of 2.0, the plastic component of settlement will be relatively minor. Back analysis of a wide range of case histories by Mandolini and Viggiani (1997) confirms that Randolph's approach is usually more appropriate than applying the group settlement ratio R_{St} to the single-pile secant stiffness at the average working load of the piles within the group.

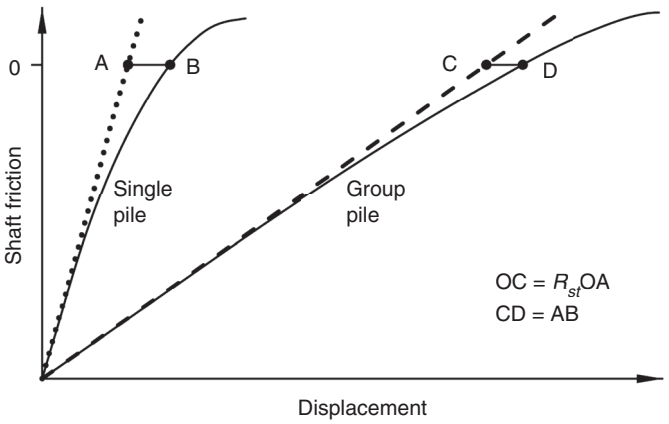
To a reasonable approximation, the group settlement ratio is (Fleming *et al*., 2009)

$$R_{St} = n^{e} \tag{55.13}$$

and

$$W = R_{St}\, W_{Se} + W_{Sp} \tag{55.14}$$

where W is the pile-group settlement, R_{st} is the group settlement ratio derived from linear elastic theory, W_{Se} is the elastic settlement under the average working load of piles within the group (Q/n), n is the number of piles in the group, Q is the total vertical load applied to the group and W_{Sp} is the plastic settlement of a pile at Q/n.



NB.
R_{st}, group settlement ratio (from elastic theory) = We/Wse. We = elastic pile group settlement, Wse = single pile elastic settlement (under same average load)

Figure 55.19 Applying elastic theory to pile interaction and nonlinear load-settlement behaviour for vertical loading
Reproduced from Randolph (1994)

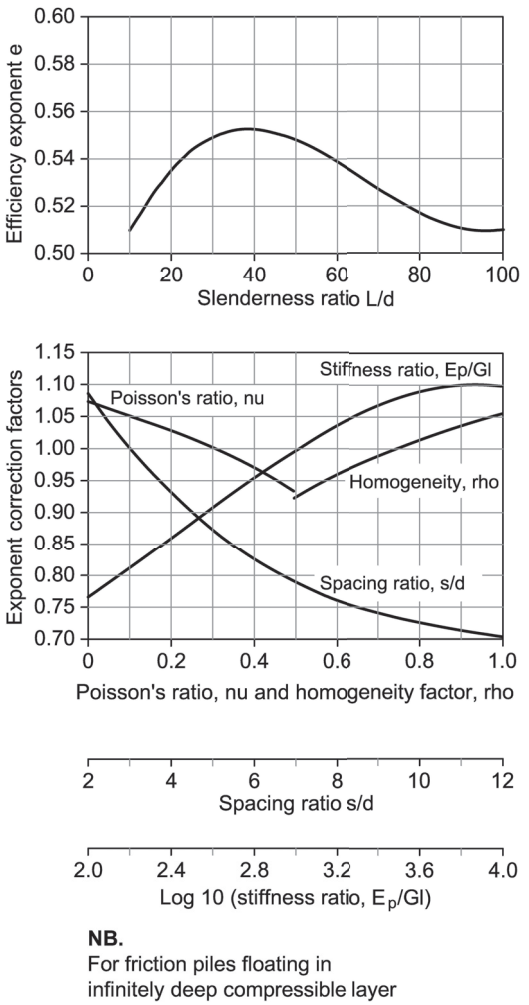
For piles working mainly in shaft friction, e is typically between 0.3 and 0.5. Based on Randolph (1994), **Figure 55.20** gives a set of design charts; the exponent e is given by a base value (depending on the pile slenderness ratio L/d) e_1 and four correction factors, c_1, to c_4:

$$e = e_1\, c_1\, c_2\, c_3\, c_4 \tag{55.15}$$

where c_1 is a function of the pile stiffness to the ground stiffness, $E_p\,/G_L$ and E_p is Young's modulus for the pile. G_L is the ground shear modulus at the pile-toe level, c_2 is a function of the pile spacing s to the pile diameter d, c_3 is a function of the rate of increase of the ground shear modulus with depth, rho = G_{av} / G_L, G_{av} is the ground shear modulus halfway down the pile and c_4 is a function of Poisson's ratio, ν'. The value of e_1 is based on $E_p\,/G_L = 1000$, $s\,/\,d = 3$, $\rho = 0.75$ and $\nu' = 0.3$. These charts assume an infinitely deep elastic layer. Reference to **Figures 55.10** and **55.11**, allows judgements to be made about the influence of a finite layer thickness and stiffer or weaker layers below the pile toe. Poulos (1989) indicates that for friction piles at typical working loads (factor of safety in excess of 2.0), plastic deformation would be expected to only increase single-pile settlement by between 10% and 25% compared with the elastic response derived from the initial pile stiffness.

55.5.4 Equivalent pier and equivalent raft

Equivalent pier: the equivalent pier approach replaces the pile group by a buried 'cylinder' of appropriate dimensions and stiffness (**Figure 55.21**). Solutions for calculating the single-pile settlement can then be used to calculate the settlement of the equivalent pier (provided due account is taken of the compressibility of the cylinder, the stiffness of the soils below the base and the shear stiffness of the interface around the sides of



NB.
For friction piles floating in infinitely deep compressible layer

Figure 55.20 Design charts (linear elastic soil) for pile-group settlement ratio, R_{st}
Reproduced from Fleming *et al.* (2009)

the cylinder). Alternatively the settlement of the equivalent pier can be simulated within an axisymmetric numerical model.

For a pile-group of area A_g the diameter of the equivalent pier d_{eq} is

$$d_{eq} = \left(\frac{4}{\pi} A_g\right)^{0.5} = 1.13\,(A_g)^{0.5}. \tag{55.16}$$

The equivalent Young's modulus E_{eq} of the pier is

$$E_{eq} = E_S + (E_p - E_S)\,(A_p\,/A_g) \tag{55.17}$$

where E_P is Young's modulus of the pile, E_S is the average Young's modulus of the soil penetrated by the piles, A_P is the total cross-section area of the piles in the group and A_g is the total plan area encompassed by the pile group.

The elastic solutions for settlement of a single pile presented by Randolph (1994), which allow for the relatively low aspect ratio of equivalent piers (for typical pile-group geometries) can

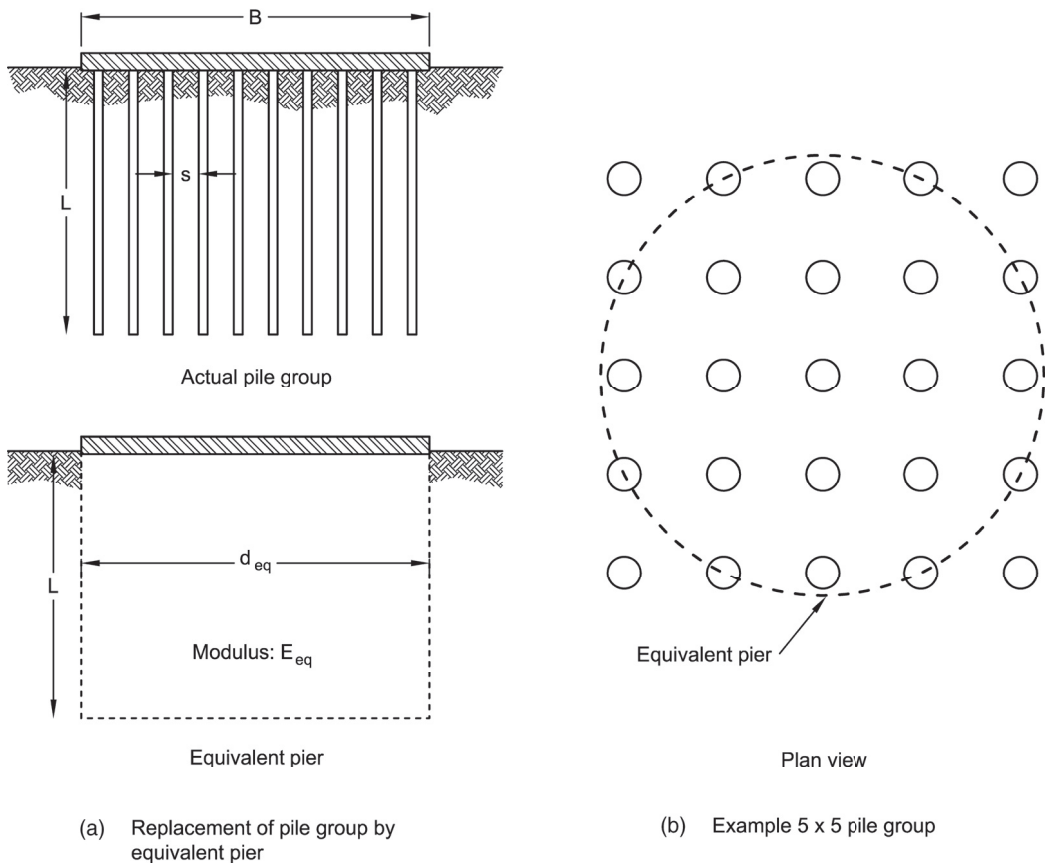
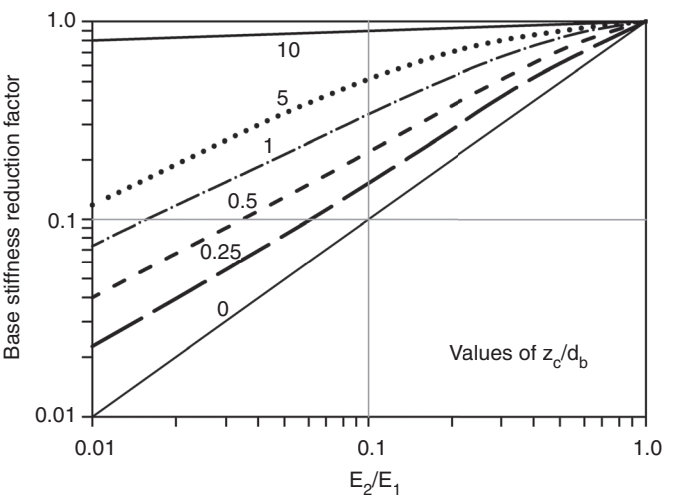


Figure 55.21 Replacement of pile group by an equivalent pier

be conveniently programmed in modern spreadsheets to facilitate rapid analyses of pile-group settlement. Alternatively, the nonlinear approach proposed by Fleming (1992) can be used (refer to Chapter 54 *Single piles*), although it should be noted that the strength and stiffness properties for the base and shaft resistance around the equivalent pier should mainly be representative of the undisturbed soil mass, rather than the disturbed remoulded soil immediately adjacent to an individual pile.

The influence of an underlying compressible layer on the load-settlement behaviour of the equivalent pier can be assessed (based on studies by Poulos, 2005) from **Figure 55.22**. **Figure 55.22** provides a plot of the base stiffness reduction factor as a function of Z_c/d_b and E_2 and E_1 (where E_2 and E_1 are the Young's moduli of the upper and lower layers, respectively), refer to **Figure 55.6(c)**. The base stiffness reduction factor is defined as the ratio of the base stiffness with the underlying layer E_2 to the base stiffness of E_1 only (i.e. with no underlying layer present).

Equivalent raft: the equivalent raft method has been used extensively for estimating pile-group settlement and is described in many textbooks. The pile group is replaced by a raft foundation acting at a representative depth below the surface with some equivalent dimensions. Many variants are available; however, the approach by Tomlinson (1987) is relatively simple and widely used. It is shown in **Figure 55.23**. As



NB.
For definition of z_c, d_b, refer to **Figure 55.6(c)**

Figure 55.22 Base stiffness reduction factor for pile group (equivalent pier) underlain by compressible layers
Reproduced from Poulos (2005) with permission of ASCE

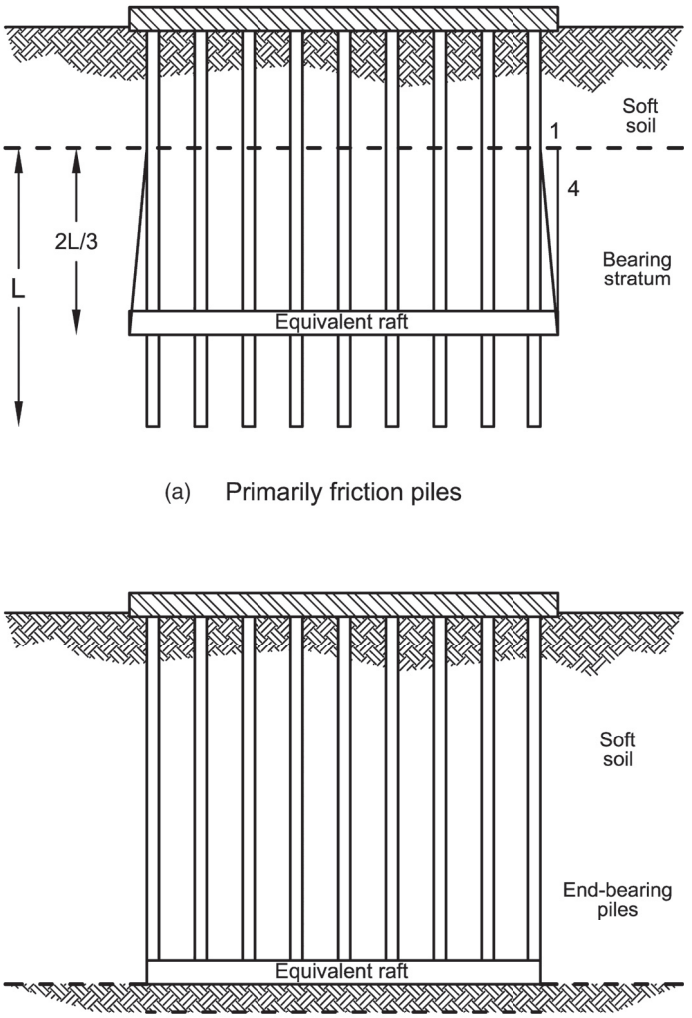


Figure 55.23 Replacement of pile group by an equivalent raft

noted by Poulos *et al*. (2001), this method relies on considerable engineering judgement to assess the representative depth and appropriate raft dimensions, which depend on an understanding of the relevant load transfer mechanism for the pile group under consideration. Once the equivalent raft is established then the settlement can be computed using the analysis methods used for shallow foundations.

The average pile-group settlement W_{av} is then

$$W_{av} = W_{\text{raft}} + \Delta W \tag{55.18}$$

where ΔW is the elastic compression of the piles above the equivalent raft level (acting as free standing columns). Assuming elastic soil behaviour:

$$W_{\text{raft}} = F_{av}\ F_D\ q \sum_{i=1}^{n} \left(\frac{I_E}{E_S} \right) h_i \tag{55.19}$$

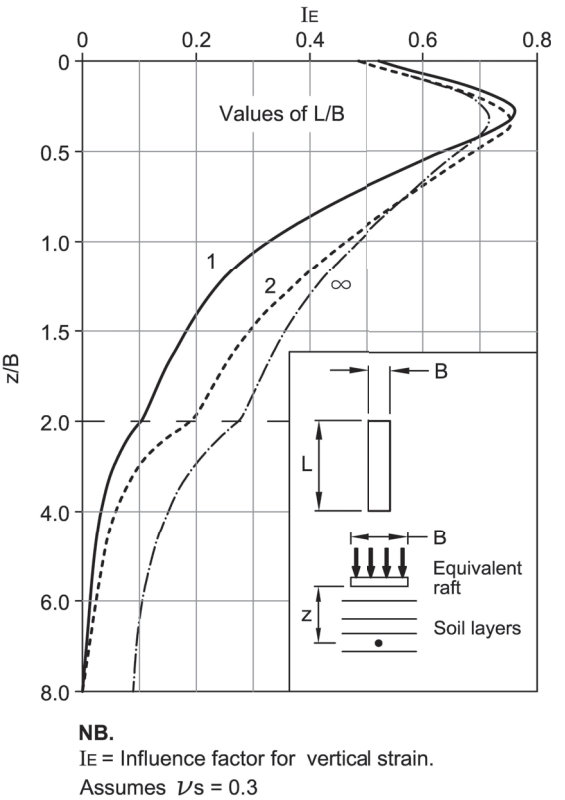
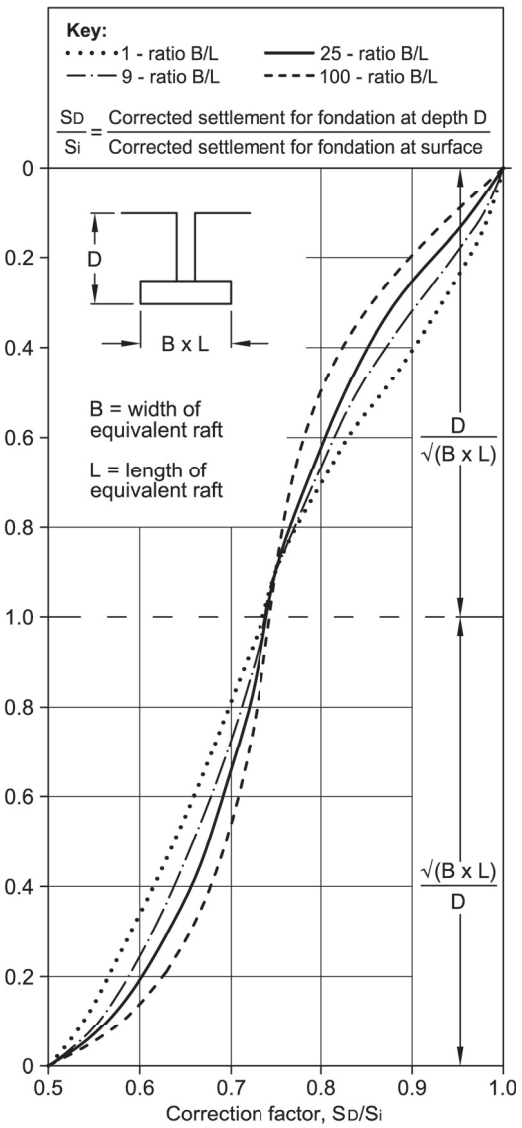


Figure 55.24 Equivalent raft settlement using elastic theory. Strain influence factor versus depth
Data taken from Randolph (1994)

where Wraft is the equivalent raft settlement, q is the average pressure applied to the raft, I_E is an influence factor (refer to **Figure 55.24**) for the ith layer, h_i and E_S are the layer thickness and secant Young's modulus, respectively, for the ith layer, F_D is a depth correction factor (as derived by Fox, refer to **Figure 55.25**) and F_{av} is equal to 0.8 (to correct from centreline settlement to average raft settlement). The embedment depth of the raft should be assumed to be the depth below the competent bearing stratum, rather than the ground surface (**Figure 55.23**).

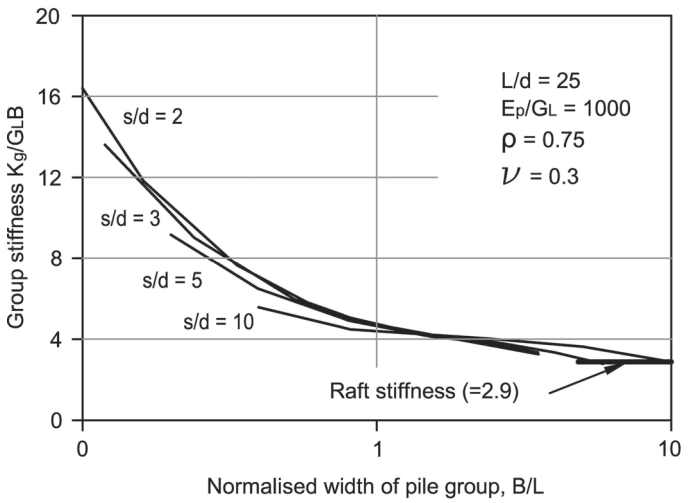
It should be noted that care is required both in method and input parameter selection; for example analytical methods and parameters that are purely empirically derived from observations of shallow foundation settlement are unlikely to be appropriate. In general, the use of elastic methods, such as (55.19) (e.g. Poulos, 1993), are likely to be suitable for most situations. For overconsolidated clays, if a drained Young's modulus is used, then total settlement (i.e. undrained plus consolidation settlement) will be calculated. Elastic analyses will *not* be appropriate if a layer of normally or lightly overconsolidated clay is present at depth below the pile-toe level, *and* the pile-group bearing pressures below the base cause the preconsolidation pressure (or yield stress) to be exceeded.

The main advantage of the equivalent raft method is that it enables the influence of softer or stiffer soil layers below the



NB.
D = depth below bearing stratum, ignore overlying soft layers

Figure 55.25 Equivalent raft settlement using elastic theory. Fox's depth correction factor
Data taken from Randolph (1994)



NB.
G_L = shear modulus of soil at pile toe, B = width of pile group, K_g = pile group stiffness = (total load/average settlement), ρ = increase of shear modulus with depth (= G_{AV}/G_L), G_{AV} = shear modulus of soil halfway down pile

Figure 55.26 Normalised stiffness of pile groups and equivalent raft versus normalised width
Reproduced from Randolph (2003)

toe level of a pile group to be assessed in a simple and transparent way. This is particularly important in situations where softer soil layers may exist at some level below the pile group; as noted above, some commercially available software cannot analyse the situation reliably and a separate check is essential.

55.5.5 Which is appropriate, equivalent raft or equivalent pier?

When deciding between an equivalent pier or raft it is important to carefully consider the load transfer mechanism. If the piles are mainly dependent on shaft friction and the pile group is relatively deep and narrow, the resistance mobilised around the perimeter of the equivalent pier is likely to substantially exceed the applied working load. Hence, the load reaching the base of the pile group is likely to be negligible and the working load will be dissipated in shear around the pile-group perimeter. For this situation, an equivalent pier will be appropriate. In contrast, if the piles are mainly end-bearing or the pile group is relatively wide, then a substantial proportion of the applied working load will need to be resisted at the base of the pile group. In this case the equivalent raft will be an appropriate analogue for analysis purposes.

The choice between using either an equivalent raft or equivalent pier is mainly dependent on the overall pile-group aspect ratio or normalised width: **Figure 55.26** illustrates this quite clearly. When the normalised pile-group width (B/L; B is the group width and L is the pile length) is less than 1.0, the normalised pile-group stiffness is much higher than for the equivalent raft. Hence, the equivalent raft approach would tend to be overconservative and the equivalent pier should be used. Once the normalised width exceeds 1.0, then the pile-group stiffness tends towards that of an equivalent raft. Randolph (1994) has suggested that, for friction piles, if the pile-group aspect ratio exceeds about 3 or 4, then an equivalent raft is appropriate, whereas for lower values of R an equivalent pier is more appropriate. Typically, the accuracy of the equivalent pier is likely to be within 20% of more rigorous solutions, Randolph (2003), which is perfectly satisfactory for practical design purposes.

55.6 Differential settlement

Pile-group analysis commonly assumes that the pile cap is perfectly rigid or, occasionally, that the pile cap is perfectly flexible. Most pile-group analysis software cannot carry out analyses with a pile cap of intermediate stiffness. When pile caps are of small plan area, then assuming they are rigid is usually reasonable for practical purposes. However, as the pile-group size increases, it will be important to check the stiffness of the pile cap. This will be important for two reasons:

(i) as the pile-cap stiffness reduces the differential settlement across the pile group will increase;

(ii) as the pile-cap stiffness reduces the axial load distribution will become more uniform.

The decision to use pile groups is often driven by the wish to reduce settlement, in particular to reduce differential settlement, in order to reduce the risk of superstructure damage (refer to Chapter 52 *Foundation types and conceptual design principles*). Hence, a check on likely differential settlement will be important, especially for large pile groups or sensitive structures.

Pile-group differential settlement depends on the pile-group aspect ratio and on the pile-cap stiffness. Randolph and Clancy (1993) have shown that the normalised differential settlement of pile groups, with a fully flexible pile cap, is mainly a function of the pile-group aspect ratio R. Normalised differential settlement is reasonably independent of the precise number of piles, spacing, etc. Hence, for a flexible pile cap:

$$\Delta W_{\text{flex}} = f\,(R/4)\,W_{av} \qquad (\text{for } R \le 4) \qquad (55.20)$$

$$\Delta W_{\text{flex}} = f\,W_{av} \qquad (\text{for } R > 4) \qquad (55.21)$$

with $f = 0.3$ for differential settlement between the centre and mid-side, $f = 0.5$ for centre to corner; ΔW_{flex} is the differential settlement across a flexible pile cap, R is the pile-group aspect ratio and W_{av} is the average pile-group settlement.

The pile-group differential settlement ΔW is:

$$\Delta W = F_R\,\Delta W_{\text{flex}} \qquad (55.22)$$

where ΔW_{flex} is derived from equations (55.20) or (55.21) and F_R is the pile-cap rigidity, which can be assessed from solutions given in Chapter 53 *Shallow foundations* (for varying raft flexural stiffnesses).

55.7 Time-dependent settlement

Time-dependent settlement of pile groups under foundation loading can occur due to:

(i) primary consolidation, due to dissipation of excess pore pressure, in clays and silts (Hooper, 1979, Katzenbach *et al*., 2000);

(ii) settlement due to creep, in sands, gravels and weak rock (Mandolini and Viggiani, 1997).

The pile-group aspect ratio and load transfer mechanism are important factors in controlling the amount of time-dependent settlement. For small aspect ratios when load transfer is primarily due to shear, the time-dependent settlement at normal working load would be expected to be small. Studies by Poulos (1993) indicate that consolidation settlement would be less than about 15% of total settlement. In contrast, compression of the ground below the pile-group base will become increasingly important when the aspect ratio increases to large values (say, in excess of 3 or the B/L values are greater than 1.0), and then time-dependent behaviour will approach that for shallow raft foundations. Mandolini *et al*. (2005) compared observations of 'end of construction' settlement with observations of time-dependent movements for rafts and piled rafts in Frankfurt and London Clay (both stiff overconsolidated plastic clays). End of construction settlement (which probably includes some primary consolidation settlement) varied between 50% and 75% of the total, with the ratio increasing as B/L reduced (from 2.5 to 1.0). Poulos and Small (2001) suggest that creep settlement may become important at low factors of safety, i.e. less than about 1.4, but that creep is unlikely to be significant at normal working loads.

A separate issue that can cause relatively large time-dependent settlement is when large groups of full displacement (closed-end) piles are driven at close spacings into soft clays, silts or peats. Significant heave and lateral displacement can occur during pile driving. The shearing and remoulding of soft soils can generate substantial excess pore pressures, which will lead to large settlement of the heaved clay as the pore pressures dissipate. If the pile group is end-bearing then negative skin friction forces will result and if it is acting in friction then large pile-group settlements will develop (Bjerrum, 1967; Adams and Hanna, 1970; and Brzezinski *et al*., 1973). This will be independent of, and additional to, any settlement due to foundation or structure loading.

55.8 Optimising pile-group configurations

Once the decision has been made to use pile groups, some thought will be needed on how to optimise the layout of the pile group in terms of the length, diameter and spacing of the piles. This will be dependent on a number of factors:

(i) *A shallow foundation would have adequate bearing capacity, but is it likely to suffer excessive settlement?* If a shallow foundation such as a raft has adequate bearing capacity and the requirement is to control settlement, differential settlement or raft bending moment, then a piled raft may be the appropriate solution; this is discussed in more detail in Chapter 56 *Rafts and piled rafts*. A piled raft would usually comprise a small number of widely spaced piles, normally located beneath the superstructure columns.

(ii) *Does a shallow foundation have adequate bearing capacity?* If not a conventional pile design approach would be

utilised. A single pile may be adequate (Chapter 54 *Single piles*), although for heavily loaded foundations a deep large-diameter pile may be needed. An alternative may be to form a small pile group by installing several smaller piles and a pile cap. This may be advantageous since the mobilisation costs for a smaller piling rig would be lower: the overall cost-benefit of different piling options will be dependent on several site-specific issues (refer to Chapter 52 *Foundation types and conceptual design principles*), in particular, site access and headroom. A holistic approach to site development is needed in terms of the requirements for all foundations across the site and matters such as overall construction sequence and programme, and the available space for foundation construction plant and for building pile caps.

(iii) *Site geology*: if there is a distinctive competent bearing stratum at depth and the piles act predominantly as end-bearing piles, then the appropriate pile length will be obvious, and the main design variables will be diameter and spacing. Alternatively, if the bearing stratum exhibits a gradual increase of strength or stiffness then the benefit of increasing the pile length, versus the number or diameter of piles, may not be immediately obvious; this is discussed in more detail below.

(iv) *Applied loads*: if high horizontal loads will be applied, then increasing the pile diameter will generally be more effective than increasing the pile length. A critical pile length L_C can be defined (Fleming *et al*., 2009) for lateral loading. L_C/d depends on the relative soil to pile stiffness, but is typically between $6d$ and $10d$ (where d is the pile diameter). Piles longer than L_C will not reduce lateral pile deflection. Structural strength will often be the critical issue for laterally loaded pile groups, and structural strength rapidly increases with increasing pile diameter. For friction piles, if vertical loading and associated pile-group settlement is the key consideration, then increasing the pile length will usually be more cost-effective than increasing the pile diameter. For friction piles there is a critical pile length L_t beyond which further increases in length will not reduce settlement (Fleming *et al*., 2009). L_t is approximately 1.5 $(E_p/G_L)^{0.5}$ d, where E_p is Young's modulus for the pile concrete, G_L is the shear modulus at the pile toe and d is the pile diameter. For friction piles an equally important consideration is the pile spacing. Provided ultimate group capacity is adequate, a relatively small number of widely spaced piles can provide similar pile-group stiffness as a larger number of more closely spaced piles. The design charts, shown in **Figure 55.20**, can facilitate a rapid comparison between the effectiveness of varying any of pile-group length, diameter and spacing on pile-group settlement performance. Pile spacing is often chosen to be $3d$ for conventionally designed pile groups; increasing the pile spacing to $4d$ or $5d$ may be worthy of consideration since pile interaction effects will be substantially reduced. Pile spacings below $3d$ are usually inappropriate since pile interaction effects become more significant and the potential benefits (in terms of bearing capacity and settlement) may be negligible relative to the increased costs.

In general, on the basis of observed performance (e.g. Mandolini *et al*., 2005) it is reasonable to assume that pile-to-pile interaction is negligible for practical purposes at pile spacings of $8d$ or more for both vertical and horizontal loading. At these spacings, the pile design can be based on single-pile behaviour; refer to Chapter 54 *Single piles*.

Raking piles: raking piles are sometimes used to support foundations subject to horizontal loads and they can provide a very stiff support system. Although horizontal deformation will be relatively small compared with vertical piles (under an equivalent horizontal load), the use of vertical piles is usually favoured for several practical and technical reasons:

(i) Construction tolerances tend to be far poorer for raking piles than for vertical piles. The relevant tolerances are the position and alignment tolerances, that is, comparing the intended toe position with its actual position.

(ii) Construction problems: e.g. installation damage for driven piles or potential integrity problems for bored cast *in situ* piles. Any potential problems associated with pile installation, such as groundwater inflows, pile bore instability, penetration into or through hard strata or obstructions, etc. will all be far more challenging for raking than vertical piles. Because of this, raking piles will often be more expensive and slower to install than vertical piles.

(iii) Raking piles should *not* be used in situations where there are likely to be large global ground movements, such as for bridge foundations on soft soils when adjacent approach embankments are likely to induce large vertical and horizontal ground movements (or, for similar reasons, adjacent to deep excavations); nor for heavily loaded foundations that are likely to undergo large settlement. In both situations the relative soil–pile movements are likely to induce bending stresses in the raking piles. In these circumstances, raking piles are vulnerable to being overstressed.

Raking piles were often used historically because of the lack of adequate analytical tools to assess the lateral load-deformation behaviour of vertical piles. Nowadays, there is a wide range of elastic (and nonlinear) continuum pile-group analysis software available to practitioners, which allows lateral load behaviour to be readily analysed. Therefore, for most circumstances, groups of vertical piles should be adequate. Nevertheless, there are some situations when raking piles can be cost-effective, examples include maritime applications when vertical loads may be negligible but there are substantial horizontal loads, such as for ship berthing facilities. Marine applications also

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tend to favour the use of driven piles, which are generally better suited to being installed at a rake than bored piles.

55.9 Information requirements for design and parameter selection

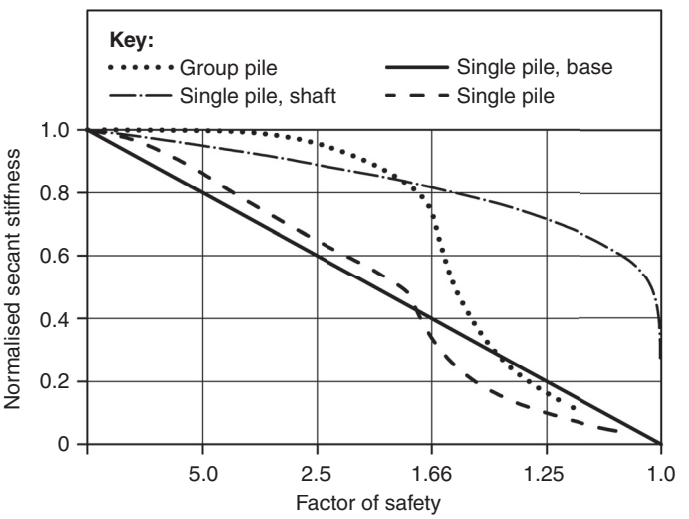
Golder and Osler (1968) described the performance of five pile groups (each with 32 piles, 20 m long), which were constructed to support an industrial facility. The piles were founded at a high level in a layer of dense sand. A pile test indicated a settlement of about 1 mm at the working load and pile-group settlement was expected to be less than 10 mm. Fifteen years after construction, the observed settlements were more than 70 mm and continuing to increase. The settlement of the pile groups at each end of the row of five groups showed significant tilt, due to interaction with the adjacent pile groups. Subsequent analysis indicated that layers of compressible clay (about 3 m below the pile-toe level) were mainly responsible for the excessive pile-group settlements, which were predicted to reach 90 mm. About 80 mm, out of the total of 90 mm, was due to settlement of the underlying clay layers. This (and other) experience indicates that supporting a structure on a large pile group does not necessarily eradicate all risk of excessive foundation movement. It also highlights the importance of a proper understanding of the overall ground conditions and ground–structure interaction. Clearly, if considered in isolation, the results of a pile load test can be misleading.

The requirements for a proper desk study and comprehensive ground investigations are just as important for piled foundations as they are for other aspects of geotechnical engineering. For pile groups, an appreciation of the relative strength and compressibility of strata below the pile group is particularly important. A minimum of the pile-group width and, depending on the overall geology, up to two to three times the width should be investigated below the pile-toe level. Hence, deep boreholes may be necessary, together with careful logging and index testing at close centres, to check for weak layers within and below the anticipated bearing stratum. If lateral loading is significant, then the strength and compressibility of near-surface soils (typically the uppermost 6*d*, where *d* is the pile diameter) will be particularly important. This is challenging since near-surface soils can be very variable. Cost-effective investigations of near-surface soils will probably include some form of penetration or probing tests (refer to Chapter 47 *Field geotechnical testing*) to obtain rapid and cheap coverage across the site, together with several trial pits to facilitate interpretation.

The critical load case for pile-group design (especially for bridges and maritime structures) will often include a significant transient live load (due to, for example, vehicle impact or wind loading). For this type of short-term loading then undrained strength and stiffness parameters should be used for pile groups founded in clays.

Randolph (1994) discusses the implications of nonlinear soil behaviour on both single-pile and pile-group settlement. **Figure 55.27** summarises the output from these analyses, plotted as normalised secant stiffness versus factor of safety for a large pile group. The pile-group settlement remains relatively linear until the overall factor of safety reduces below about 1.6. At a factor of safety of 2.0, the secant stiffness for the soil within the pile group is more than 80% of the initial tangent stiffness. The relationship between choice of elastic modulus and pile-group size is shown schematically in **Figure 55.28**. **Table 55.4** provides guidance on the selection of the appropriate linear elastic moduli for single piles and pile groups of varying size at different factors of safety.

For lateral loading, the pile-group behaviour can be more strongly influenced by soil stiffness degradation. For linear



NB.
Normalised secant stiffness = 1.0, when G_{mob} (E_{mob}) = G_0 (E_0); G_{mob} = mobilised shear modulus, G_0 = shear modulus at very small strain.
E = Young's Modulus. For large pile group (>25 piles)

Figure 55.27 Normalised secant stiffness versus factor of safety. Nonlinear settlement for a single pile and a pile group
Modified from Randolph (1994)

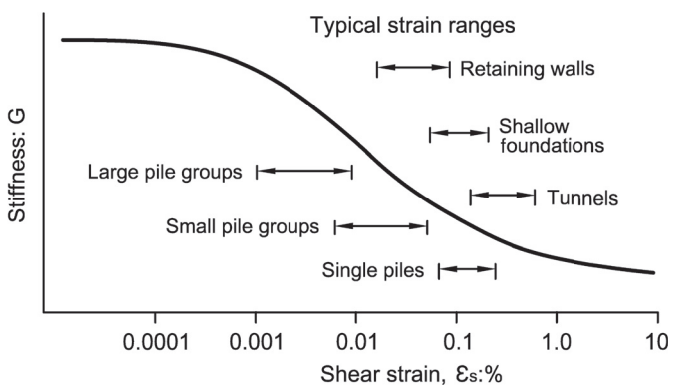


Figure 55.28 Influence of pile-group size on the selection of the appropriate 'elastic' modulus

Factor of safety	G/Gmax		
	Single pile	Small- to medium-sized pile group	Large pile group
> 5.0	> 0.6	0.7 to 0.8	> 0.9
3.0	0.4 to 0.5	0.6 to 0.8	0.8 to 0.9
2.0	0.3 to 0.45	0.5 to 0.7	0.7 to 0.8

Notes:
The above values are for vertical loading and are intended as preliminary guidance only. A small- to medium-sized pile group has 5 to 25 piles. A large pile group has more than 25 piles. G_{max} is the shear modulus at very small strain for vertical shear. The values are relevant for linear elastic analysis and they assume the pile spacing is 3d. As the spacing increases, the guide values will be increasingly conservative.

Table 55.4 Tentative values of shear modulus ratio (G/Gmax)

elastic analyses, this can be crudely replicated by selecting a large strain (say a strain amplitude of 1% or above) stiffness for the soil adjacent to the top of the pile, and an intermediate strain (say a strain amplitude of 0.1%) stiffness for the soil at a depth equal to the critical pile length L_c with a linear variation between these levels. For laterally loaded pile groups, the sensitivity of pile bending moments and group deformation to variations in shallow soil stiffness (within the upper 6d to 10d, where d is the pile diameter), perhaps by doubling and halving soil stiffness compared with a best estimate, should be checked. For critical situations, specialist advice should be sought and more sophisticated nonlinear analyses may be required. Fleming *et al*. (2009) suggest assuming shear moduli for lateral loading that are a factor of two lower than those used for axial loading, in recognition of the high strain levels, which will develop locally adjacent to the top of a laterally loaded pile. An alternative recommendation by Fleming *et al*. (2009) is to assume zero shear modulus at the ground surface, increasing to the full value for axial loading at a depth equal to L_c (the critical pile length for lateral loading). Hardy and O'Brien (2006) discussed the influence of soil nonlinearity on the predictions of bending moment in pile groups subjected to horizontal loads. **Figure 55.29** shows that the pile bending moments are sensitive to the soil constitutive model (hyperbolic or linear elastic) and, for the linear elastic model, the magnitude of the selected moduli. If the design is substantially influenced by lateral load effects then nonlinear analyses offer the possibility of more economic design and more realistic predictions of pile-group deformation (O'Brien, 2007). However, these analyses are more challenging and will require specialist input and guidance. The above observations are consistent with those reported elsewhere (e.g. Mandolini *et al*., 2005). Clearly, for lateral loading when using linear elastic analyses a cautious approach is needed and some checks on the sensitivity of pile-group deformation and pile stresses to variations in shear modulus are essential.

When considering the selection of input parameters for pile-group analysis, and, in particular, comparing single pile with

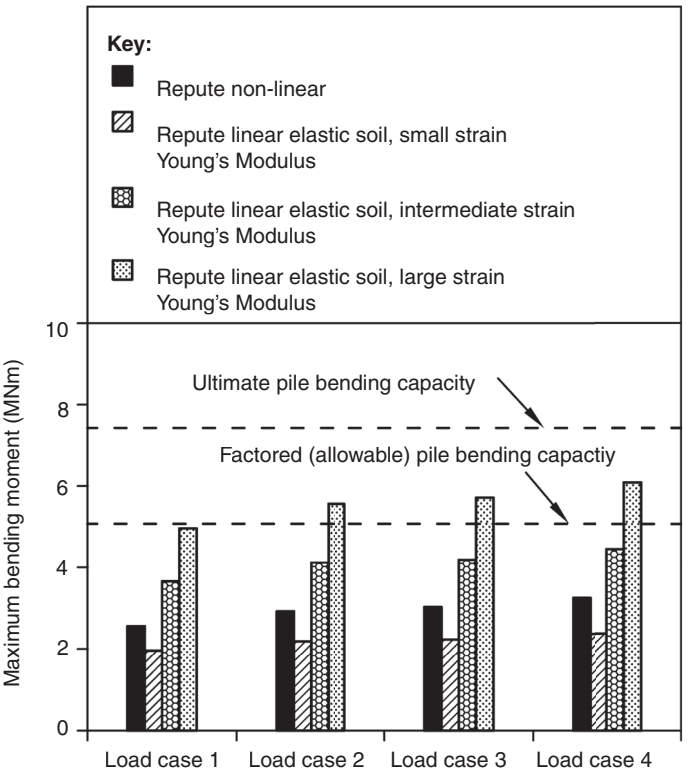


Figure 55.29 Influence of soil model and selected Young's modulus on the peak bending moment under horizontal loading
Reproduced from Hardy and O'Brien (2006)

group performance, it is helpful to consider the different regimes that will influence behaviour, **Figure 55.30**, and the relevant information, which will be needed for making decisions. Five separate zones, and associated moduli, can affect behaviour:

(i) The secant stiffness of the soil or rock close to the pile shaft. The stiffness of this zone will strongly influence the settlement of a single pile at typical working loads. This zone will be affected by the construction process (remoulding and horizontal stress changes), either due to pile boring or pile driving.

(ii) The stiffness of the soil or rock close to the pile base. The stiffness of this zone is dominated by the construction process and is more sensitive to the construction process than shaft stiffness. The base stiffness will be important for end-bearing piles; however, for many piles (which often gain significant shaft resistance) the base stiffness only becomes influential at low factors of safety. For driven piles the base stiffness will be relatively high, compared with the original soil stiffness. For bored piles, the ground stiffness will tend to be lower, sometimes much lower, than the original soil or rock stiffness, and will be dependent on factors such as the effectiveness of base cleaning activities (Fleming, 1992).

(iii) The stiffness of the ground close to the ground surface, will strongly affect pile behaviour under horizontal

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loading. Ground investigations frequently overlook the characterisation of this zone and it can be heterogeneous. It should also be emphasised that site redevelopment (excavation and filling) will often change the nature of this zone. Pile concrete stiffness under lateral loads will tend to be lower (due to yield, creep and cracking) than that for compression.

(iv) Soil stiffness between the piles. The stiffness of this zone is important for pile-group settlement. This zone will not be significantly affected by construction activities. As noted earlier, the relevant modulus, for linear elastic analysis, will be close to the very small strain stiffness of the soil or rock and will also depend on the pile-group size.

(v) Soil stiffness well below the pile-toe level. As for (iv) this zone will not be affected by construction activities. It will not affect the behaviour of single piles, but will influence pile-group behaviour. The settlement of pile groups with a large pile-group aspect ratio will be strongly influenced by these deeper layers.

Hence, the information required will involve a consideration of:

(i) Construction effects and the back analysis of single-pile behaviour (for example, Fleming, 1992; England, 1999). For laterally loaded piles, tests on single piles will usually be free headed whereas piles in a group will generally be fixed headed. This different head fixity must be allowed for when comparing isolated piles and pile-group behaviour.

(ii) Data from relevant case histories (for example, Mandolini *et al.*, 2005). Case histories are particularly useful for calibration of software. Software calibration is particularly important for large pile groups or more sophisticated non-linear analytical methods.

(iii) Site-specific ground investigation data. Useful additional data (although seldom provided by routine investigations) would be measurements of G_0 or G_{max}. Modern empirical correlations (for example Atkinson, 2000) are helpful for the selection of the appropriate moduli.

Pile capacity is mainly dependent on local conditions at the pile–soil interface, which are strongly affected by construction methods and exhibit significant variability (NCHRP Report 507, 2004). In contrast, pile-group stiffness is determined primarily by far-field conditions, and is relatively unaffected by construction processes (**Figure 55.31**). Hence, pile-group deformation can be more reliably predicted than the ultimate capacity of a single pile (Randolph, 2003). This is in stark contrast to many other aspects of geotechnical engineering (such as predicting movements around retaining walls or shallow foundation movements), where predicting movement is perceived as more difficult than predicting ultimate capacity.

Piles around the perimeter of a pile group tend to carry higher loads than those in the centre. Hence, it would usually

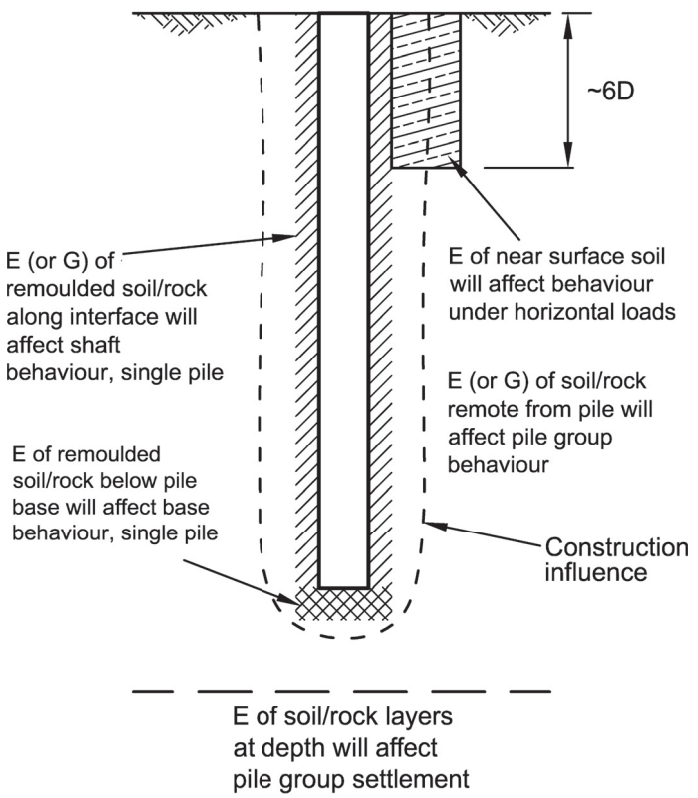


Figure 55.30 Different ground moduli that may influence single-pile and pile-group behaviour

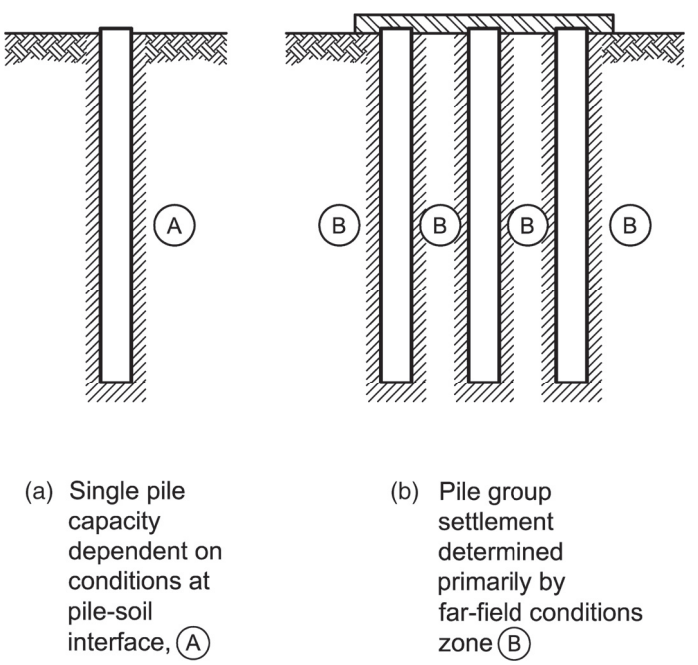


Figure 55.31 Influence of (a) the pile–soil interface on single-pile capacity and (b) far-field conditions on pile-group settlement
Reproduced from Randolph (2003)

be appropriate to focus most of the pile testing effort (both integrity and load tests) on the perimeter piles.

55.10 Ductility, redundancy and factors of safety

Burland (2006) highlighted the profound importance of understanding the mode of foundation failure mechanisms. Routine geotechnical and structural engineering practice is based on the assumption of ductile behaviour. The load-settlement behaviour of pile foundations is usually ductile. However, this is not always the case; for example, experience indicates that some rock socket piles can exhibit brittle behaviour, examples include: piles socketed into some rocks, such as limestone and sandstone; or end-bearing piles toed into thin competent strata and underlain by soft clays. If brittle behaviour is anticipated then a cautious approach is essential. Rather than summing shaft friction and end-bearing capacity, it would be more appropriate to consider a deformation-based capacity (i.e. discounting some fraction of either shaft or end-bearing resistance). Preliminary pile tests to failure would clearly be helpful. Pile-group design would need to limit pile-group deformation to ensure brittle failure was not induced.

For more normal situations, when pile load-settlement behaviour is ductile, the following issues will influence the selected factors of safety:

(i) pile-cap and sub-structure stiffness;

(ii) number of piles in the group (i.e. level of redundancy);

(iii) code requirements;

(iv) nature and direction of applied loading, e.g. cyclic or monotonic loads, vertical or horizontal loads;

(v) analysis methods;

(vi) reliability and scope of ground investigations.

Table 55.5 gives a commentary on the above issues.

Many codes and standards do not make specific statements about factors of safety for pile-group design. However, Eurocode 7 provides the following (Clause 7.6.2.1):

- The stiffness and strength of the structure connecting the piles in the group shall be considered when deriving the design resistance of the foundation.
- If the piles support a stiff structure, advantage may be taken of the ability of the structure to redistribute load between the piles. A limit state will occur only if a significant number of piles fail together; therefore a failure mode involving only one pile need not be considered.
- If the piles support a flexible structure (or flexible pile cap), it should be assumed that the compressive resistance of the weakest pile governs the occurrence of a limit state.
- Special attention should be given to possible failure of edge piles caused by inclined or eccentric loads from the supported structure.

In general, the axial loads calculated by linear elastic analyses should *not* be compared with the geotechnical capacity of individual piles. As noted by Burland (2006), when attempts are made to satisfy conventional factors of safety for each and every pile in a pile group (compared with axial loads from linear elastic analyses), then the result is a grossly conservative pile-group design with significant (and unnecessary) cost increases. A pragmatic solution for small vertically loaded pile groups is to design the piles and pile cap to have sufficient structural strength (compared with predicted forces from elastic analysis), and then check that the overall geotechnical capacity for the group (the sum of individual piles or block enclosing the group perimeter, whichever is less) satisfies the code factor of safety against geotechnical failure. For large pile groups (in excess of 25 piles) with rigid pile caps, nonlinear analyses can provide more realistic axial load distributions. For large pile groups, the flexibility

Factor	Comment
Pile-cap and sub-structure stiffness	A stiff pile cap or sub-structure can redistribute axial loads; hence, the individual pile factor of safety is not significant. If the pile cap is flexible then individual pile factors of safety need to be considered.
Number of piles in the group	If more than 5 piles, then there is redundancy and 'failure' of a pile within group does not imply failure of the group. For large pile groups there is considerable redundancy.
Code requirements	Many codes do not discuss pile groups in detail and mainly focus on single piles. EC7 provides some guidance, see text. AASHTO (and NCHRP Report 507, 2004) gives guidance on reduced risk of failure associated with varying levels of redundancy.
Direction of loading	For horizontal and moment loading, carefully check structural strength of piles, pile-cap and pile-to-pile cap connections. A factor of safety in excess of 1.3 to 1.4 along the perimeter pile row should be met if large long-term moment loading, to avoid excessive group rotation in the long term (creep).
Analysis method	For nonlinear methods reliable calibration of model is important. Use simple methods to check factor of safety against failure. Computer-based methods more appropriate for assessing deformation and stresses induced in piles.
Reliability and scope of ground investigations	Most important factor to consider. Especially important to verify the strength and stiffness of layers below the pile group. Near-surface materials important for laterally loaded pile groups. The greatest uncertainty lies with establishing the geological model, the idealisation of the ground profile for analysis and the selection of appropriate geotechnical parameters.
Nature of loading	Guidance in this chapter is solely for pile groups with predominantly static, monotonic loading. Under prolonged cyclic loading, significant degradation of shaft resistance can occur, with associated substantial increases in deformation and reductions in ultimate capacity (Jardine, 1991).

Table 55.5 Factors that may influence the factor of safety for a pile-group

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of the pile cap can substantially influence the axial load distribution and should be checked. Hence, for vertically loaded pile groups, a key issue is to check that the pile cap has adequate structural strength and stiffness to be able to safely redistribute the axial load across the group, and then the geotechnical factor of safety need only be applied to the overall group resistance, *not* to individual piles. For pile groups subject to moment or horizontal loads, the geotechnical resistance of the outer rows of piles should be checked (again the geotechnical capacity of individual piles does not need to be considered).

In the context of assessing an appropriate geotechnical factor of safety for axial resistance, the redundancy of a foundation system is also an important consideration. Design guidance in the USA (NCHRP Report 507, 2004) suggests that a fully redundant pile group (of more than five piles) requires a factor of safety of about 70% of that for a single pile, based on achieving the same overall reliability (i.e. the statistical probability of failure is the same). This is a logical consequence of the inherent variability of the geotechnical capacity of individual piles, given the significant influence of construction methods on ultimate resistance. If the consequence of pile failure is severe, say a single pile supports a bridge pier (where there may be limited opportunity for load redistribution), then the appropriate factor of safety may need to be larger than the code requirement. It follows that for a large pile group (say 25 or more piles) with a stiff pile cap, then the appropriate geotechnical factor of safety for axial resistance could be lower than normally required. However, for large pile groups the geotechnical factor of safety is unlikely to be critical; pile-group deformation and structural forces induced in the piles and pile cap are likely to be more critical.

The structural strength of piles, pile cap and pile-to-pile cap connections should always be checked and be sufficient to resist the induced axial forces, bending moments and shear forces. For pile groups subject to large moment or horizontal forces this is particularly important, since inadequate structural strength in any part of the overall foundation system could lead to a brittle and progressive collapse of the structure.

55.11 Pile-group design responsibility

Two options are normally considered: contractor's design or engineer's design.

(a) *Contractor's design* – This is commonly used, especially for small building projects. The engineer employed by the client as the overall project designer will provide the piling tenderers with relevant information, including: site history, ground conditions, applied structural loads, acceptance criteria for pile loading tests, etc. Then the contractor is responsible for detailing the appropriate pile length, diameter and reinforcement to resist the specified loads. The pile-cap is usually designed by the overall project designer.

(b) *Engineer's design* – Under this arrangement, design responsibility lies with the overall project designer, who would design and detail the pile lengths, reinforcement, etc. This arrangement is often followed for major civil engineering projects, such as bridges or large building projects. The contractor is responsible for workmanship and method of construction.

There are advantages and disadvantages with both options. The specialist piling contractor will have a lot of expertise on how to best utilise his materials, plant and labour. He may also have local experience on the likely pile capacities that can be achieved in particular geological sequences. Therefore option (a) will favour those situations where the geotechnical pile capacity is the dominant consideration, where buildability is very challenging (due to poor access, etc.) or if particular proprietary piling systems are well suited to local conditions. Option (a) is less appropriate if pile-group deformation is critical or if there are broader ground–structure interaction problems (due to complex loading or if global ground movements are a concern). A particular problem with option (a) for pile-group design is the appropriate specification of pile loads and pile test acceptance criteria. For example, axial loads will not be uniform across the group, and the stresses induced in the piles and pile cap will be a function of the pile length, diameter, etc., and the pile-cap bending stiffness. The deformation of the pile group is dependent on several factors and not just the settlement characteristics of an isolated pile. Hence, the overall designer needs to have a good understanding of these soil–structure interaction issues before realistic and sensible specification requirements for pile capacity and settlement (for an isolated pile) can be developed. In many cases, if design responsibility is to be split then it is more appropriate to split responsibility at the top of the pile-cap level rather than at the top of the pile level. The overall designer then only needs to specify the appropriate loads at the pile-cap level, and the acceptable limits for overall pile-group deformation and differential settlement across the pile cap. Option (b) will also be more appropriate for large pile groups or for pile groups underlain by compressible layers (where single-pile performance cannot be easily correlated with pile-group performance).

55.12 Case history

Pile-group analyses are briefly described for the Emirates Twin Towers in Dubai and they are compared with observed performance. The foundation design is described in detail by Poulos and Davids (2005).

The ground conditions mainly comprise interbedded calcareous sandstone and siltstone. The measured unconfined compressive strength is variable and typically between about 0.5 MN/m^2 and 1.5 MN/m^2. The stiffness at very small strain was measured by cross-hole geophysical tests and shear moduli varied between 2 and 3.5 GN/m^2. A summary of the ground profile is given in **Table 55.6**. The twin towers were founded on two triangular-shaped pile groups comprising 92 and 102 1.2-m-diameter piles, 40 m long. The estimated ultimate geotechnical capacity of a single pile was about 42 MN.

Strata number	Strata designation	Typical depth of strata base (m bgl)	Description
1	Silty sand	5	Uncemented calcareous silty sand, loose to moderately dense
2	Sand	10	Variably and weakly cemented calcareous silty sand
3	Calcareous sandstone	29	Calcareous sandstone, slightly to highly weathered, well cemented
4	Cemented sand (?)	35	Calcareous silty sand, variably cemented, with localised well cemented bands
5	Calcareous siltstone and conglomerate	56	Variably weathered, very weakly to moderately well cemented
6	As Unit 5	70	As Unit 5

Table 55.6 Case history: ground profile

Strata	Linear elastic[1]	Nonlinear[2]	
	Secant Young's modulus (MN/m²)	Tangent Young's modulus (small strain) (MN/m²)	Interface Strength τ(kN/m²)
Silty sand	15	20 + 8Z	20 + 5.5Z
Sand	50		
Calcareous sandstone	250	1500 + 40Z	150 + 12Z
Cemented sand (?)	50		
Calcareous siltstone and conglomerate	200	3000 + 50Z	500

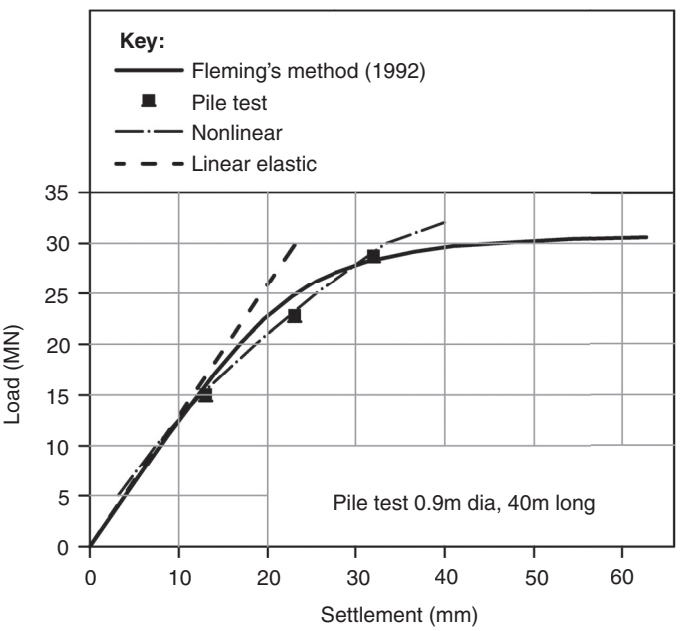
Notes:
1. Linear elastic model, pile base stiffness, E′ = 40 MN/m².
2. Non-linear model, hyperbolic constants, *R* shaft = 0.65, Rbase = 0.99, maximum shaft friction = 500 kN/m², maximum base pressure = 2700 kN/m² and pile base modulus E'_0 = 750 MN/m² (E'_0 = small strain tangent stiffness).

Z is the depth below the ground surface.

Table 55.7 Linear elastic and nonlinear input parameters used in the REPUTE analyses of the case history

Based on data published by Poulos and Davids (2005) two different analyses have been carried out, the first using a computer-based method and the other using a simple empirical method. The computer-based analysis, using the software REPUTE, was used with two different models to simulate the stress–strain behaviour of the ground. The models were a linear elastic model and a nonlinear hyperbolic model. The input parameters are summarised in **Table 55.7**. The load-settlement behaviour of a single pile was modelled initially and calibrated against preliminary pile test data for a 0.9 m-diameter, 40-m-long pile, **Figure 55.32**, prior to modelling the entire pile group. For the single-pile calibration, Fleming's method (Fleming, 1992) was helpful in deriving reasonable nonlinear parameters for use in the hyperbolic model.

The analysis predictions of the axial load distribution across the pile group and the pile-group settlement are summarised in **Figure 55.33**. The linear elastic analysis predicts a highly non-uniform distribution of the axial load; in particular, axial loads in the corners are especially large. The linear elastic analysis predicts maximum axial loads of 43 MN compared with a maximum of 31 MN from the nonlinear analysis. The ratio of corner to central pile axial load from the linear elastic analysis is about 5, and for the nonlinear analysis, it is about 2. Hence, the local factor of safety for a pile in the group (based on linear elastic analysis) is less than 1.0, although the overall factor of safety for the pile group is about 2.0. This is similar to the results from boundary element analysis reported by Poulos and Davids (2005), where they state that 'a number of the piles reached their full geotechnical design resistance (i.e. the factor of safety for some piles was about 1.0), but the foundation



At a load of 15MN, Geo Factor of safety ~ 2.0	
Analysis/Data	Settlement (mm)
Linear elastic	11.6
Nonlinear	12.3
Pile test	12.9

Figure 55.32 Case history: pile test in weak rock

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as a whole could still support the ultimate design loads'. The pile-group settlement was observed to be about 8 to 10 mm after 70% of the load had been applied, and the estimated final settlement is between 20 and 40 mm. The nonlinear analysis indicates a settlement of 25 mm, compared with 55 mm from the linear elastic analysis (**Figure 55.33**).

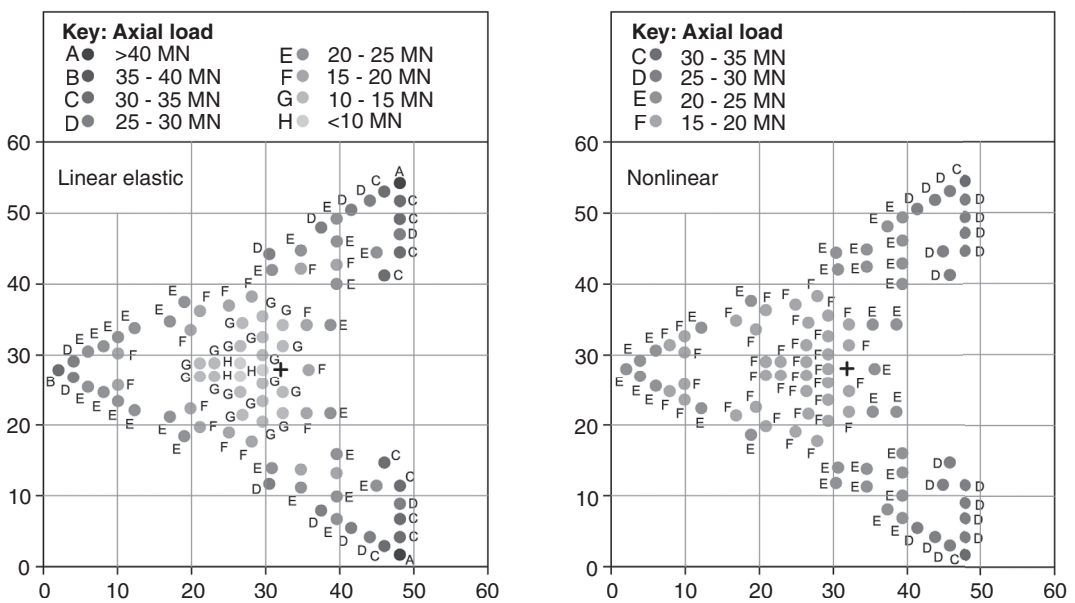
The pile-group aspect ratio is about 2.9 and Mandolini's empirical correlations for pile-group settlement (refer to section 55.5.2) gives settlement ratios R_{se} between 4 and 7. Based on the site geology (weak rock with stiffness increasing with depth) the best-estimate and lower-bound pile-group settlement ratios are relevant (equations (55.9) and (55.12)). The single-pile settlement at the average working load of 21 MN is about 10 to 11 mm. Hence, based on Mandolini's empirical method, the pile-group settlement would be expected to be between about 40 and 75 mm. Although conservative relative to observations and the estimated final value, it is more accurate than the designer's original estimate of about 90 to 140 mm (based on a linear elastic boundary element analysis). Poulos and Davids describe the two main reasons for the initial overprediction using boundary element analysis:

(i) overestimation of pile-to-pile interaction;

(ii) underestimation of the ground stiffness for Unit 5 (**Table 55.6**).

Poulos and Davids produced revised analyses where the stiffness varied radially from each pile, so that the ground stiffness between the piles was five times stiffer than that adjacent to the piles (to values of about 2 to 2.5 GN/m^2, i.e. close to the geophysics, G_0, values), and the ground stiffness for Unit 5 was increased from 80 to 600 MN/m^2. This revised analysis indicated a settlement of between 23 and 40 mm, (allowing for differential settlement across the pile cap). With the benefit of hindsight, it seems likely that the laboratory-measured stiffness for Unit 5 was too low because of sample disturbance.

This case history re-emphasises the challenges in deriving realistic ground stiffness values, especially in weak rocks, and the potential errors even when sophisticated analyses are utilised. It also highlights the value of simple empirical methods to check computer output. It is likely that the axial loads around the perimeter of the pile group were high and the factor of safety against geotechnical failure for the perimeter piles was much lower than code values (nonlinear analyses predict 'local' factors of safety of about 1.3 to 1.4 for the perimeter piles). Nevertheless, the overall group factor of safety is code compliant (equal to about 2.0), the piles and pile cap have adequate strength and stiffness to redistribute forces across the group and therefore the overall pile-group performance has been satisfactory.



1.2m dia piles, ultimate capacity ~42MN, Geo Facor of Safety for group ~2.0		
	Peak Axial Load	Group Settlement (mm)
Linear elastic	43MN	55mm
Nonlinear elastic	31MN	25mm
Estimated settlement (*in situ* measurements) ~20 to 40mm		

Figure 55.33 Case history: pile group in weak rock. Axial load distributions and settlement for the linear elastic and nonlinear models

55.13 Overall conclusions

- The overall behaviour of pile groups is affected by their geometry. An important parameter is the pile-group aspect ratio (**Figure 55.3**). Where R is large, the group settlement will be many times larger than the settlement of an isolated pile (at the same average load). For pile groups with a large aspect ratio, $R > 3$, then an 'equivalent raft' analogy is appropriate for simplified analyses; where R is < 3 then an 'equivalent pier' analogy will be more appropriate.
- The failure of pile groups is rare; the risk of failure increases when: global movements develop around the group (typically for out-of-balance loads on soft clays and peats); end-bearing piles toed into a thin competent layer are underlain by soft clay; or piles are toed into steeply dipping fractured rock or rock containing voids.
- Pile-to-pile interaction, under vertical or horizontal loads, depends on many factors (**Table 55.1**), including pile spacing and the variation of ground stiffness with depth, especially 'hard' or 'soft' layers beneath the pile toe (for vertical loads) or adjacent to the pile head (horizontal loads). For horizontal loads, the pile head fixity is important, whereas for large pile groups under vertical loads the pile-cap stiffness is important.
- For small- to medium-sized vertically loaded pile groups (say, less than 16 to 25 piles), linear elastic analysis will usually be adequate for a practical design. However, for larger groups or pile groups subject to large horizontal loads, linear elastic analyses are likely to be inappropriate. For small- to medium-sized pile groups simplified methods of analysis (based on empirical settlement ratios, elastic interaction factors or an equivalent pier) will often be sufficient to calculate the group settlement.
- When considering the selection of input parameters for pile-group analysis, it is helpful to consider the five separate zones (and associated deformation moduli) that can affect behaviour (**Figure 55.30**): (i) the stiffness of the ground adjacent to the pile shaft, (ii) the zone immediately below the pile toe, (iii) the stiffness of the ground close to the pile head, (iv) the ground stiffness between the piles and (v) the ground stiffness at depth below the pile-toe level.
- The appropriate values for factors of safety selected for pile-group design depend on several factors, refer to **Table 55.5**. These include pile-cap and sub-structure stiffness, the number of piles in a group (redundancy), code requirements, the nature and direction of loading, analysis methods and the reliability and scope of ground investigations. In addition, the ductility or brittleness of pile load-settlement behaviour is an important consideration.
- If design responsibility is split for pile-group design then careful thought is required regarding the most appropriate point to divide responsibility (pile-head or pile-cap level). When pile-group deformation is critical, or if global ground movements or complex loads are applied, then pile and pile-cap design should be considered together, so that the soil–structure interaction is properly assessed.

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It is recommended this chapter is read in conjunction with

- Chapter 19 *Settlement and stress distributions*
- Chapter 22 *Behaviour of single piles under vertical loads*
- Chapter 54 *Single piles*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 56
Rafts and piled rafts

Anthony S. O'Brien Mott MacDonald, Croydon, UK
John B. Burland Imperial College London, UK
Tim Chapman Arup, London, UK

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Rafts can provide a very cost-effective foundation solution. Their design requires a careful assessment of the soil–structure interaction. Many factors, including soil and raft stiffness, superstructure and sub-structure stiffness, local yield and soil nonlinearity and time-dependent effects, can influence differential settlement and structural forces in the raft. Errors associated with assuming that a raft sits on a bed of uniform soil 'springs' are discussed, and a practical alternative is outlined.

Piled rafts, which are a hybrid foundation comprising a raft and piles, can offer many benefits. Currently there is little guidance in the codes and there is confusion about appropriate design methods. Piled-raft behaviour is potentially very complex. Significant simplifications are possible, which facilitate practical design and greater confidence in analysis output. A key aspect is a clear definition of two different types of piled raft: the raft-enhanced pile group and the pile-enhanced raft (previously known as a raft with settlement-reducing piles). The design concepts for each type of piled raft are very different. The intermediate zone between these two types should be avoided.

56.1 Introduction

This chapter provides guidance on the design of rafts and piled rafts.

Chapter 52 *Foundation types and conceptual design principles* describes the main foundation types, which include shallow foundations such as pads or strip footings and deep foundations such as piles. If individual pad or strip footings are relatively large or closely spaced it may be more practical and cost effective to create a single reinforced-concrete raft beneath the structure. The rule of thumb that is commonly used is that if the pad or strip footings occupy more than about half of the superstructure footprint, then it is usually more appropriate to use a raft. However, site-specific issues may change this simple criterion. The four main reasons for using raft foundations are:

(i) The individual excavations for numerous pad or strip footings are relatively close to one another, and for construction purposes it is easier to create a single large excavation.

(ii) The differential settlement between individual footings is excessive, and the use of a relatively rigid raft is needed to minimise differential settlement to a tolerable value. As noted in Chapter 53 *Shallow foundations* (Figure 53.8) it is necessary to check the influences of the interaction between pads and strips when assessing the overall settlement and differential settlement of a structure. Figure E1.10 indicates that buildings on raft foundations have withstood large total settlements (in excess of 100 mm) with negligible damage, because the bending stiffness of the raft has been sufficient to reduce differential settlement to tolerable levels.

(iii) There is a reduced risk of damaging differential settlements in heterogeneous ground conditions: in some situations it may be difficult to confidently assess the likely total and differential settlement, and past experience has shown that rafts can provide effective and economic foundations that mitigate this risk. An example is the use of rafts for house foundations on deep layers of non-engineered fill (refer to Chapter 58 *Building on fills*), sometimes with the building weight balanced by partial excavation to reduce the net imposed load. There may be other situations where it is deemed appropriate to use rafts in order to minimise the risk of damage due to site-specific geohazards, refer to Chapter 52 *Foundation types and conceptual design principles.*

(iv) There is a reduced risk of bearing-capacity failure and excessive settlement or differential settlement through using cellular rafts in soft clays: the raft is placed at the base of an excavation and the building weight is compensated for by the weight of soil that is excavated.

Piled foundations are often used because there is a concern about excessive settlement of shallow foundations, even though a raft would have an acceptable factor of safety (FoS) against bearing-capacity failure. Conventional design of the pile group would then ignore the resistance provided by the pile cap or raft, and solely consider the resistance provided by the piles (refer to Chapters 54 *Single piles* and 55 *Pile-group design*); i.e. all the design load is transferred through the piles. In the past two decades there has been increasing recognition that this conventional assumption is overly conservative when shallow foundations can be founded on competent soils (such as stiff clays). It is now known that a hybrid foundation can be designed, which makes use of both a raft and piles, and allows for load sharing between the raft and piles. This foundation

type is usually known as a piled raft. Unfortunately, current codes and standards do not provide specific guidance on piled-raft design, and there are still some misunderstandings about the design of piled rafts. The main intent of this chapter is to provide clear and coherent guidance on the key concepts that should be used for piled-raft design.

There are several reasons why a piled raft may be used:

(i) to minimise total settlement, and, in particular, differential settlement;

(ii) to reduce bending moments and shear forces in the raft (especially effective where there are a small number of heavily loaded columns);

(iii) a further practical advantage from (ii) above is that the raft thickness can be minimised, overcongested reinforcement can be reduced, and, perhaps, excavation below the water table avoided.

This chapter builds upon many of the design issues described in Chapters 53 *Shallow foundations*, 54 *Single piles* and 55 *Pile-group design* and it is assumed that these will be referred to by the reader. This chapter is split into the following sections:

56.2 – *Analysis of raft behaviour*: describes the main soil–structure interaction issues that affect raft design and particularly the pitfalls associated with a 'raft on springs' design approach.
56.3 – *Structural design of rafts*: introduces the key practical design and detailing issues for raft design.
56.4 – *Design of a real raft:* reviews the main steps in the design process and buildability considerations, and includes a mini case-history example.
56.5 – *Piled rafts, conceptual design principles*: this outlines fundamental differences in behaviour between different types of piled raft, provides a new terminology for piled rafts, and gives guidance on appropriate ground conditions.
56.6 – *Raft-enhanced pile groups:* summarises the main design issues and outlines appropriate design approaches, including those for compensated raft-enhanced pile groups.
56.7 – *Pile-enhanced rafts:* introduces a simple design approach.
56.8 – *Case history*: of a pile-enhanced raft.
56.9 – *Key points:* provides a summary of the main design issues.

56.2 Analysis of raft behaviour

56.2.1 Design requirements

Raft design requires an assessment of:

(i) total settlement, and, particularly, differential settlement;

(ii) bending moment and shear forces across the raft, either due to uniformly distributed loads or concentrated loads from superstructure columns;

(iii) practical issues related to the thickness of the raft, the density of reinforcement, location of the groundwater table, size of excavation, etc.

Total settlement can be assessed by the methods of analysis outlined in Chapter 53 *Shallow foundations*; (iii) above will be discussed in more detail in section 56.4 below. The next section focuses on the assessment of differential settlement, bending moment and shear force. Initially some of the key soil–structure interaction issues will be discussed together with the pitfalls associated with commonly used 'raft on spring' methods. Finally, a practical approach to raft–soil interaction analysis will be described.

56.2.2 Raft–soil interaction

Several different factors can influence raft differential settlement, bending moments and shear forces, and these include:

(a) the relative soil–raft stiffness, **Figure 56.1** and **56.2**;

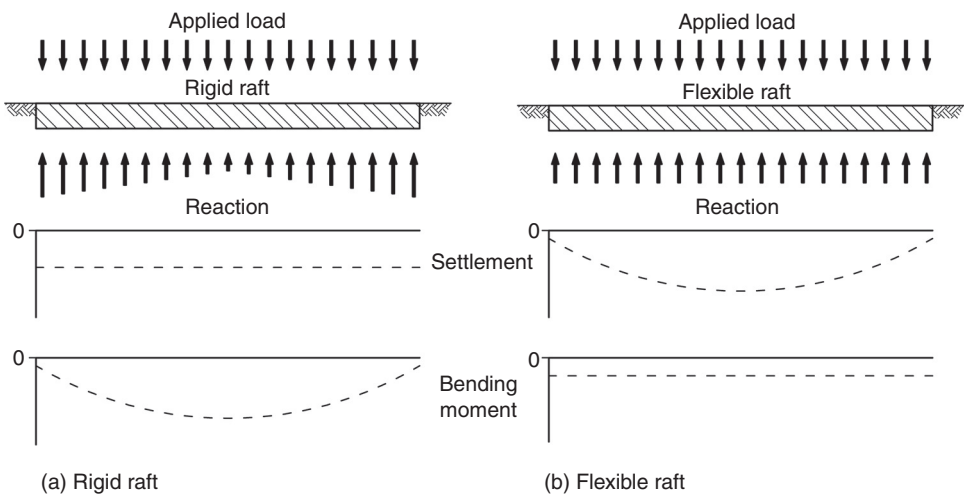
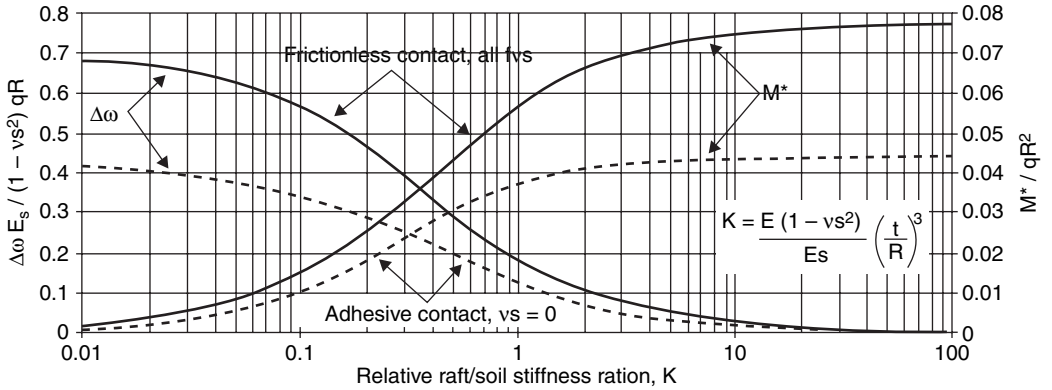


Figure 56.1 Influence of raft stiffness on behaviour

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NB.
"Adhesive" contact means full soil strength mobilised at raft-soil interface, this is not an appropriate assumption for design purposes.
M* = maximum raft bending moment, Δω = diffenrential settlement
E, ν = Young's modulus and Poisson ratio respectiverly, for raft
Es, νs = Young's modulus and Poisson ratio respectively, for soil
t = raft thickness, R = raft radius

Figure 56.2 Maximum differential settlement ($\Delta\omega$) and bending moment (M^*) for a uniformly loaded circular raft on a deep elastic layer
Data taken from Brown (1969) and Hooper (1974)

(b) the influence of the superstructure or sub-structure stiffness on the 'effective' raft stiffness, **Figure 56.3**;

(c) the influence of variations in the soil stiffness and the thickness of the compressible layer;

(d) the influence of soil nonlinearity and local yield, **Figure 56.4**.

Figure 56.1 illustrates the fundamental influence of the raft stiffness on differential settlement and bending moment. For a raft that is practically rigid, **Figure 56.1(a)**, the differential settlement is negligible, but the maximum bending moment is very high. In contrast, for a raft that is practically flexible, **Figure 56.1(b)**, the differential settlement is large, and the bending moment is zero. The stiffness of many real rafts will be between these two extremes, and then the relative soil–raft stiffness and soil–structure interaction needs to be considered.

Figure 56.2 shows the variation of differential settlement and maximum raft bending moment for a uniformly loaded circular raft on a deep elastic soil layer, based on studies by Brown (1969) and Hooper (1974). This indicates that for the relative soil–raft stiffness $K < 0.1$ the raft is effectively 'flexible' (and therefore ineffective in reducing differential settlement) and for $K > 5$ the raft is practically rigid (and therefore differential settlement is negligible). For the zone $0.1 < K < 5$ the relationship between the raft bending moment and differential settlement changes rapidly. The practical value of these sorts of charts is that:

- A simple calculation of K can indicate if the raft is sufficiently stiff to limit differential settlement to acceptable values.
- For rafts at either extreme, very simple models can be used to assess both structural forces and foundation settlement, since the soil–structure interaction has a limited effect.
- For rafts of intermediate stiffness ($0.1 < K < 5$) a more sophisticated method of analysis will be necessary, since the soil–structure interaction will have a dominant effect on raft behaviour.

A general approach to calculating K for rectangular rafts of varying aspect ratios is given in Chapter 53 *Shallow foundations*.

Figure 56.3 illustrates the importance of properly assessing how the sub-structure or superstructure may contribute to the 'effective' stiffness of the raft. The raft is only 0.76 m thick, and would be expected to be flexible given its small t/B ratio (t is raft thickness and B is its width or length). However, agreement between measured and computed differential settlement was only obtained by allowing for the high bending stiffness of the lowermost two storeys of the structure, which comprised stiff continuous shear walls. The actual differential settlement was negligible, but only at the expense of relatively high bending moments, which would be underpredicted if the sub-structure stiffness had been ignored. The influence of the superstructure or sub-structure on raft behaviour will depend on their relative stiffnesses, as discussed by Brown and Yu (1986), Lee and Brown (1972), Fraser and Wardle (1975), amongst others. Hence, as for the case above, the stiffening effect of rigid reinforced-concrete shear walls on the raft is significant; whereas for a flexible light steel-framed structure the effect on raft behaviour will be small. This example illustrates the advantages that shear walls can have in reducing differential settlements of a raft.

In general as the soil stiffness increases, the raft settlement and differential settlement will tend to reduce, and, as a consequence, the raft bending moments will also reduce. Hence, the compressibility of the soil, especially close to the raft base (typically within half the raft width) must be carefully assessed. For wide rafts, the influence of relatively stiff soils (say dense

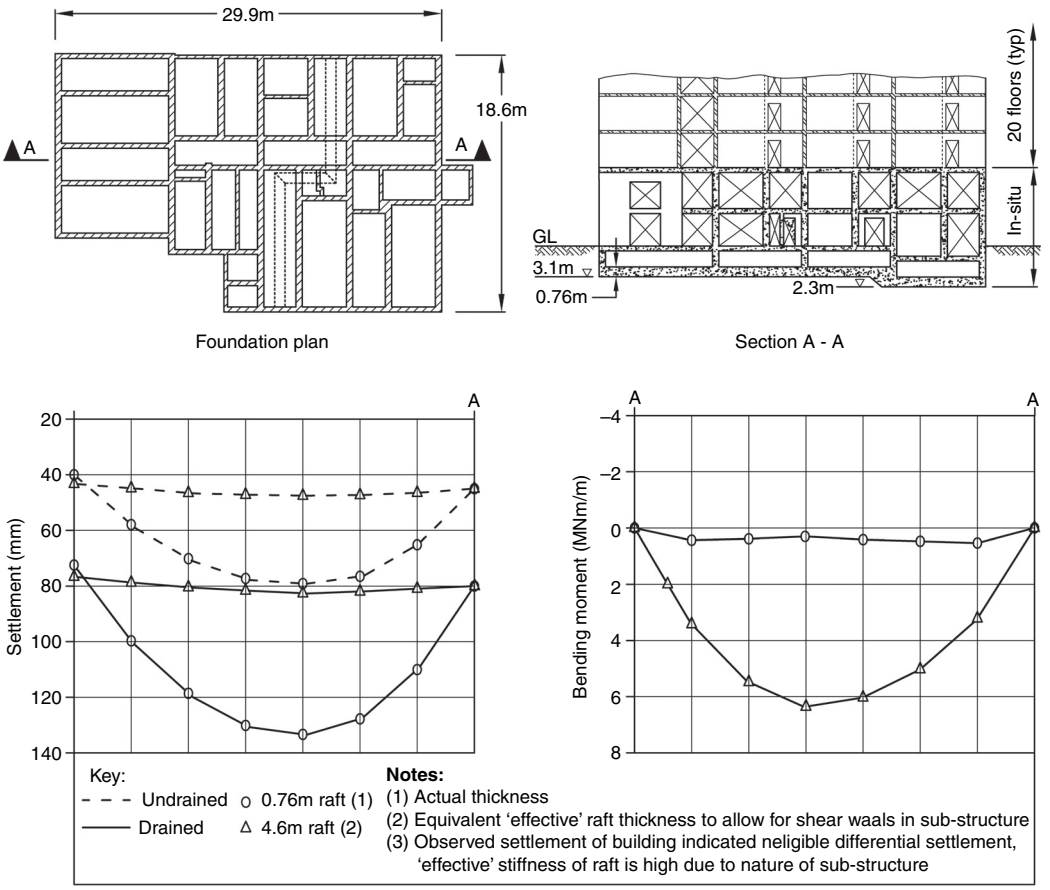


Figure 56.3 Influence of sub-structure stiffness on 'effective' raft stiffness
Modified from Hooper and Wood (1977), all rights reserved

sands or weak rock) at depth can be important (especially if within a depth of one times the raft width). These stiff layers will reduce raft settlement, differential settlement and bending moment. **Figure 56.4** illustrates the important influence of local yield on the distribution of contact stress along the underside of a raft. For an idealised linear elastic soil, the contact stresses are relatively low across the centre of the raft, and then increase towards the edge of the raft to extremely high values of p/q (where p is the local contact stress and q is the vertical pressure applied to the raft). For a real soil (which will fail *locally* once the contact stress fully mobilises the soil strength) the distribution of contact stress differs from the idealised elastic profile. As the applied vertical pressure increases (varying from a pressure equivalent to an undrained factor of safety against bearing-capacity failure of about 10 to a value of 1.1) the contact stress becomes more uniform, particularly for drained conditions. The rule of thumb commonly used by designers of limiting the contact stress to between 50% and 100% above the average applied pressure from the structure appears reasonable.

Referring back to **Figure 56.2**, it can be seen that if it is assumed that the full soil shear strength can be mobilised at the soil–raft interface, then the predicted maximum bending moment is substantially reduced. However, as noted by Hooper (1983) it is usually very unwise to rely on significant raft–soil interface strength for the purposes of structural design. Usually, a frictionless or very low interface strength should be assumed to avoid unsafe provision of raft reinforcement. This should be remembered if numerical models (finite elements, etc.) are used for raft design, and it will be essential to include special interface elements along the underside of the raft model, with appropriately low values of strength specified.

56.2.3 Soil models for analysis: springs vs continuum

Structural engineers usually wish to model the soil as a series of springs, since this allows conventional structural analysis software to be used. Traditionally the soil springs have a uniform stiffness equivalent to the soil's modulus of sub-grade reaction. There are problems with this approach, which are well known in the geotechnical engineering community; for example, the spring stiffness cannot be directly related to measurable soil parameters, since parameters such as the sub-grade reaction moduli are also affected by the foundation width. Also the spring stiffness cannot directly allow for the effects of soil layering. However, the more fundamental problems with 'spring-type'

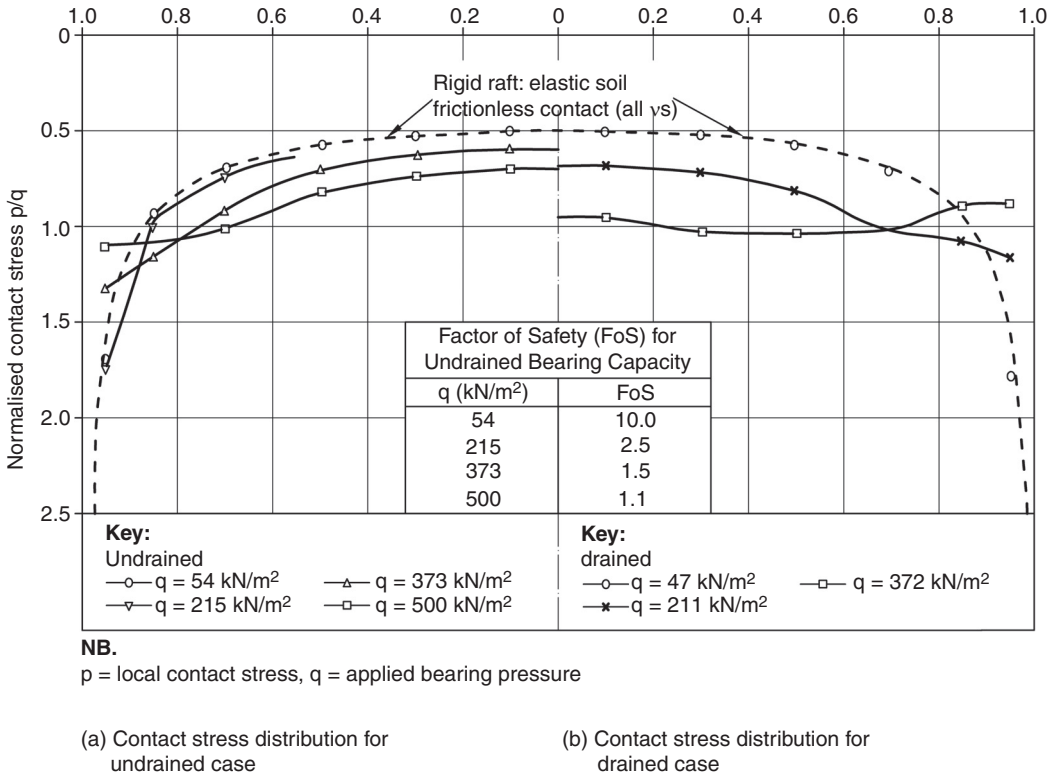


Figure 56.4 Influence of local yield on raft contact stress distribution
Modified from Hooper (1983)

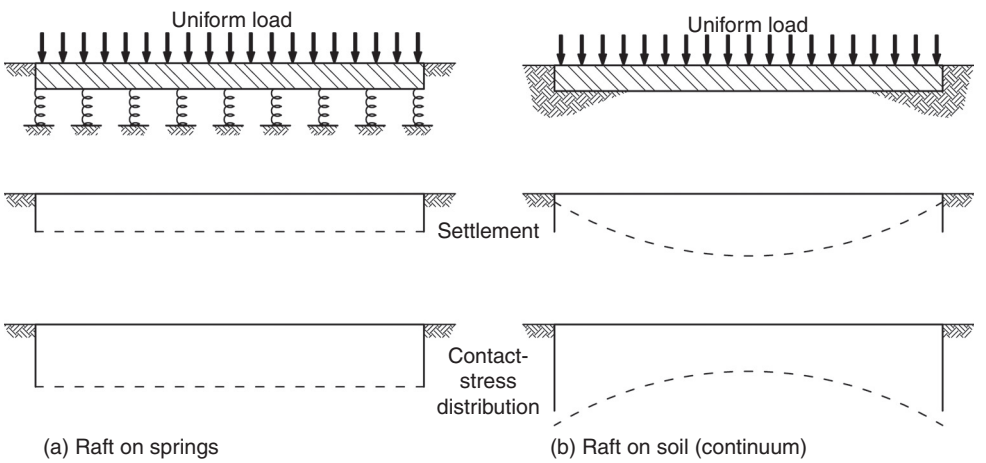
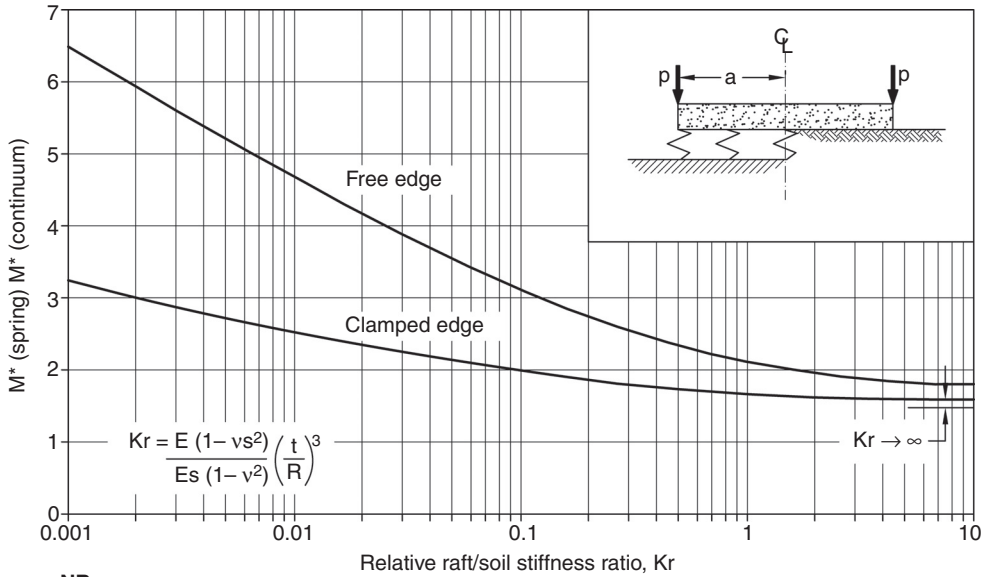


Figure 56.5 Raft on springs compared with raft on soil

models are often less understood. **Figure 56.5** illustrates a fundamental flaw in spring-type models: if a raft of uniform stiffness is subject to uniform loading and is analysed using soil 'springs', then it will settle a uniform amount, **Figure 56.5(a)**. This is inconsistent with real raft behaviour: a raft of finite stiffness will settle more in the centre than at the edges, **Figure 56.5(b)**. For a spring model to generate this curvature, the soil springs at the centre must be less stiff than those at the edges. Hence, appropriate soil spring stiffnesses cannot be uniform and are dependent on a range of factors, which will be dependent on: the raft and the structure, the soil and the distribution of applied loads. The serious errors that can result from a non-interactive approach are illustrated in **Figures 56.6** and **56.7**. For a raft subject to edge loads, **Figure 56.6**, the error



NB.
M* = maximum raft bending moment
E, ν = Young's modulus and Poisson ratio respectively, for raft
Es, νs = young's modulus and Poisson ratio respetively, for soil
t = raft thickness, R = raft radius

Figure 56.6 Ratio of maximum bending moments for an edge-loaded circular raft of varying stiffness (*Kr*), spring vs continuum model
Modified from Hemsley (1987)

associated with a spring model is dependent on the relative soil–raft stiffness and on the rotational stiffness of the edge of the raft (e.g. it may be 'clamped' by a stiff perimeter wall). In this case the spring model is conservative. However, it is not possible to provide any generalised set of 'correction' factors, since spring models can lead to significant overestimates or underestimates of bending moment depending on the magnitude and distribution of the applied load (Hemsley, 1987). This is illustrated in **Figure 56.7**, where the raft differential settlement and bending moments predicted by the spring model are wildly different (and of opposite sign) to the more realistic continuum model.

The key practical conclusion is that the traditional approach of providing sub-grade reaction moduli for 'uniform' soil springs and then running an independent structural analysis can lead to serious errors and should *not* be used for raft design.

56.2.4 Simplified methods for raft–soil interaction analysis

For the simple situation of a rectangular raft that is uniformly loaded, the maximum bending moment and differential settlement can be assessed from charts developed by Horikoshi and Randolph (1997). The estimation of differential settlement is discussed in Chapter 53 *Shallow foundations* (Figure 53.21 and associated text, 53.6.7). Hemsley (1998) also provides charts that are useful for preliminary assessment purposes. Unfortunately, the various published charts are usually too idealised to be sufficient for detailed structural design. Usually the structure-specific loading distribution, spatial arrangement of walls, columns, etc. differs substantially from those assumed in published solutions. Nowadays, it is possible to carry out 3D numerical modelling of soil–raft systems; although it is usually inappropriate (due to complexity, cost, time and lack of necessary specialist input) to implement a highly complex analysis for the majority of structures. A practical approach, which avoids the pitfalls associated with a non-interactive spring-type model (discussed above), is summarised in the flowchart in **Figure 56.8**. This relies upon a structural spring model and a soil continuum model being run in tandem, and both are iterated several times until there is sufficient convergence between the spring stiffness values (say 95% convergence). Depending upon the design phase (preliminary or detailed design), the geotechnical category for the site, the structure, etc., the structural and soil models can be of varying levels of sophistication. The 'interactive spring stiffness', S_i, is defined as:

$$S_i = \frac{\text{Structural spring reaction force}}{\text{local soil continuum displacement at spring location}} \quad (5.1)$$

The soil continuum model assumes the raft is flexible.

The iterative analysis is usually started by estimating the spring stiffness from the overall raft settlement (calculated by conventional soil mechanics methods, Chapter 53 *Shallow*

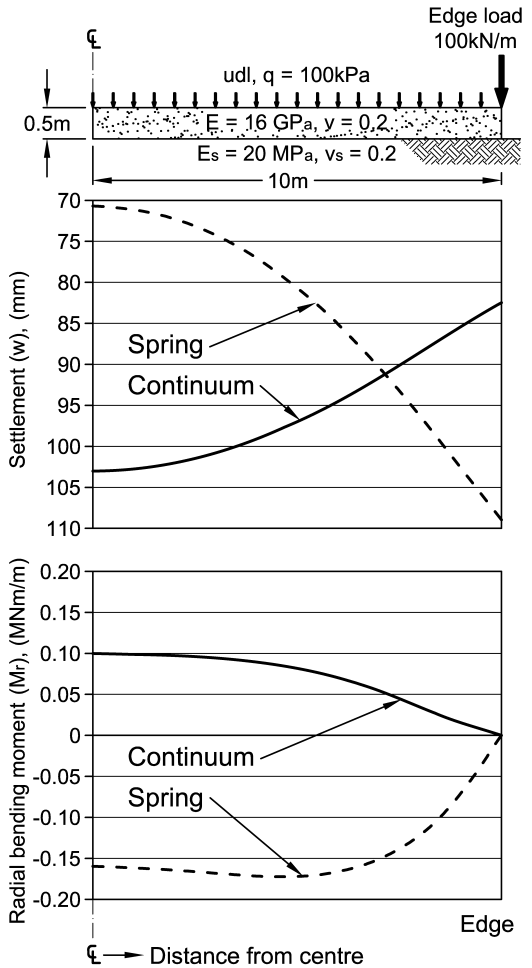
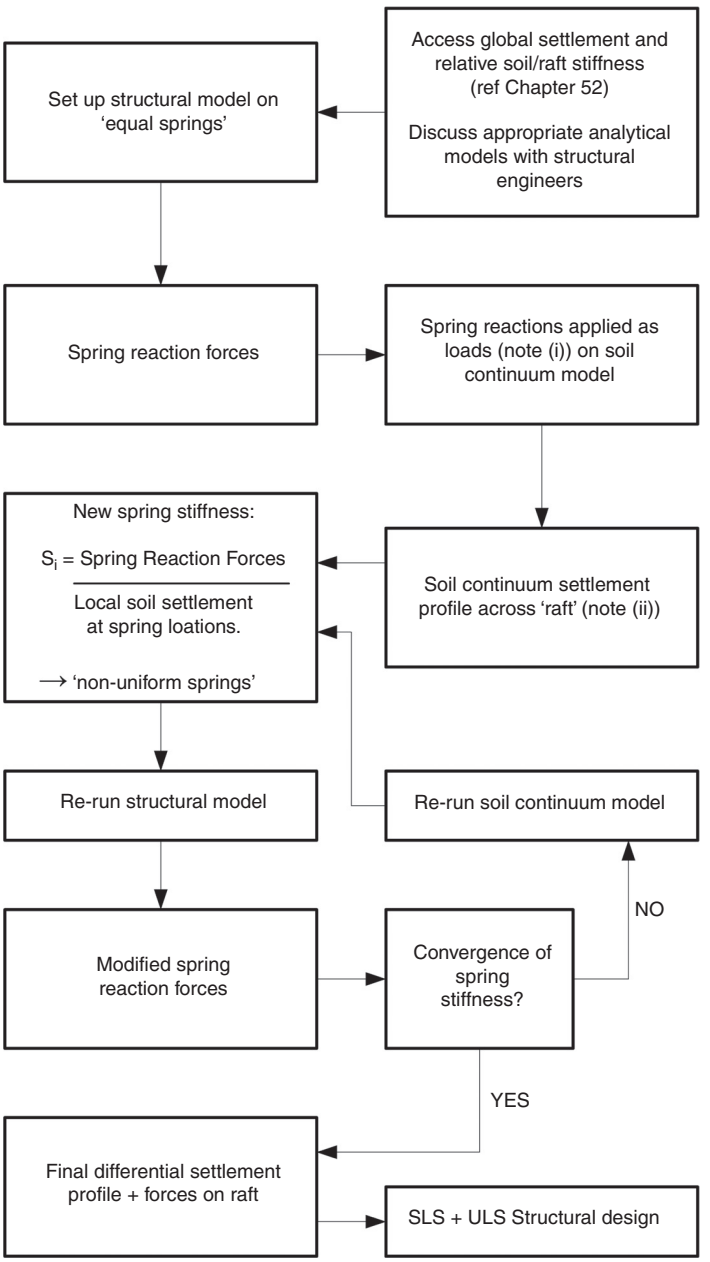


Figure 56.7 Settlement and bending moment for circular raft subject to uniform pressure and edge load, continuum model versus spring model
Data taken from Hemsley (1987)



Notes
(i) Raft assumed "flexible" in soil continuum.
(ii) Consider gross and "net" forces for short-term and long-term settlement respectively, refer to text.

Figure 56.8 Flowchart for raft–soil–structure interaction analyses

foundations). These spring stiffnesses are then used in a structural model (with appropriate raft and sub-structure stiffnesses, applied loading for columns, walls, etc.). The spring reaction forces are then applied as loads in the soil continuum model, with appropriate characterisation of the soil behaviour (stiffness, layering, etc.). This will provide a non-uniform settlement profile along the 'raft' base, which is then used to recalculate the new spring stiffnesses of Equation 5.1. These values are re-input into the structural model and re-run. This process is repeated until the deflections from the soil continuum and structural models are similar at all spring locations. The stiffness of the soil springs will usually vary significantly beneath the raft, typically with softer springs in the centre and stiffer springs around the edge of the raft.

Some care is required when the raft is located at the base of an excavation, and there is a significant difference between the gross and net bearing pressures (refer to Chapter 52 *Foundation types and conceptual design principles* for definitions). The structural analysis will require gross pressures; however, the long-term soil settlement will be dependent on net bearing pressures (based on the changes in effective stress). For rafts on clay, gross bearing pressures would be used for undrained settlement and net bearing pressures for drained (total) settlement (or swelling, if the net effective bearing pressure is

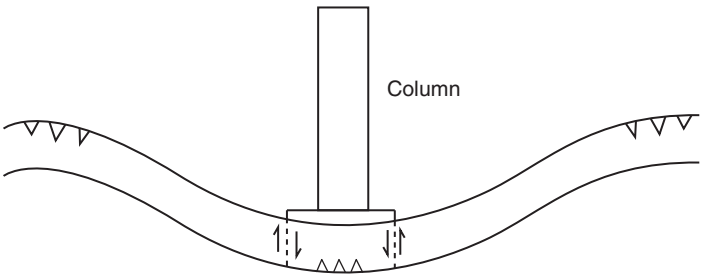


Figure 56.9 Modes of failure of rafts

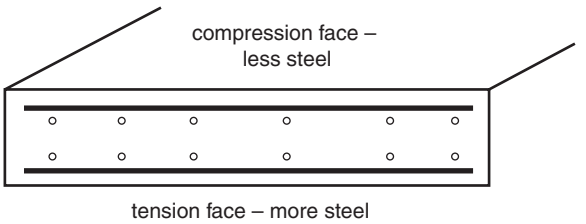


Figure 56.10 Typical bending of steel in a raft designed for sagging

negative, allowing for overburden removal and any changes in the water table level).

56.3 Structural design of rafts

As for all foundations, rafts must remain structurally integral and not deflect excessively and cause unacceptable angular distortions in the structure above. There are two main modes of potential structural failure:

(i) bending;

(ii) shear.

These are illustrated in **Figure 56.9**.

While unreinforced concrete has some capacity to resist both these sets of forces, generally steel reinforcement is used to augment capacity and provide adequate resistance. In both cases, structural codes (such as BS8110 or Eurocode 2) provide detailed guidance on how to design reinforced concrete. The purpose of this part of the chapter is to outline basic principles.

Simple beam theory states:

$$\frac{f}{y} = \frac{E}{R} = \frac{M}{I} \tag{5.2}$$

where f is the peak stress in the extreme fibre of the section at distance y from the neutral axis; E is Young's modulus for the concrete and R is the induced radius of curvature (note that for small curvatures, the radius of curvature is high); M is the corresponding applied moment in the concrete section and I is the moment of inertia for the section, which is (1/12) bd^3 for a rectangular section, where d is the depth and b is the breadth, equal to 1 m per metre run.

The equation is often simplified to

$$M = EI / R \tag{5.3}$$

so that the product of the moment and the radius of the induced curvature is equal to the section's flexural rigidity EI, and from this it can be deduced for any section that the bending moment and radius of curvature are inversely related. Hence adding more steel, so that the section can resist greater moments, will reduce the radius of curvature, i.e. it will increase the actual curvature and hence differential settlements. However, a stiff section may attract load, so the benefit of continually adding more steel may reduce.

Excessive bending in a structural element also causes cracks in the extreme tension fibres of the concrete, which can reduce durability. The significance of cracks in structural elements that are exposed to saturated soil with no free oxygen is a topic of debate, but structural codes provide firm guidance.

Bending resistance is provided by steel on the tension face, and sometimes also by steel on the compression face too. Thus a raft often includes two distinct layers of steel, in two perpendicular directions. This is shown in **Figure 56.10**. For lightly reinforced rafts, mesh reinforcement is sometimes used.

A critical consequence of bending is the induced curvatures, which lead to differential settlements between different parts of the raft. A key issue in raft design is to control these differential settlements so that the supported structure above remains fully functional, i.e. angular distortions in the structure are acceptable, finishes are undamaged and the building function isn't impaired by excessive tilt.

Shear reinforcement in a concrete raft is needed when either a column or a pile may 'punch' through the element, as shown in **Figure 56.11(a)**. If a base plate is added beneath the column, the plane of potential rupture is forced further out and a greater perimeter of concrete is mobilised to resist rupture, see **Figure 56.11(b)**. **Figure 56.12** shows a simplified shear stress example. The shear force down is 5000 kN - 200 kN/m^2 × 1 m^2 = 4800 kN and the area of concrete resistance is 1.2 × 1 × 4 = 4.8 m^2. Therefore, the shear stress is 4800 kN / 4.8 m^2 = 1000 kN/m^2 = 1 N/mm^2. Since unreinforced concrete can resist a shear stress of up to 1 N/mm², normally no special shear reinforcement is needed. For higher induced shear stresses, extra reinforcement may be needed to resist these shear stresses.

In the case shown in **Figure 56.12**, the induced shear stress in the concrete v_c is 1 N/mm², which is at the limit where shear reinforcement might usually be needed. As shear reinforcement is difficult to fabricate, other options to avoid or limit its use might include:

- widening the column base to extend the shear perimeter;
- thickening the raft, although the extra excavation and concrete will be expensive, and in a basement may necessitate also a deeper retaining wall;

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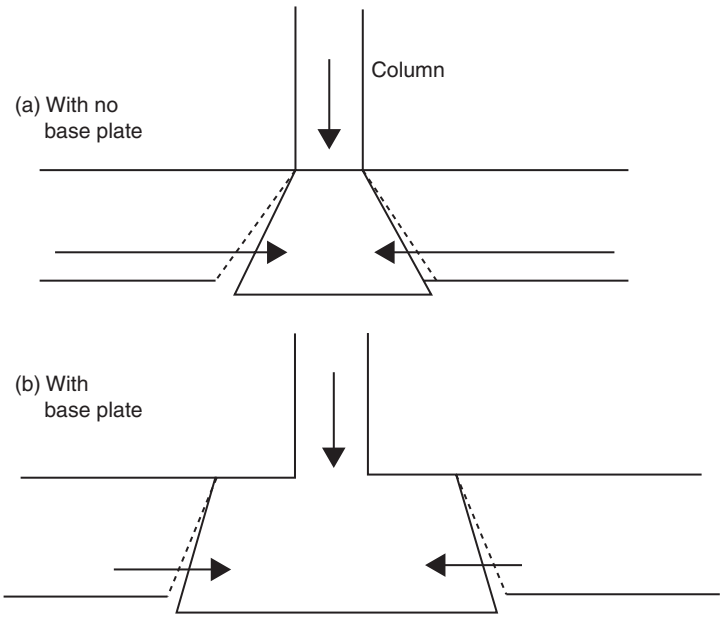


Figure 56.11 Punching of a column through a raft slab

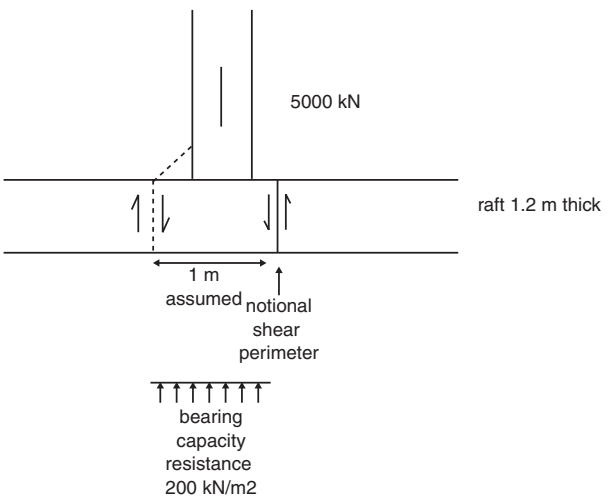


Figure 56.12 Simplified shear stress example

- adding a 'shear-reducing' pile beneath the column location to increase the soil resistance locally and hence reduce the net load that needs to be shed out into the rest of the raft.

56.4 Design of a real raft

56.4.1 Bracketing by load cases

Raft design needs to accommodate a wide range of design cases and the induced forces and deflections are very sensitive to those load cases. The design cases include loads from the structure and loads and reactions from the ground. Loads from the structure range from a minimum of the dead load only to a maximum of the full dead and live loads. The ground loads include water pressures and heave pressures (which can be maxima or minima) from under a basement and a range of ground reactions based on upper and lower bound ground stiffnesses. These various combinations give a wide range of curvatures, often with sagging or hogging at the same location for different cases, and hence the steel needed in upper and lower reinforcement layers can originate from apparently odd load combinations. An example of a set of load combinations for a raft design is given in **Table 56.1**.

56.4.2 Design steps

Raft design, almost more than the design of any other geotechnical element, depends on mutual comprehension between geotechnical and structural engineers. As the raft design process is more complex, often piles are chosen ahead of rafts, even though the raft design may be more efficient in terms of cost, ease of construction, programme, risk reduction and even carbon saved.

The design complexity depends on:

- the magnitude of loads, connected to the building height and column grid layout;
- whether there is a basement, and if there will be water or heave pressures;
- the number of load cases that should be considered;
- the magnitude of variation in loads, particularly between adjacent columns;
- whether the ground is relatively soft;
- the drive for extreme economy in the raft.

The complexity of analysis needed crudely depends on the compatibility between the structural and the geotechnical analyses. A typical process is shown in **Figure 56.13**. Where the two sets of analyses yield relatively close agreement at first iteration, then a less rigorous design process can be followed.

As stated earlier, the structural analyses usually assume the ground reaction can be modelled by simple springs while the geotechnical analyses model the ground response following Boussinesq or similar principles that more accurately model settlement interactions between adjacent loaded areas, such as the OASYS PDISP analysis program. The errors associated with spring models are described in section 56.2.3 above. Geotechnical analyses are usually seriously limited in their ability to model structural behaviour.

Sometimes a sufficiently accurate simplification can be achieved by performing a second structural iteration, refining the springs to better approximate the likely soil displacements. Otherwise a longer series of iterations between structural and geotechnical analyses will yield a better approximation, as discussed in 56.2.4 and shown in **Figure 56.8**. The structural analysis is initially run to give a load disposition over a number of loaded areas, those loads are then fed into a geotechnical

Basic load cases (1)	Additional load cases (4)
ULS	Wind and SLS
1.4A1 + 1.4A2 + 1.6A3 + 1.4A4 + 1.4A5 + 1.6A6	DL + SDL + IL + Water tank + Earth + Surcharge
	DL + SDL + IL + Water tank + Earth + Surcharge + Wind (X)
	DL + SDL + IL + Water tank + Earth + Surcharge + Wind (–X)
	DL + SDL + IL + Water tank + Earth + Surcharge + Wind (Y)
	DL + SDL + IL + Water tank + Earth + Surcharge + Wind (–Y) found to be most critical
SLS	SLS – DL [2]
A1 + A2 + A3 + A4 + A5 + A6	SDL + IL + Water tank + Earth + Surcharge
	SDL + IL + Water tank + Earth + Surcharge + Wind (X)
	SDL + IL + Water tank + Earth + Surcharge + Wind (–X)
	SDL + IL + Water tank + Earth + Surcharge + Wind (Y)
	SDL + IL + Water tank + Earth + Surcharge + Wind (–Y) found to be most critical
	Critical wind [3]
	SDL + IL + Water tank (1,2) + Earth + Surcharge + Wind (–Y)
	SDL + IL + Water tank (1,3) + Earth + Surcharge + Wind (–Y)
	SDL + IL + Water tank (2,3) + Earth + Surcharge + Wind (–Y)
	SDL + IL + Water tank (3,4) + Earth + Surcharge + Wind (–Y)

Notes:
1 A1: Dead load (DL); A2: superimposed dead load (SDL); A3: imposed load (IL); A4: water tank (e.g. the primary treatment works building has a few tanks and different combinations of fully loaded tanks were considered in the differential settlement check); A5: earth pressure; A6: surcharge (from soil above the roof); A7: wind X; A8: wind –X; A9: wind Y; A10: wind –Y.
2 DL is omitted for settlement and differential settlement checks on the assumption that the proposed utilities and pipeline connections will be installed after the DL has taken effect.
3 Based on the –Y wind direction being the most critical for the different tank load combination effects on the utilities and pipeline connections.
4 The load cases were for a sewerage treatment facility.

Table 56.1 Example of a set of load cases for a raft design

analysis, which gives a new disposition of settlements taking into account adjacent effects; those settlements are used to refine the springs assumed in the structural analysis and it is again re-run. The process is continued until a reasonable state of convergence is achieved between the structural and geotechnical analyses.

Finite element (FE) techniques can be used to give even more precise results. An FE approach combines the structural and geotechnical modelling into a single analysis. But even 2D FE modelling can require too much simplification as both building layouts and settlements are very 3D by their nature, so often a 3D analysis is required. And once 3D analyses are applied to a wide range of load combinations, the whole design process can become very time consuming and expensive.

In addition to these interactions, crude checks are needed at an early stage for global and local bearing-capacity failure. It is rare to find a raft that could exhibit a kinematically valid bearing-capacity failure mode, as the more likely modes are excessive displacement or a structural failure, but this relatively rapid check remains important as an indicator of potential design problems.

Whenever there are significant water loads, a check for net buoyancy should also be made for the critical load stages, which could include a future partial demolition of the building above.

56.4.3 Factor of safety considerations

Usually a factor of safety calculation is less critical in raft design than in other types of geotechnical design, as usually movements (serviceability) are more critical than ultimate failure considerations. As for other aspects of design, it is easy to muddle geotechnical and structural safety factors and design codes. Both British Standards and Eurocodes offer little guidance. Hence, careful thought is needed to identify and resolve these potential inconsistencies. Generally as the magnitude of the resisting forces is very large, a lumped factor of safety of 2 is assumed to be sufficient margin against bearing-capacity failure.

Simplistically for a pure raft, the structural analysis considers the deflected shape and external forces acting on the element while the geotechnical analysis considers movement and the global bearing-capacity failure of the whole raft foundation. Local bearing-capacity calculations can sometimes be relevant, but are usually obviated as the structural check against failure prevents a local failure of a discrete part of the raft from being kinematically admissible.

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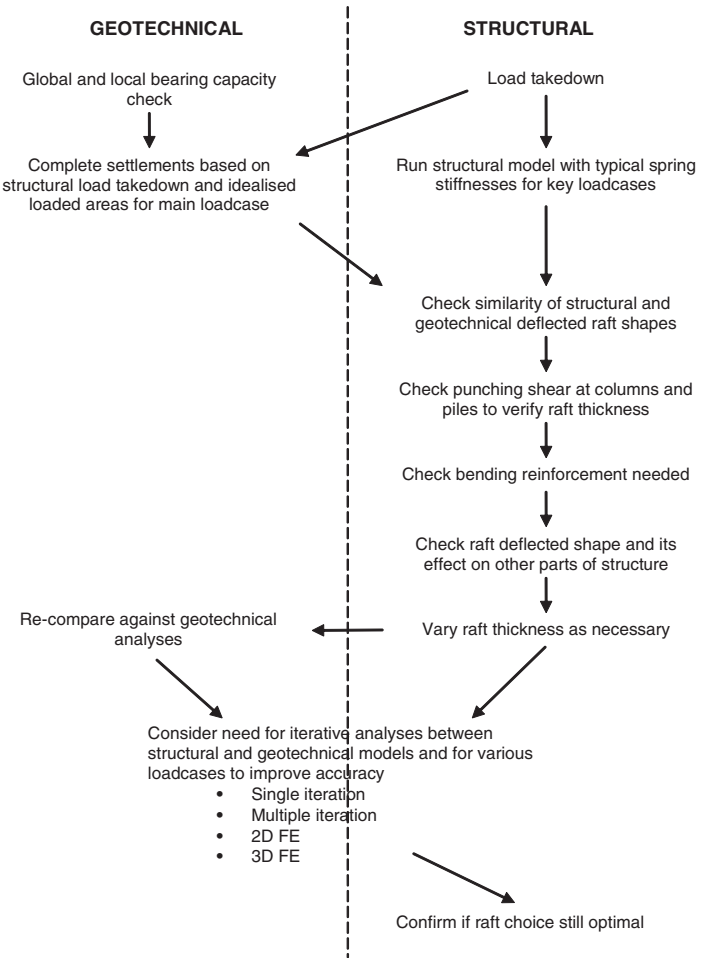


Figure 56.13 Simplified raft design process

56.4.4 Buildability considerations for raft construction

The first set of buildability choices relate to the raft's formation level, which corresponds to the chosen raft thickness. The formation layer should ideally be dry, well drained and above the water table. Often it will be necessary to thin a raft to bring its formation level above a likely water table level. From a structural perspective, it is expedient to thicken a raft beneath columns to resist the increased shear forces, but such stepped formations are difficult and potentially less safe to build.

Steel is relatively expensive and slow to place in cages, especially significant quantities of shear steel. There is always a trade-off between a thicker raft with less steel and a thinner raft with more steel. If a raft has too much steel it can be difficult to compact the concrete and so the raft may be more prone to defects, long-term durability and water-penetration issues.

If the ground contains some contamination or there is a risk that it may, a thinner raft needing less excavation may be advantageous. Ground gas penetration can be an issue, especially at joints, for instance with a retaining wall or at any penetrations through the raft.

If the design calls for an under-slab drainage layer to reduce the design water pressures, the layer needs to be considered carefully. Consideration of price alone suggests gravel-filled grip drains, while broader buildability and construction-safety considerations will probably result in a no-fines concrete drainage layer that can also act as a blinding layer. Care must be taken to ensure that the no-fines concrete has high permeability, rather than just porosity, and will not be prone to clogging. Often a roddable pipe network is included as a precaution, connected to manhole covers that can lift if water pressures exceed the design levels.

56.5 Piled rafts, conceptual design principles

56.5.1 Different types of piled raft and key definitions

Piled rafts have significant benefits when used in suitable ground conditions:

(i) costs can be significantly reduced (the number of piles, the raft thickness and reinforcement quantities can all be reduced);

(ii) construction can be simplified and the time for foundation construction can be reduced.

Associated with (i) and (ii) there may also be safety and sustainability benefits. A number of different terms have been used to describe piled rafts and the associated design philosophies. For the purpose of this chapter, the following terms will be used:

(a) *Raft-enhanced pile group*: both the piles and raft will work within a pseudo-elastic range of behaviour. The pile-group capacity will *not* be fully mobilised at the anticipated working load (based on the expected raft or pile load share). The key parameter that governs behaviour is the relative stiffness of the pile group to the raft. It will be important to assess the upper and lower bound stiffnesses for the raft and pile group. Although the piles will usually be stiffer than the raft and attract most of the design load, the raft can be designed to resist a substantial proportion of the design load. The pile and raft stiffness will be a function of the stiffness of the ground, and the variation of ground stiffness with depth.

(b) *Pile-enhanced raft:* the piles will be designed to mobilise all their ultimate capacity. The raft will usually carry the bulk of the design load. The piles will usually be located beneath heavily loaded superstructure columns. For this type of design it will be important to assess both the lower and upper bound pile capacity with a high level of confidence, and the pile load-settlement behaviour *must* be *ductile*, i.e. the pile resistance must be maintained at relatively large settlements (*circa* 50 to 100 mm).

These different modes of behaviour are illustrated in **Figure 56.14**, which shows a load-settlement curve for a raft

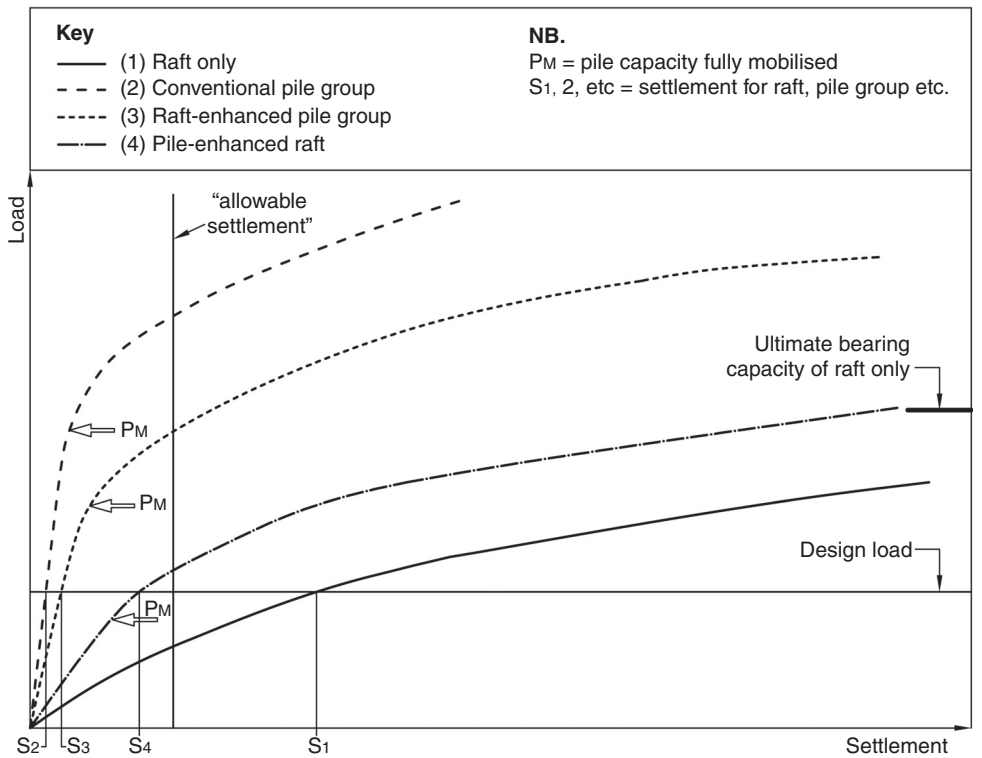


Figure 56.14 Load-settlement behaviour of rafts, conventional pile groups and different types of piled raft

(curve 1), a conventionally designed pile group (curve 2), a raft-enhanced pile group (curve 3) and a pile-enhanced raft (curve 4). The raft has a substantial ultimate bearing capacity, but the settlement at working load is deemed to be excessive (as noted in Chapter 52 *Foundation types and conceptual design principles*, the magnitude of 'allowable' settlement needs to be carefully scrutinised). The conventionally designed pile group is very stiff and its settlement at the working load is far smaller than the 'allowable' settlement (as noted in Chapter 55 *Pile-group design*, conventional analysis of pile groups often significantly overpredicts pile-group settlement). The raft-enhanced pile group (curve 3) is also relatively stiff and this is achieved with a smaller number of piles than the conventionally designed pile group, because the piles are operating more efficiently. The pile-enhanced raft (curve 4) exhibits more settlement than the conventional pile group and raft-enhanced pile group, but is stiffer than the raft and can achieve the allowable settlement for this structure. The load-settlement curve for the pile-enhanced raft is within the nonlinear range. It is apparent that if the allowable settlement is set at too low a value, then the opportunity to use piled rafts, especially 'pile-enhanced rafts', will usually be lost. Hence, defining a realistic value of allowable settlement is important; Chapter 52 *Foundation types and conceptual design principles* (section 3) discusses this topic in more detail. The term 'settlement-reducing piles' was used by Burland *et al*. (1977) to describe the concept of a pile-enhanced raft, and they stated that:

> *Traditionally engineers engaged in pile group design have asked themselves 'How many piles are required to carry the weight of the building?' When settlement is the conditioning factor in the choice of piles designers should perhaps be asking the question: 'How many piles are required to reduce the settlements to an acceptable amount?' The number of piles in answer to the second question is invariably significantly less than in answer to the first question, provided it is accepted that the load-carrying capacity of each pile will probably be fully mobilised.*

A clear distinction must be drawn between raft-enhanced pile groups and pile-enhanced rafts. The design concepts and methods are very different. As shown in **Figure 56.15***, there is an intermediate zone between a raft-enhanced pile group and a pile-enhanced raft, which should be avoided*. If it isn't avoided, with the wide range of load cases, it is entirely possible that one column will feel a soft response while a neighbouring column may experience a stiff response, setting up large differential displacements in the building.

In a raft-enhanced pile group or pile-enhanced raft, the two sets of elements need to be considered together, with the sum of both components of capacity being useful in resisting the imposed loading. However, the detail of the consideration is

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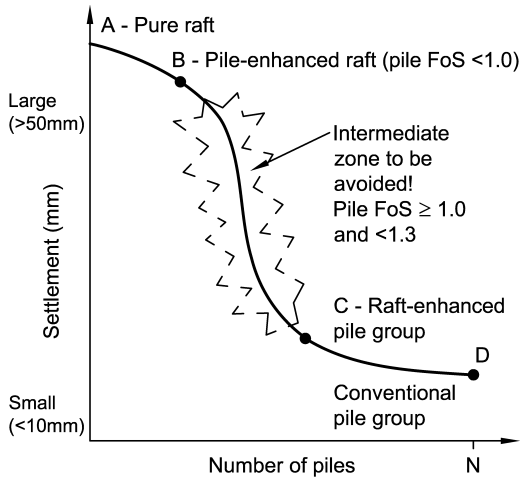


Figure 56.15 Pile-enhanced rafts versus raft-enhanced pile groups (avoid intermediate zone)

different in philosophy, and clarity is needed about what each component is meant to contribute. These issues are discussed in sections 56.6 and 56.7 below.

For a pile-enhanced raft, consideration of the ultimate global geotechnical capacity isn't usually an issue, and the piles are provided to reduce peaks of either raft displacement or shear stress in the raft. For these cases it is not usually relevant to separately add the piles' capacity component into the overall geotechnical bearing-capacity calculation.

For raft-enhanced pile groups, however, the main advantage of considering both is that the extra raft component of capacity allows fewer piles to provide the same overall margin against failure. Clearly the ultimate bearing stress used for the raft's component must be compatible with the pile's performance, and so may need to be reduced according to compatibility of the deformation between the two; see Equation 5.4 below.

Some additional terms are used in the context of raft and piled-raft design, and these include:

(i) *Compensated raft*: refer to Figure E1.1(d), a raft designed as a 'deep box', with the void within the raft reducing the net increase in bearing pressure.

(ii) *Compensated piled raft*: refer to **Figure 56.16**, this type of foundation is situated within a deep excavation, and because of the large reduction in overburden stress, this type of piled raft has additional design considerations (discussed in section 56.6 below) beyond those of a piled raft close to the ground surface. Heave-induced tension is likely to be an issue for pile design and raft/pile load sharing, and Chapter 57 *Global ground movements and their effects on piles* should be referred to.

(iii) *Creep-piling*: the term 'creep piles' has been used to describe the behaviour of pile-enhanced rafts, founded on soft lightly overconsolidated clays, where sufficient

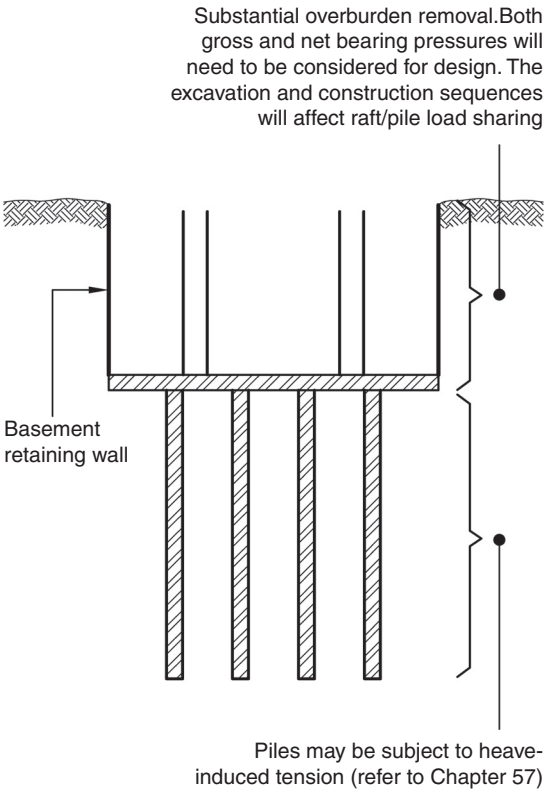
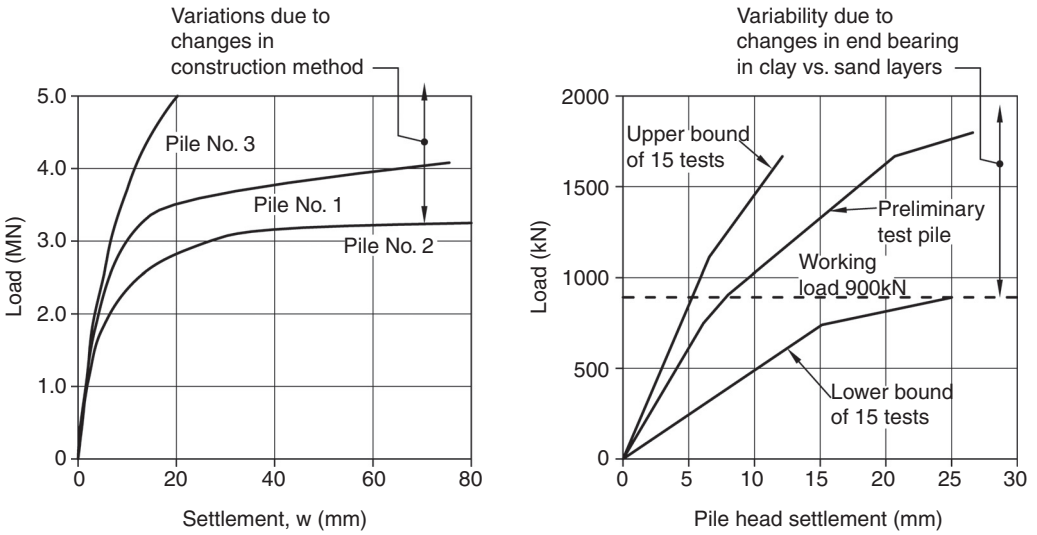


Figure 56.16 Compensated piled raft

piles are included to reduce the net bearing pressure on the raft to a value which is less than the soft clay's pre-consolidation pressure or yield stress (refer to Chapter 52 *Foundation types and conceptual design principles*, and Figure E1.24(b)). This is a highly specialist application of piled-raft design. It requires a high level of expertise to safely implement and will not be discussed in detail in this chapter. Although this is a special application of pile-enhanced rafts, it is not new and Zeevaert (1957) described the use of a compensated raft with friction piles in the compressible soft volcanic clays of Mexico City.

56.5.2 Appropriate pile types and ground conditions for piled rafts

The potential variations in pile load-settlement (stiffness) behaviour are important for raft-enhanced pile groups, whereas the variability in pile ultimate capacity is a key consideration for pile-enhanced rafts. Chapter 54 *Single piles* discusses the fundamental differences between friction piles (shaft capacity >> end bearing) and end-bearing piles. Variations in ultimate pile capacity and pile stiffness are sometimes observed at an individual site; these are usually due to variations in pile construction methods (or workmanship problems) or to variations in soil properties across the site. Some examples are shown in **Figure 56.17**. **Figure 56.17(a)** illustrates the behaviour of CFA piles in alluvial soils, and it can be seen that the stiffness



(a) CFA piles in alluvial soils (after Mandolin *et al.*, 2005)

(b) Driven piles in glacial till (after Corke *et al.*, 2001)

Figure 56.17 Examples of load-settlement curves for CFA and driven piles

is reasonably similar despite wide variations in ultimate capacity (due to changes in construction method). However, **Figure 56.17(b)** illustrates the wide variation in pile stiffness and capacity of driven piles in a soil, which varies between sandy clay and clayey sand. The variation is mainly due to the large changes in end-bearing resistance when the piles are toed into 'sand' rather than 'clay'. If piles are operating at shaft factors of safety of between 1.0 and 1.3 then pile resistance, at a particular raft deformation, may be extremely variable, which could lead to excessive raft differential settlement. Many rafts are designed for a wide range of load cases and so there is a high risk of excessive differential settlement between adjacent columns. **Figure 56.18** indicates that where the *shaft* factor of safety is more than about 1.2 to 1.3, the pile settlement becomes much smaller and more consistent. Hence, it is not the overall factor of safety (i.e. shaft plus base), but the *shaft* factor of safety which mainly controls its stiffness. For end-bearing piles, both their stiffness and ultimate capacity tends to be more variable than for friction piles, because end-bearing behaviour is more sensitive to local variations in ground conditions and construction method than shaft behaviour. Therefore, before deciding upon the use of a raft-enhanced pile group or a pile-enhanced raft it is important to consider the type of piles that can be used, the nature of the ground conditions beneath the site and the potential variations in pile capacity (especially for pile-enhanced rafts) and stiffness together with the methods that can be used to minimise these variations. A raft-enhanced pile group will typically be more tolerant of variations in ground conditions and pile behaviour; however, a pile-enhanced raft may often be more economical than a raft-enhanced pile group.

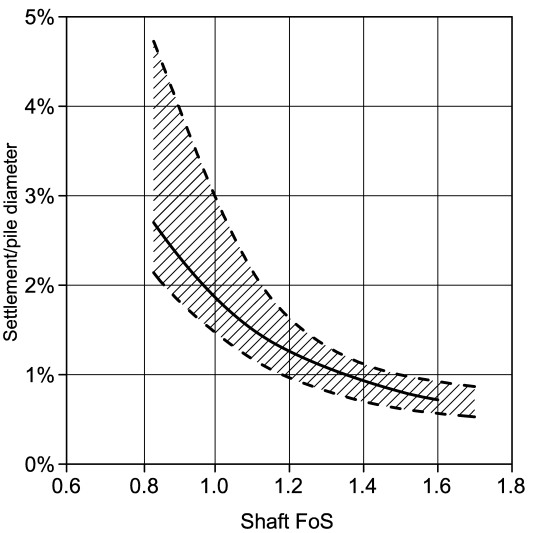
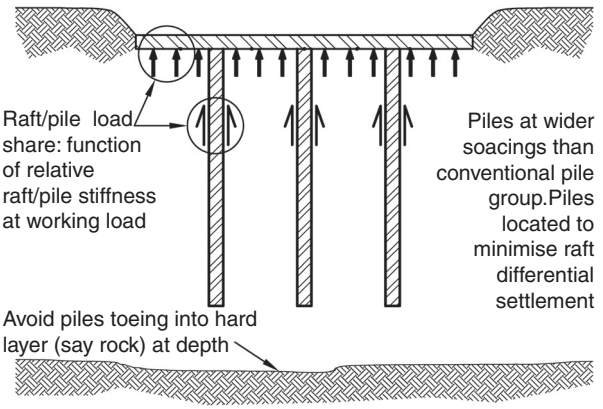


Figure 56.18 Pile settlement versus factor of safety on shaft capacity
Modified from Corke *et al.* (2000), all rights reserved

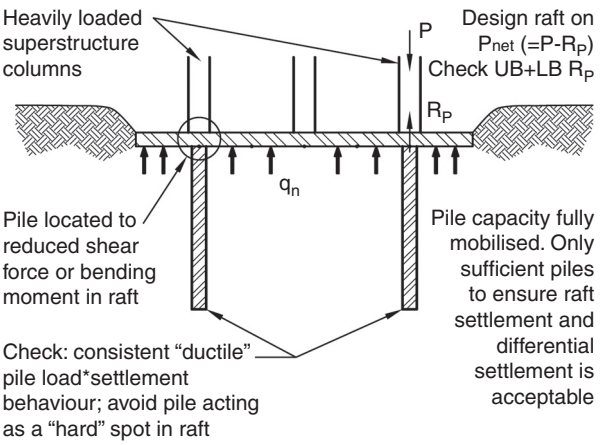
Figure 56.19 illustrates a raft-enhanced pile group and a pile-enhanced raft, together with guidance on the soil profiles that would be most appropriate for their application. A raft-enhanced pile group can be used in a broad range of ground conditions, provided the soils beneath the raft are competent enough (i.e. stiff clays or dense sands) to ensure that the raft can carry a reliable proportion of the design load throughout the structure's design life. Both friction piles and end-bearing piles can be used. For a pile-enhanced raft relatively uniform ground conditions are desirable, and typically would comprise



Favourable ground conditions
Comptenet soils at raft level (stiff clays, dense sands) and depth. Interbedded sands/clays OK.

Typical design scenario
A value-engineered pile group, often the raft was provided as a pile cap, but is then used to provide significant capacity.

(a) Raft-enhanced pile group



NB.
qn = net contact stress on underside of raft
P = column laod, Rp + pile capacity
UB = upperbound, LB = lower bound

Favourable ground conditions
Deep deposits of homogeneous clays. Typically stiff clays at raft level.

Typical design scenario
A raft which doesn't quite work, a small number of piles are added to resolve local non-compliances (eg. differential settlemeny, shear or bending moment)

(b) Pile-enhanced raft

Figure 56.19 Raft-enhanced pile group versus pile-enhanced raft

relatively deep deposits of stiff homogeneous clays. In these ground conditions, the ultimate capacity of friction piles (and the lower to upper bound range) should be fairly consistent across a site (provided a good standard of workmanship during pile construction is maintained) and their load-settlement behaviour should be ductile. As noted on **Figure 56.19**, the starting point for raft-enhanced pile groups and pile-enhanced rafts will often be very different. The former is usually a large pile group, which is value-engineered to make use of the pile cap for resisting the applied loads. In contrast, the latter is a raft that doesn't meet all the design criteria, so a few piles are used to resolve the non-compliance. **Table 56.2** provides a summary of some piled rafts that have been reported in the technical literature. It is worth noting the wide range of piled-raft geometries, ground conditions and observed settlements. The long-term performance of piled rafts has been satisfactory, and their use has resulted in significant economies compared with conventional foundation design.

There are some ground conditions that are unfavourable for the use of piled rafts and these include:

(i) Soil profiles containing heterogeneous non-engineered fill, soft clays or loose sands close to the underside of the raft, which are underlain by more competent soils at depth in which the piles will be located. In this case, the raft will not provide significant stiffness or bearing capacity compared with the piles. The piles should then be designed conventionally.

Reference / structure	Ground conditions	Pile spacing (s/*d*)	Pile length (m)	Raft width (m)	Pile-group area raft area	Settlement (mm)	Raft load (%)
Urawa Building (Yamashita *et al.*, 1994)	Interbedded sands and clays	8	15.8	23	0.9	15	51
Stonebridge Park Building (Cooke *et al.*, 1981)	London Clay	3.6	13	21	0.95	12	23
Carigliano Bridge (Russo and Viggiani, 1995)	Interbedded clays or sands and gravels	3.0	50	10.6	0.88	52	20
MesseTurm (Sommer *et al.*, 1991)	Frankfurt Clay	3.5 to 6.0	27 to 35	59	0.85	140	45
Japan Centre (Katzenbach *et al.*, 2000)	Frankfurt Clay	5.5	22	43	0.45	60	60
DG Bank (Katzenbach *et al.*, 2000)	Frankfurt Clay	4 to 6	30	54	0.52	110	50
Hyde Park Barracks (Hooper, 1979)	London Clay	4.3	25		0.72	20 to 25	40
QE2 Conference Centre [1] (Burland and Kalra, 1986)	London Clay	N/A	16	33.5	N/A	16 to 20	>90
Napoli Tanks (Mandolini *et al.*, 2005)	Medium dense sand	5 and 5.8	13.7 and 9.7	12.5 and 10.5	0.82	10 to 15 and 20 to 25	46 and 50
Canary Wharf South (Nicholson *et al.*, 2002)	Stiff clay (raft level), very dense sand (pile toe level)	4	25.5	25	0.9	45	16

Note
1 The sole purpose of the piles was to reduce the shear stresses and bending moments in the raft, beneath a few heavily loaded columns.

Table 56.2 A selection of piled-raft case histories

(ii) Site locations and soil profiles that may be vulnerable to significant settlement due to external causes. Chapters 9 *Foundation design decisions*, 52 *Foundation types and conceptual design principles* and 53 *Shallow foundations* describe a number of situations where significant ground movements may develop independently of structural loads. These must be carefully assessed before piled rafts are used.

There is a particular risk with (i) and (ii) above that the ground support provided to the raft will be lost or seriously reduced over time. The key issues and questions that need to be considered by designers are summarised in **Table 56.3**.

56.6 Raft-enhanced pile groups

56.6.1 Design process

Poulos (2001) outlined the main steps in piled-raft design:

(i) 'Conceptual' design: assess the feasibility of using a piled raft and the required number of piles to broadly satisfy the design requirements (settlement and bearing capacity).

(ii) 'Preliminary' design: assess where the piles are required, the pile characteristics (length, diameter, etc.) and the differential settlement of the raft.

(iii) 'Detailed' design: finalise the optimum number, location and configuration of the piles, and calculate the detailed distribution of settlement, the bending moment, the shear force in the raft, pile loads, moments, etc.

Stages (i) and (ii) should be based on an appraisal of the ground conditions, the type of pile which will be suitable and on relatively simple calculations (outlined below). The detailed design stage will require appropriate analytical software, which can account for the interactions between piles, raft, soil and superstructure. This can vary from a modified version of the iterative and interactive spring approach outlined above for assessing raft behaviour through to 3D nonlinear numerical modelling. Although, even when sophisticated analytical methods are used, there would normally be two separate models: one focusing mainly on structural behaviour and the other focusing mainly on soil behaviour.

In the conceptual design stage it is essential to consider first:

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Ground conditions	Are the overall geology and hydrogeology well known and understood? Is there competent ground (stiff clay or medium dense or dense sand) at raft level?
Global ground movement	Are significant ground movements likely to develop adjacent to the raft in the future? Could these lead to a loss of support to the raft?
Raft bearing capacity and settlement	Would the raft have an adequate bearing capacity (say factor of safety in excess of 2) under anticipated loads? What is the likely upper and lower bound settlement?
Allowable settlement and differential settlement	Is the raft settlement acceptable? Can the allowable settlement be increased?
Pile type	Given the nature of site, what types of piles are appropriate: friction or end-bearing? Is there potential variability in pile stiffness and capacity? Is the behaviour ductile?
Piled raft	If the ground is competent at raft level, could a piled raft be used if the raft settlement is excessive, or if the bending moment or shear force in the raft is excessive? What type of piled raft is appropriate: a raft-enhanced pile group or a pile-enhanced raft?
Analysis requirements and design competency	Are appropriate analytical tools available? Are specialist staff available to deliver design?
Additional field tests	Are additional ground investigations required (e.g. to verify ground conditions at the raft or ground stiffness characteristics)? What additional pile tests are required?

Table 56.3 Piled rafts – key questions to consider

(a) raft behaviour without the piles (bearing capacity, settlement and differential settlement);

(b) pile-group behaviour without the raft (ultimate capacity, settlement and differential settlement).

For (a) and (b) relatively simple methods (described in Chapters 53 *Shallow foundations* and 55 *Pile-group design*) are appropriate. If the ultimate capacity of the raft on its own is a small fraction of the required ultimate capacity, or would be adversely affected by global ground movements that are independent of the applied structural loads, then a conventional pile-group design would usually be appropriate. However, if the raft can be founded on reasonably competent ground, which can provide reliable long-term resistance, then the raft can be used to significantly reduce the piling requirements that are required to achieve the design criteria (ultimate capacity, settlement, etc.).

The next steps are to assess the ultimate capacity and settlement of the combined piled-raft system. A critical part of this process is to assess load sharing between the raft and piles across a range of different loads.

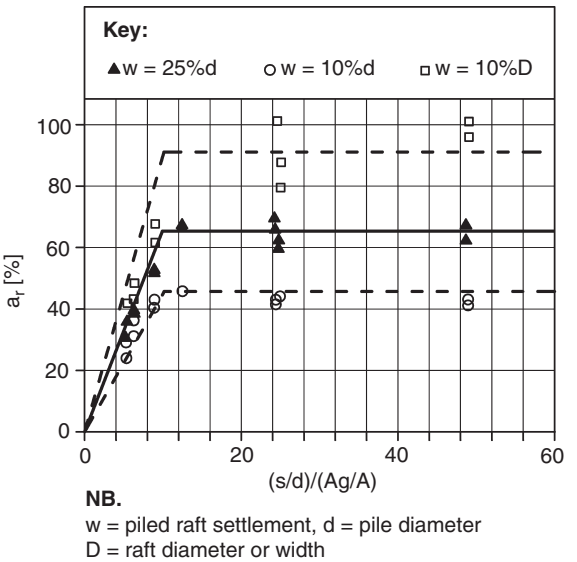


Figure 56.20 Pile raft ultimate capacity factor, α_r, versus pile spacing and area
Modified from Mandolini *et al.* (2005), all rights reserved

56.6.2 Simple models for load sharing and piled-raft behaviour

Ultimate capacity: the vertical bearing capacity can be taken as the lesser of:

(i) the capacity of the block containing the piles and raft, plus a portion of the raft beyond the pile-group perimeter;

(ii) the factored ultimate capacity of the raft plus the piles, as indicated in Equation 5.4 below.

Mandolini *et al.* (2005) indicate that pile-group block failure is likely at small values of pile spacing, below S_{crit}, with S_{crit}/d varying between 2.5 (for 3x3 pile groups) and 3.5 (for 9x9 pile groups), where d is the pile diameter. For pile spacings larger than S_{crit} then the piled-raft ultimate geotechnical capacity, Q_{PR}, can be taken as the lesser of

$$Q_{PR} = \alpha_R Q_R + \alpha_P Q_P \qquad (5.4)(a)$$

where α_R varies as shown in **Figure 56.20** and $\alpha_P = 1.0$. It is clear from **Figure 56.20** that the value of α_R is dependent on the definition of failure (i.e. the magnitude of the piled-raft settlement at 'failure'). A limiting value for α_R of 0.65 for a piled-raft settlement equal to 25% of d is generally appropriate for this bearing-capacity check, although at typical pile spacings and group geometries α_R is between 0.3 and 0.5; or

$$Q_{PR} = f_{PR}(Q_R + Q_P) \qquad (5.4)(b)$$

where $f_{PR} = 0.8$ (based on Mandolini *et al.*, 2005).

The lesser of Equations 5.4(a) or 5.4(b) should be taken as the ultimate geotechnical capacity of the foundation and used for ULS checks. The ultimate geotechnical capacity of individual components of the foundation (i.e. raft, pile group or individual piles) does not need to be code compliant.

Note that Q_R is the ultimate capacity of the raft alone and Q_P is the ultimate capacity of the pile group.

Load sharing at working loads: based on observations of load sharing for 22 piled rafts (mainly for piled rafts in London Clay and Frankfurt Clay), Mandolini *et al*. (2005) provide a useful plot of raft load (%) vs the dimensionless parameter $(s/d)/(A_g/A)$, with s and d the pile spacing and diameter respectively, A_g is the area of the pile group (plan area of pile-group 'block', i.e. not the sum of individual pile cross-sections) and A is the area of the raft; this empirical relationship is shown in **Figure 56.21**. Typically, the raft takes between 20% and 50% of the overall load for raft-enhanced pile groups; some caution is required if the raft load share is estimated to be more than 50%.

Based on elastic theory, Randolph (1994) has shown that the proportion of the load carried by a raft is

$$\frac{P_r}{P_t} = \frac{P_r}{P_r + P_p} = \frac{\left(1-\alpha_{rp}\right)K_r}{K_p + \left(1-2\alpha_{rp}\right)K_r} \qquad (5.5)$$

where P_r is the load carried by the raft, P_p is the load carried by the piles, P_t is the total load, K_r is the stiffness of the raft, K_p is the stiffness of the pile group, K is the stiffness (load divided by settlement) and α_{rp} is the interaction factor between the raft and piles.

The stiffness of the raft and the pile group can be assessed by the methods described in Chapters 53 *Shallow foundations* and 55 *Pile-group design*, respectively. If the pile spacing is greater than 8d then the pile to pile interaction will be negligible and the pile-group stiffness will be the sum of the individual pile stiffnesses, as assessed by the methods described in Chapter 54 *Single piles*.

It should be noted that the load share between raft and piles will change over time, due to the influence of:

(i) Changes in ground stiffness due to the transition from undrained to drained conditions for clay soils and the influence of creep in sandy soils. Typically, this will lead to the load carried by the raft reducing with time.

(ii) Changes in the groundwater pressure over time. This is particularly important for compensated piled rafts where the buoyancy force is typically a significant proportion of the total load so that the raft carries a greater proportion of the total load (e.g. Sales *et al.,* 2010).

Hence, analyses of pile and raft load sharing will need to take account of the changes in ground stiffness between short- and long-term conditions and, particularly for compensated piled rafts, changes in applied loads and the buoyancy force with time. For example, at Stonebridge Park the raft initially carried 40% of the total load during construction and this reduced to less than 25% in the long term as the London Clay consolidated. For situations where the proportions of the load carried by raft and pile components varies over time, each must be designed for their greater percentage of load, and care must be taken that the combined solution doesn't end up more expensive than a pure piled solution or a pure raft.

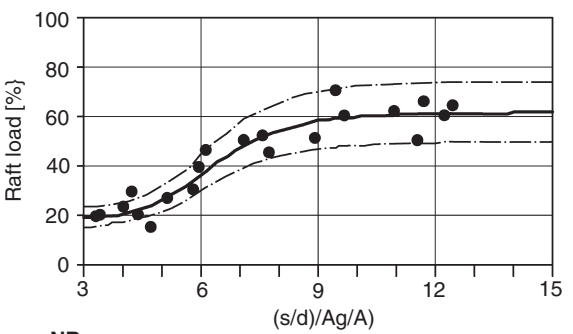


Figure 56.21 Observed load sharing between raft and piles, 22 case histories
Modified from Mandolini *et al.* (2005), all rights reserved

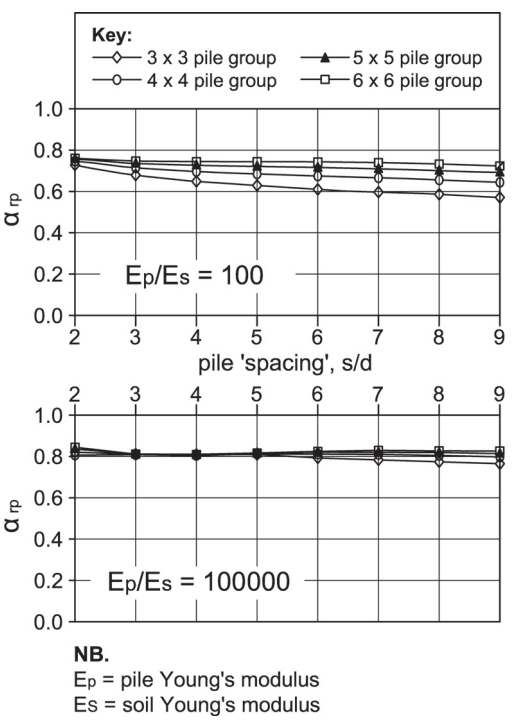


Figure 56.22 Pile–raft interaction factor, (α_{rp}), versus pile spacing and pile-group size
Data taken from Clancy and Randolph (1993)

Figure 56.22 summarises the variations of α_{rp} for varying pile spacing, pile-group size and soil–pile stiffnesses. It can be seen that α_{rp} only varies across a small range of values, between 0.65 to 0.8 (Randolph, 1994).

Poulos (2001) suggested a simplified tri-linear load-settlement curve for preliminary analysis, shown in **Figure 56.23**. For simple preliminary calculations it could be assumed that the raft and pile-group behaviour is linear–elastic, in which case the stiffness of the piled raft, K_{pr}, is

$$K_{pr} = [K_p + (1–2\alpha_{rp})K_r] / [1 – (\alpha_{rp})^2 (K_r/K_p)] \qquad (5.6)$$

The pile-group capacity is fully mobilised at a total applied load of P_1 (**Figure 56.23**), given by

$$P_1 = (Q_P) / (1\text{-}P_r/P_t) \qquad (5.7)$$

where P_r/P_t is calculated using Equation 5.5 and Q_P is the ultimate capacity of the pile group.

For loads beyond P_1 the stiffness of the piled raft is simply equal to the raft alone (K_r), until the ultimate capacity of the piled raft (from Equation 5.4) is reached. The relative stiffness between the raft and pile group controls the load sharing, the overall stiffness of the raft-enhanced pile group and the magnitude of P_1, as shown on **Figure 56.23**. The plausible upper and lower bound stiffnesses of the raft, K_r, and the pile group, K_p, need to be carefully assessed for application in Equations 5.5 and 5.6 (see **Figure 56.24**).

It is strongly recommended that this simplified overall load-settlement response is calculated for any piled raft. At later design stages the component behaviour of the raft and piles can be computed by assuming nonlinear elastic behaviour. A raft-enhanced pile group should be operating within the initial, relatively stiff, part of the tri-linear load-settlement curve (Figure 56.23) with the design load ≤ P_2, whereas a pile-enhanced raft will be operating in the 'softer' second part of the load-settlement curve with the design load > P_3. For the plausible range of applied loads, it is important that the raft-enhanced pile group does not traverse across P_1 on **Figure 56.23**. To reduce the risk of traversing across P_1, a mobilisation factor should be applied to the ultimate pile-group capacity, Q_P. For friction piles a factor of about 0.75, and for end-bearing piles a factor of about 0.6, would be appropriate. The mobilisation factor should not be confused with a factor of safety: it is simply a pragmatic means of ensuring that the raft or pile load sharing (and associated structural forces) is constrained within reasonably predictable bounds. Large conventionally designed pile groups typically have overall factors of safety of well in excess of 2.5 to 3.0. Assuming a raft carried about 30% of the overall load (a fairly typical value) then the above mobilisation factors would typically allow the number of piles to be reduced to about a half to two-thirds of a conventionally designed pile group.

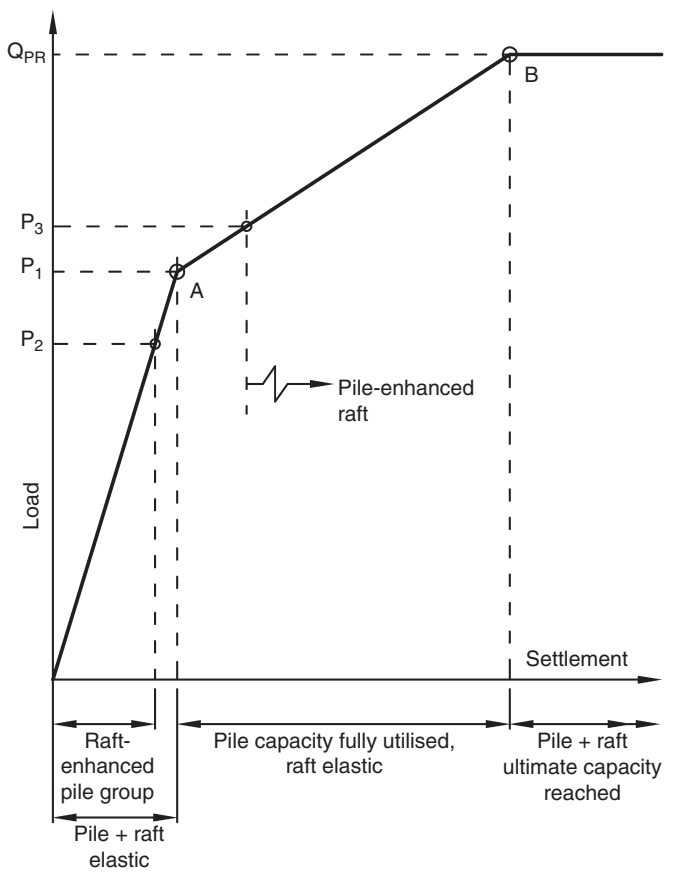


Figure 56.23 Simplified load-settlement curve for a piled raft

56.6.3 Pile–raft interaction

Parametric studies by Randolph (1994), Poulos (2001) and Katzenbach *et al.* (1998) have provided useful insights into piled-raft behaviour, and some key aspects of behaviour are summarised in **Table 56.4**. In the general case of a piled raft, very complex interactions take place between the various elements. These are summarised in **Figure 56.25**, which shows a piled raft with non-uniform applied loading and the raft in

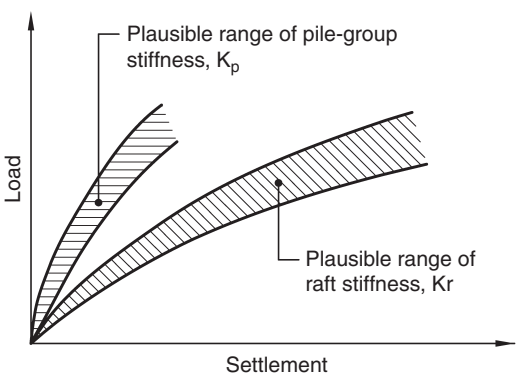


Figure 56.24 Raft-enhanced pile groups. Key design check against relative raft and pile stiffnesses

Design issues	Comment
Number of piles	Increasing the number of piles will be of limited benefit, once a threshold number is exceeded
Pile location	Typically, for optimum effectiveness focus on the area beneath the most heavily loaded columns and across the central half of the raft
Pile length and spacing	A smaller number of longer piles will be more effective than a large number of short piles. Keep the pile spacing as wide as possible, typically in excess of 4*d*
Raft thickness	A thicker raft will reduce differential settlement, at the expense of higher bending moments. Raft thickness has a negligible effect on load sharing or maximum settlement
Distribution of applied loads	Important for assessment of differential settlement and raft bending moment, negligible effect on maximum settlement or raft/pile load sharing
Soil–structure interaction analysis	Most methods tend to overpredict the proportion of load carried by the piles, and therefore underpredict the load carried by the raft. Hence, care is needed in the structural design of a raft. Sophisticated 2D analyses are likely to be less accurate than approximate methods that allow for the 3D nature of problem

Table 56.4 Piled rafts – some factors influencing behaviour

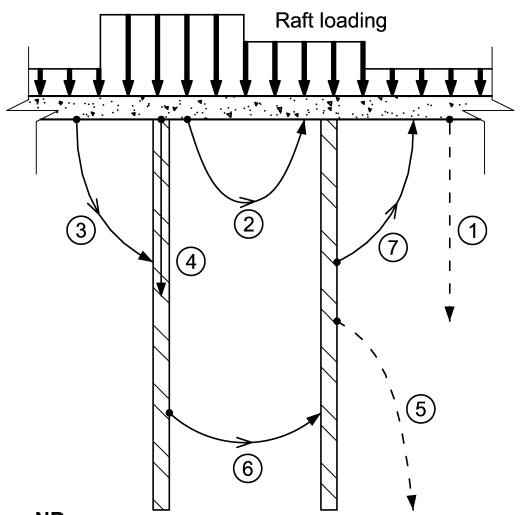


Figure 56.25 Interactions between different components of a piled raft

contact with the underlying soil. The figure shows the following interactions:

1. *Raft–soil interaction:* The contact stresses between the raft and the soil are transmitted into the soil and settlement of the raft takes place.
2. *Raft–soil–raft interaction:* Interaction takes place through the soil with other parts of the raft.
3. *Raft–soil–pile interaction:* The raft contact stresses are also transmitted through the soil and interact with the piles.
4. *Raft–pile interaction*: Loads are transmitted into the piles directly by the raft.
5. *Pile–soil interaction*: The pile loads disperse into the ground surrounding the piles.
6. *Pile–soil–pile interaction*: Interaction takes place between each pile through the soil to other piles.
7. *Pile–soil–raft interaction*: Interaction also takes place between each pile through the soil to the underside of the raft.

A comprehensive analysis of the behaviour of a raft-enhanced pile group would require sophisticated 3D numerical modelling, which would need to include:

- A nonlinear stress–strain model for the soil: as noted in Chapter 55 *Pile-group design* the modelling of small-strain nonlinearity can lead to a more economic pile-group design, because interaction effects between piles are more realistically simulated (and will be lower than predicted by a linear elastic-plastic soil model). Similarly, interaction effects between piles and raft will also be more realistically predicted.
- Appropriate modelling of the sub-structure stiffness: as discussed in section 56.2 above, the 'effective' stiffness of a raft is often relatively high, due to the influence of the superstructure.
- Interface elements between soil–raft and soil–pile elements: appropriate strength and stiffness characteristics need to be selected otherwise erroneous and potentially unsafe predictions of structural forces can be predicted (for example, refer to **Figure 56.2**).

The above requirements are challenging and require a significant input by appropriately qualified specialists. Careful calibration of these complex models is extremely important and should include:

(i) checking that the load-settlement characteristics of the raft only and a single pile are realistically simulated;

(ii) checking that the load-settlement characteristics of the pile group only (without a contribution from the raft) is realistic;

(iii) back analysis of a relevant case history for a similar raft-enhanced pile group.

An important observation from the studies reported by Poulos (2001) is that most soil–structure interaction analyses (which usually assume linear elastic or linear elastic-perfectly plastic soil behaviour beneath the raft) tend to underestimate the load carried by the raft at working loads. This is probably due to the influence of a nonlinear small-strain stiffness, which will modify the soil–structure interaction behaviour at working

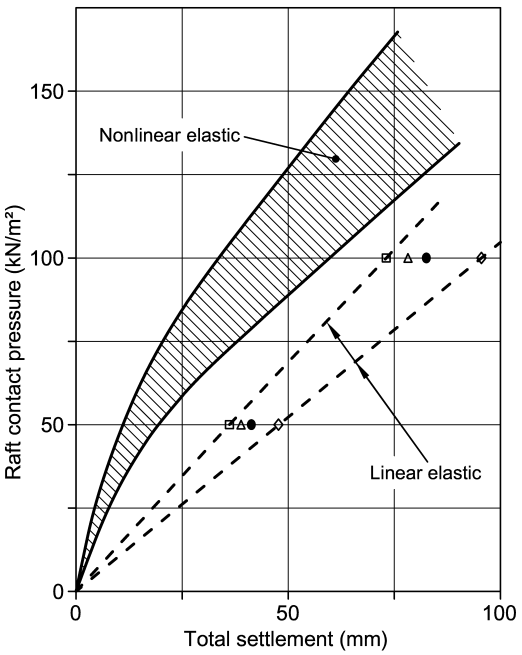


Figure 56.26 Influence of small-strain nonlinearity on raft settlement
Modified from O'Brien and Sharp (2001), Emap

loads and lead to a stiffer initial load-settlement response for the raft (Jardine *et al.,* 1986). Hence, the raft stiffness, K_r in Equation 5.5, will often be higher than estimated from conventional linear-elastic methods and soil-stiffness correlations. As discussed by O'Brien and Sharp (2001) a modified 1D method (refer to Chapter 53 *Shallow foundations*) provides a simple and practical approach to assessing the influence of nonlinearity, **Figure 56.26**. This shows that at moderate settlement, typical of raft-enhanced pile groups (25 mm to 50 mm), the raft stiffness indicated by the nonlinear method is nearly double that of linear elastic methods, whereas similar stiffnesses are indicated at large settlements (because the empirically derived stiffnesses usually used in linear elastic methods are derived from rafts that exhibited relatively large settlements and the derived moduli are consistent with 'large strain' soil stiffnesses).

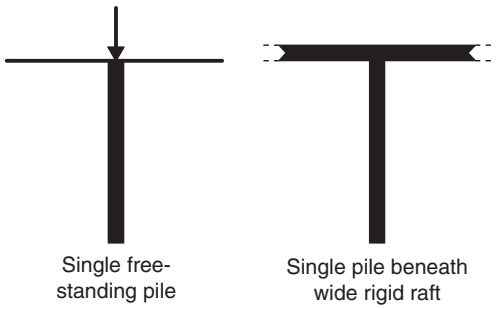
Pile–soil–pile interaction will occur when the piles are spaced at less than about 6*d* to 8*d* (*d* is the pile diameter), which will usually be the case for raft-enhanced pile groups. Hence, the piles below the raft will effectively act as a group. Pile–soil–pile interaction effects are discussed in Chapter 55 *Pile-group design*. The piles around the perimeter of the pile group will be stiffer, and attract more load, than the piles in the centre of the group. Therefore, for an efficiently designed pile group, especially within a raft-enhanced pile group, it is inevitable that some piles around the perimeter will mobilised close to their ultimate capacity. In turn, this means that nonlinear methods will need to be used for detailed design (nonlinear effects are discussed in Chapter 55 *Pile-group design*). Nevertheless, as noted above, the ultimate capacity of the pile group should not be fully mobilised within a raft-enhanced pile group. As noted below, this will simplify analysis and design.

Studies by Burland (1995) and Katzenbach *et al*. (2000) indicate how the raft–pile interaction modifies the behaviour of piles below a raft, compared with 'conventional' piles (i.e. without a raft in contact with competent soil) and how the contact stress beneath a raft is modified by the piles.

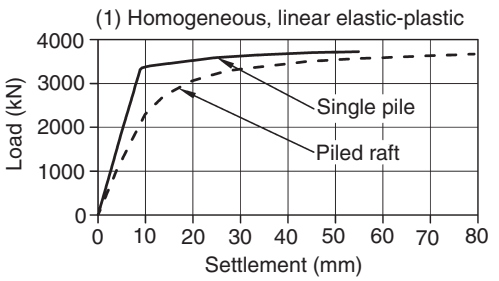
Burland (1995) describes a preliminary numerical study (later extended) of the behaviour of a single free-standing pile, 1 m in diameter and 20 m long, loaded to full mobilisation of the shaft compared with the behaviour of an identical pile beneath a very wide rigid raft foundation, which is loaded such that the shaft resistance of the pile is fully mobilised, **Figure 56.27(a)**. Three types of soil were studied (1) homogeneous, linear elastic-perfectly plastic, (2) increasing stiffness with depth, linear elastic-perfectly plastic and (3) nonlinear elastic-perfectly plastic. Cases (2) and (3) are representative of most realistic situations. Fully drained conditions were assumed and the maximum shaft friction was specified as 50 kPa, the equivalent of half the undrained strength ($\alpha = 0.5$). **Figure 56.27(b)** shows the load/settlement behaviour of the free-standing single pile compared with that of the pile beneath the raft for each of the above cases. It can be seen that for cases (2) and (3) the load carried by the pile mobilises a little less rapidly than for the equivalent free-standing pile. **Figure 56.27(c)** shows a graph of the normalised settlement against depth beneath the raft and the relative displacement (between pile and soil) of a rigid pile beneath the raft. Full mobilisation of shaft friction requires a finite relative displacement between the soil and pile (typically between about 5 mm and 10 mm). For a pile beneath a raft, the relative displacement varies from zero at the top of the pile to a maximum at the pile toe. As raft settlement increases, shaft friction is mobilised at the pile toe and progressively moves upwards with increasing raft settlement. The rate at which shaft friction is mobilised will depend significantly on the variation of soil stiffness with depth. When soil stiffness increases rapidly with depth (a common occurrence in practice) the shaft friction will then be mobilised more rapidly than for the case when stiffness is constant with depth. However, there will always be a small region close to the pile head where the shaft friction is not fully mobilised.

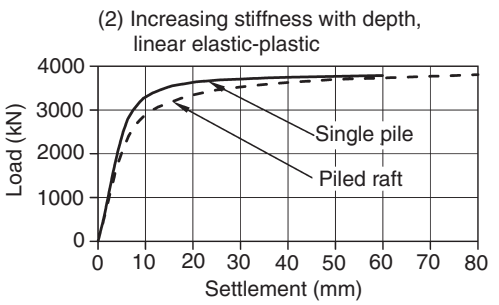
Figure 56.28 is for the case (2) soil conditions and shows the distribution of the vertical shear stress around a single free-standing pile and the corresponding pile beneath a raft at a settlement of 50 mm. It is evident that the differences between the two piles are small but the stresses are slightly more concentrated around the pile beneath the raft. At a radius of about 6 times the diameter of the pile the vertical shear stresses have reduced to less than about 5% of the shaft friction. It is worth noting that near the top of the pile beneath the raft, the shaft resistance is not fully mobilised due to the raft–soil–pile interaction.

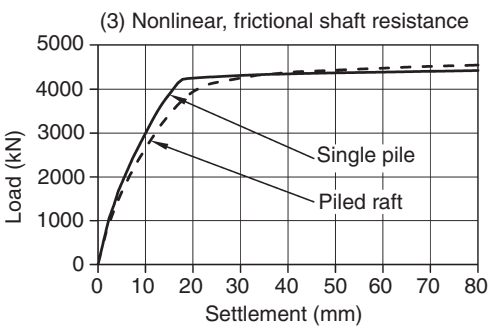
The pile beneath a raft also influences the raft behaviour. Figure 56.29, from Katzenbach *et al*. (2000), shows that the



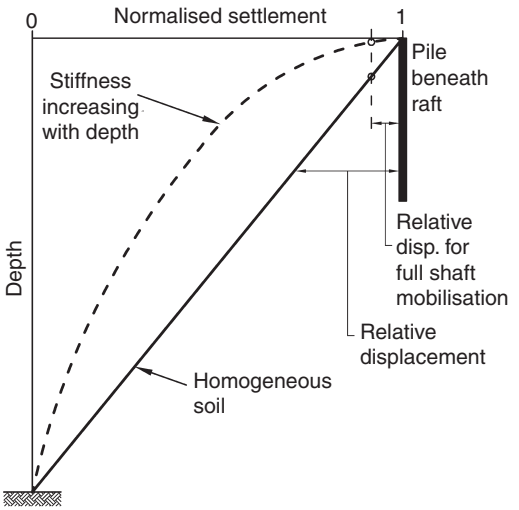
(a) Scenarios considered





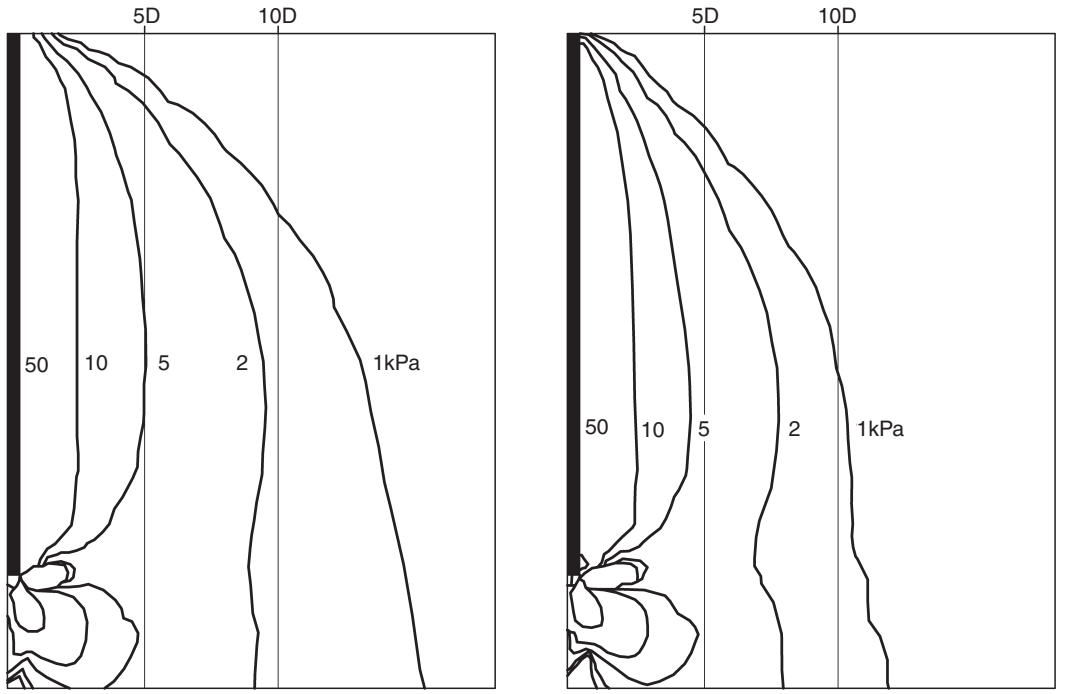


(b) Mobilisation of load with settlement - compariosn between a free standing pile and a single pile beneath a wide raft for three ground conditions. (Note that the settlement of the raft = total displacement of th raft minus the displacement at the depth of the pile base in the far field).



(c) Relative displacement between soil and pile beneath a wide raft

Figure 56.27 Influence of raft–soil–pile interaction on pile settlement characteristics
Modified from Burland (1995), all rights reserved



(a) Single free-standing pile

(b) Single pile beneath rigid raft

Figure 56.28 Contours of vertical shear stress corresponding to 50 mm settlement for a linear elastic-perfectly plastic soil with stiffness increasing with depth

Modified from Burland (1995), all rights reserved

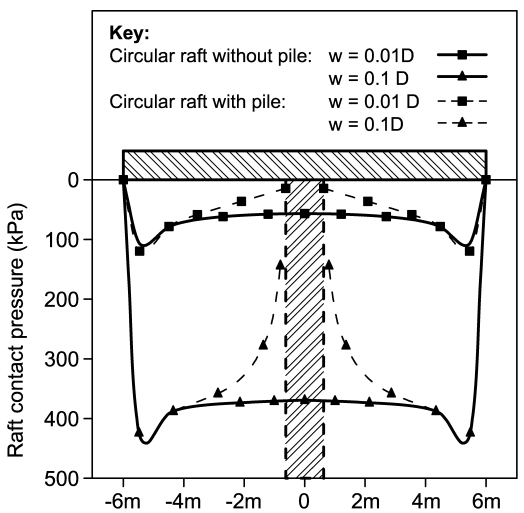


Figure 56.29 Influence of pile–soil–raft interaction on contact stress distribution beneath a piled raft

Modified from Katzenbach *et al.* (2000), all rights reserved

contact stress on the underside of a raft is locally reduced around the pile shaft (within about 2*D* to 2.5*D* of the pile centreline where *D* is the pile diameter).

The influence of nonlinear stress–strain behaviour on the pile–raft interaction has been assessed by Katzenbach *et al.* (2000). Figure 56.30 summarises the load-settlement characteristics of two piled rafts. Both rafts are 45 m wide and founded on a deep layer of stiff clay. Model M1 has 64 piles and M2 has 16 piles. The nonlinear load-settlement behaviour of the raft and pile group was included within the modelling. The load sharing between the piles and raft is summarised on **Figure 56.30(c)** for M1 and M2. It can be seen that the load share for M1 is practically constant across a wide range of applied loads. **Figure 56.30 (a)** indicates that the ultimate pile-group capacity for M1 is larger than the applied load. For model M2, the load share is also reasonably constant, until the ultimate capacity of the pile group is fully mobilised (for loads in excess of *A* as shown on **Figures 56.30(b)** and **56.30(c)**). At larger applied loads, the load share between the raft and piles becomes strongly nonlinear. Therefore, provided the pile-group capacity is *not* fully mobilised, then the analysis of raft-enhanced pile groups can be simplified, since the influence of nonlinearity on pile/raft load sharing can be assumed to be negligible. Hence, the elastic interaction factors shown on **Figure 56.22** can be assumed to be sufficiently accurate for practical design purposes.

56.6.4 Optimum location and number of piles in a piled raft

Studies by Randolph (1994) and Katzenbach *et al.* (1998) provided some useful insights into the optimum location and number of piles in raft-enhanced pile groups. **Figure 56.31**

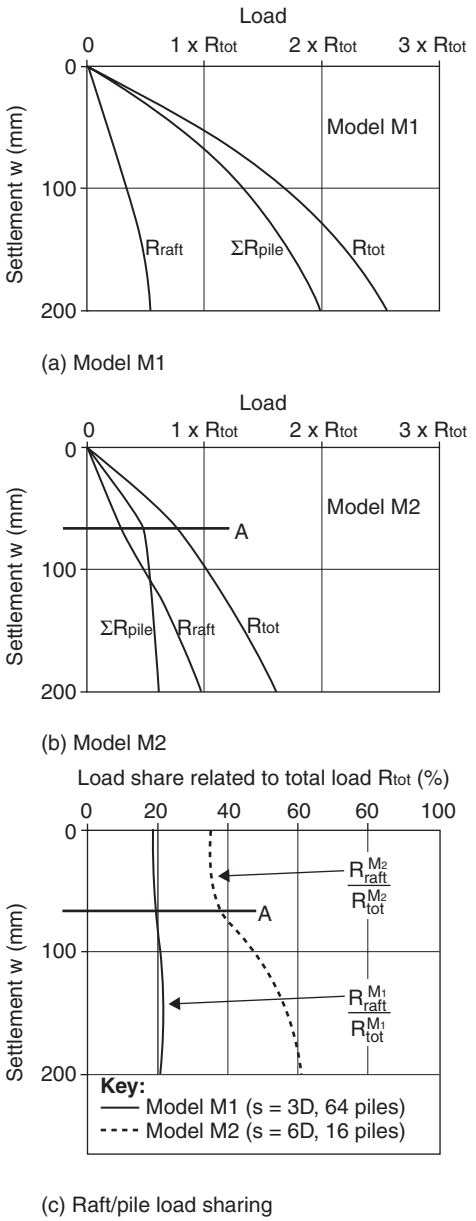


Figure 56.30 Influence of pile-group capacity on load sharing between the raft and pile group

Modified from Katzenbach *et al.* (2000), all rights reserved

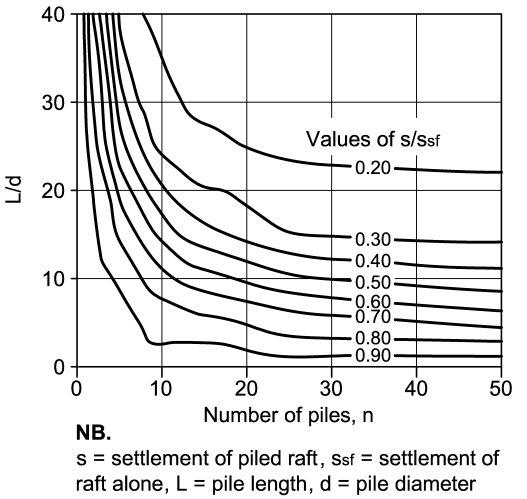


Figure 56.31 Piled-raft interaction diagram showing settlement reduction versus pile length and number of piles

plots the relative settlement (the ratio of piled-raft to unpiled raft settlement) against the number of piles and their length to diameter ratio, L/*d*. In this example, 50 piles are required for a conventional pile-group design. Considerable reductions in settlement are achieved with the installation of 10 piles, especially if those piles are relatively long (*L*/*d* >20); however, only minor additional reductions in settlement are achieved by using more than 20 piles. Similar observations have been made for very large pile groups. Viggiani (1998) back analysed the pile group for the Stonebridge Park Building in London (comprising 350 450-mm-diameter, 13-m-long piles), and found that a reduction in the number of piles to a third of the original value (i.e. 117) would have only caused a small increase in settlement (of about 5 mm to 10 mm). The raft would have then carried about 20% to 25% of the building load.

Conventional approaches for pile-group design tend to adopt a uniform distribution of piles beneath the raft. Total settlement will be reduced by the piles and differential settlement will be reduced as a consequence of the lower total settlement. A more direct and efficient approach is to locate piles in such a way as to minimise the differential settlement directly. The optimum location of piles in a piled raft will depend primarily on whether the applied structural loads can be characterised as either uniformly distributed (say through a stiff sub-structure) or are concentrated beneath a small number of heavily loaded columns. For a uniformly distributed load the principles for differential settlement control are shown on **Figure 56.32**: if a small number of piles are located beneath the central area of the raft then the differential settlement can be minimised. For a uniformly distributed load on a relatively flexible raft there will be a tendency for the raft to 'sag'. By locating the piles near to the middle, the sagging will be reduced. Similarly for a rigid raft there will be a concentration of contact stress near the edges. Again, by locating the piles near the centre of the raft a more uniform contact-stress distribution will be achieved, reducing raft bending moments and shear stresses. **Figure 56.33**, from Randolph (1994), shows an example of the effectiveness of this approach. A conventional pile group (81 0.8-m-diameter 20-m-long piles at 5*d* spacing) reduces the differential settlement by a small amount, compared with the unpiled raft. In contrast, the installation of just 9 1.5-m-diameter, 30-m-long piles, situated within the central third of the raft reduces the differential settlement far more effectively, across a wide range of loads. For concentrated superstructure loads, Poulos (2001) provides a set of linear elastic solutions, which may be useful

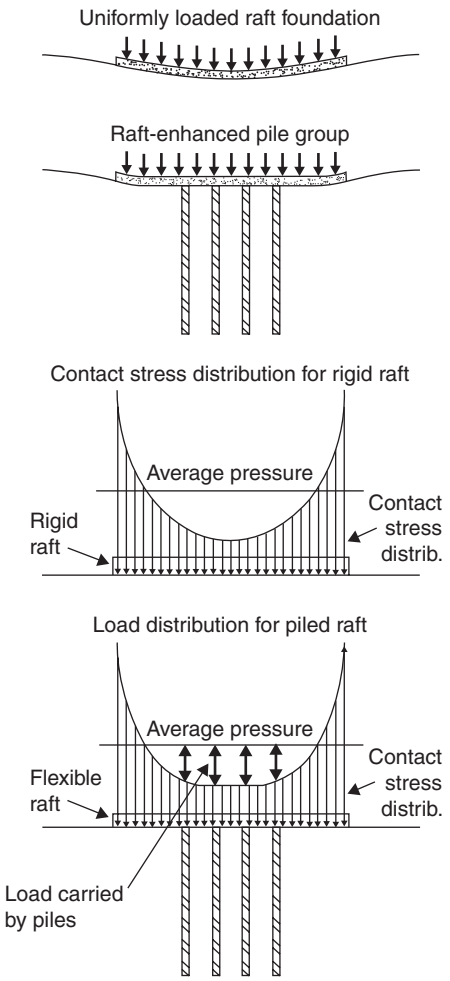


Figure 56.32 Optimising the pile locations for a uniformly loaded piled raft

Modified from Randolph (1994), all rights reserved

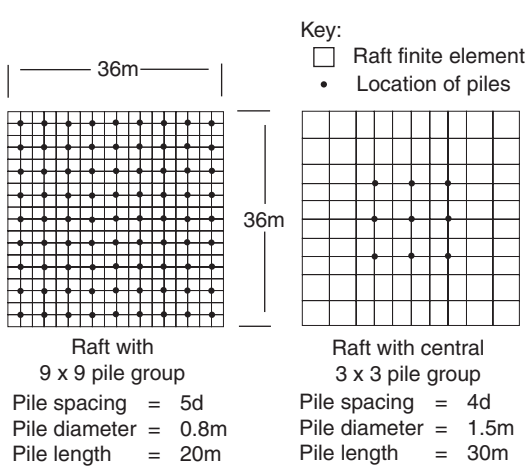
for preliminary analysis. The main effect of concentrated column loads is to generate relatively large raft bending moments compared with uniform loading, whereas raft/pile load sharing and total settlement are practically unaffected. The detailing of the raft steel needs to be carefully considered. The connections between raft bars and those sticking up from piles below is often an issue. If the raft steel is especially heavy, exceptionally careful orientation of the pile steel is required to ensure it fits the same orthogonal pattern, for instance, by use of a rectangular cage over the upper section of the pile.

In the context of practical design, the examples given in **Figures 56.31**, **56.32** and **56.33** are rather idealised; nevertheless the important practical conclusion is that in order to minimise the total and differential settlement the use of a smaller number of longer piles located across the central area of a raft will often be far more effective than a large number of short piles uniformly distributed across the whole raft.

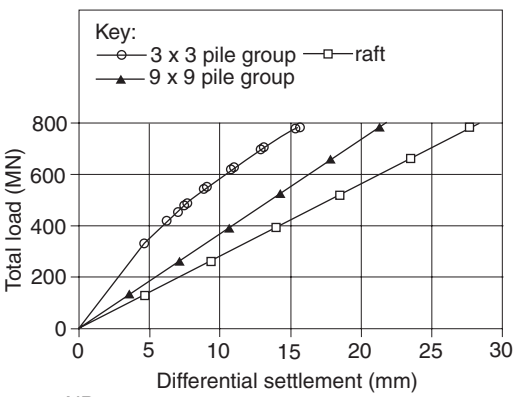
56.6.5 Compensated piled rafts

When piled rafts are located at the base of a deep excavation (known as 'compensated piled rafts') their behaviour is different to piled rafts located close to the ground surface (as discussed by Sales *et al.,* 2010) in the following respects:

- a compensated piled raft experiences less settlement;
- the piles take a smaller proportion of the load and the raft a larger proportion;
- the pile behaviour and the distribution of load between the raft and piles will be modified by the excavation and pile installation sequence;
- the presence and magnitude of a buoyant force (due to the raft being below the water table) will significantly affect the overall behaviour, and the raft load will be particularly sensitive to the magnitude of the buoyant force.



(a) Pile-group layouts



NB.
Differential settlement is between corner and centre of raft.

(b) Differential settlement

Figure 56.33 Influence of pile-group layout on raft differential settlement

Modified from Randolph (1994), all rights reserved

Construction stage (refer to text)	Time (months)	Soil condition	Load (MN)				
			Structure			Soil and groundwater	
			Dead load		Live load	Excavated soil	Buoyant force
			Raft	StrucCture			
1A	2.5	Undrained	22	-	-	–77.3	-
1B	4.4	Undrained	22	37.7	-	–77.3	–0.99
1B	8.5	Undrained	22	47.3	-	–77.3	–11.14
1B	10.25	Undrained	22	70.7	-	–77.3	–15.42
2	12.7	Undrained	22	103.5	-	–77.3	–21.5
2	15.7	Undrained	22	159.3	-	–77.3	–28.96
2	18.8	Undrained	22	168.8	-	–77.3	–29.7
2	24.8	Undrained	22	205	-	–77.3	–29.7
3	> 24.8	Drained	22	206	17	–77.3	–29.7

Table 56.5 Compensated piled raft – example for Hyde Park Barracks
Data taken from Sales *et al.* 20103

Table 56.5 shows an example of the building loads compared with the weight of excavated soil and the buoyant force from groundwater pressure, for the Hyde Park Barracks in London. This case history was described by Hooper (1973) and back analysed by Sales *et al*. (2010). The basement is nearly 9 m deep and the groundwater table is about 4 m below ground level. For this project, the weight of excavated soil is about 32% of the total load from the structure, and the weight of excavated soil plus the buoyancy force is about 44% of the total load from the structure. Clearly, for a compensated piled raft, any analysis ignoring the influence of the weight of excavated soil and the buoyancy force will give erroneous predictions of the piled-raft behaviour. The modelling of the construction process should be split into several key stages, for example:

Stage 0: Excavate to raft formation level.

Stage 1A: Raft concreting: the wet concrete will apply a load onto the base of the excavation. At this stage the raft is flexible and the 'piled raft' is ineffective. The settlement of sandy soils may be largely completed before the concrete has hardened, whereas for clayey soils some undrained settlement will occur and time-dependent consolidation settlement will occur after raft hardening (and hence lead to some raft–pile load sharing).

Stage 1B: Subsequent construction of the structure will take place with an effective piled raft after the raft has hardened, and there will be load sharing between the raft and piles. At this stage the weight of the structure is less than the weight of the excavated soil and the buoyancy force; only towards the end of 1B will the net effective weight be positive.

Stage 2: Throughout this stage, the structural weight exceeds the weight of excavated soil plus buoyancy force. The increment of soil settlement that occurs should be based on the net loads, i.e. the weight of structure minus (the weight of the excavated soil plus buoyancy force). In contrast, the loads that are transferred through the raft and piles should be based on the weight of the structure only.

Stage 3: This is the long-term condition, after consolidation of the soil.

The axial loads in the piles will be directly affected by the construction sequence. If the piles are installed before the bulk excavations for the basement, then heave-induced tensile forces will be induced in the piles, due to short-term heave of the piles (this is discussed in Chapter 57 *Global ground movements and their effect on piles*). If the piles are constructed after basement excavation, then short-term heave-induced tensile forces will not be generated in the piles. During Stage 1, when the structure weight is less than the weight of the excavated soils and the buoyancy force, there will be two components of vertical soil deformation: undrained settlement due to increments of load from the construction of the structure, and time-dependent swelling (i.e. upward movement) due to the net reductions in vertical effective stress at the base of the excavation. The overall 'net' effect (i.e. the overall downward or upward movement) will be dependent on the soil permeability compared with the speed of construction. If the soil is relatively permeable there will be a net upward movement, whereas for relatively low-permeability soils there will be a net downward movement. For the back analysis of Hyde Park Barracks, Sales *et al*. (2010) used a continuum model (finite element for the raft and boundary element for soil and piles). The soil below the raft is assumed to behave as a linear elastic-perfectly plastic material. In order to match the observed behaviour, the following were important:

(i) An increase in the 'elastic' modulus for the soil immediately below the raft (from a value of about 57 MN/m^2 to 284 MN/m^2, i.e. five times higher and close to the very small strain modulus, E_o or E_{max}, refer to Chapter 52 *Foundation types and conceptual design principles*).

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(ii) An increase in the flexural stiffness of the raft to replicate the stiffening effect of the sub-structure shear walls; the actual raft thickness was 1.5 m, but locally in the model this was increased to 10 m, to simulate the much higher effective flexural stiffness of the basement structure.

The change in groundwater pressure over time was replicated by assuming a linear increase in the buoyancy force with time from month 4 (after excavation) to the full buoyancy force after month 16. This is a simplified way of allowing for the dissipation of excess pore water pressures in the stiff clay with time. For other (more permeable) soils this could occur more rapidly.

56.6.6 Lateral loads on raft-enhanced pile groups

The potential for friction to be mobilised on the underside of a raft, means that lateral loads can be shared between the raft and piles in a similar manner to vertical loads. However, a cautious approach is necessary to ensure that the piles are structurally strong enough to resist their share of the applied horizontal load. If not, they could fail in a brittle and progressive manner. The critical scenario for lateral loading is a careful assessment of the lower-bound contact stress on the raft. This is likely to occur for the lower-bound soil stiffness close to the underside of the raft (hence a relatively 'soft' raft stiffness), and an upper-bound soil stiffness at depth (and hence a relatively stiff pile-group stiffness). The behaviour of pile groups under a horizontal load is introduced in Chapter 55 *Pile-group design*.

56.7 Pile-enhanced rafts

56.7.1 Introduction

Sections 56.2 and 56.3 describe approaches for the analysis of settlement and the structural performance of raft foundations, respectively. Usually, for a raft foundation the bearing capacity is not a limiting factor. When analysis indicates that the settlement (total or differential) is excessive or the structural resistance of the raft is exceeded locally, consideration has to be given on how to improve the design. The structural performance of a raft can be modified by increasing its thickness. However, this can prove costly. Moreover local thickening is often undesirable as changes in the excavation level are best avoided and the associated reinforcement details can be difficult and expensive to implement.

Reducing settlement will usually require piles. The conventional methods of pile design are essentially based on providing an adequate factor of safety against bearing-capacity failure. As pointed out in section 56.1 this leads to the question 'How many piles are required to carry the weight of the building?' Thus, what is essentially a limiting settlement problem is converted into a bearing-capacity problem, which can result in a very expensive solution. When settlement is the controlling factor, it is more appropriate to pose the question 'How many piles are required to reduce the settlement to an acceptable amount?' The approach of providing a sufficient number of piles to limit settlement can also be used to control differential settlement, bending moments and shear stresses in the raft. Rafts that are designed with these objectives are called *pile-enhanced rafts*. Note that this term is more appropriate than *settlement-reducing piles*, which was originally proposed by Burland *et al.* (1977) and Burland (1995).

56.7.2 Ductile load-settlement behaviour of piles

Straight-shafted piles are used for pile-enhanced rafts and an essential requirement is that their load-settlement behaviour should be ductile. This is usually the case for straight-shafted piles in clayey soils. As pointed out in Chapter 22 *Behaviour of single piles under vertical loads*, shaft resistance is essentially a frictional phenomenon and once it is fully mobilised it usually does not change significantly with further displacement. Some examples of pile tests on a wide variety of clay soils are given in the following.

Reese and O'Neill (1988) present numerous load-settlement results for straight-shafted bored piles in clay soils. These show that the carrying capacity is fully mobilised at settlements of about 10 mm and thereafter remains approximately constant for large settlements. The results of the classic research by Whitaker and Cooke (1966) on bored piles in London Clay are of particular relevance for pile-enhanced rafts. **Figure 56.34** shows the

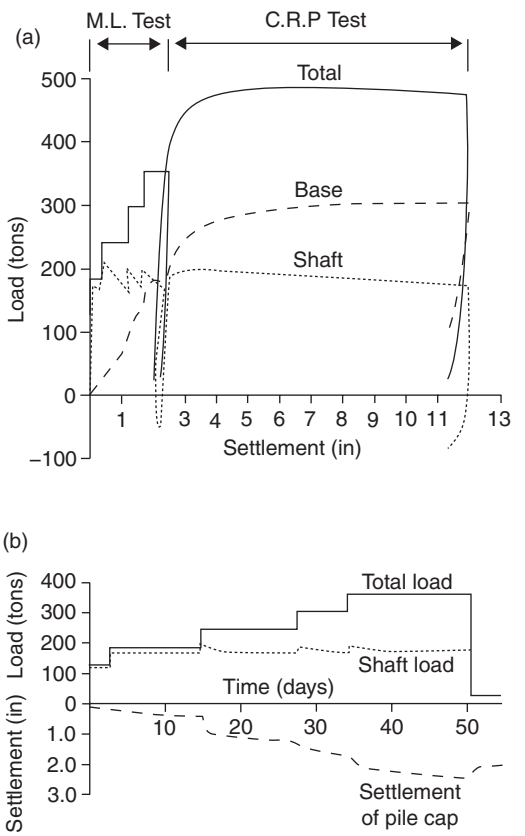


Figure 56.34 Load test on instrumented under-reamed bored pile in London Clay. Length is 12.2 m, shaft diameter is 0.79 m and base diameter is 1.68 m (Whitaker and Cooke, 1966)

results of a test on an under-reamed bored pile in London Clay in which a load cell was used to separate the base resistance from the shaft resistance. For the first part of the test the loads on the pile were applied in increments with the load at each stage being maintained constant until settlement had practically ceased. In some cases the load was held constant for 15 or more days. The second part of the test consisted of carrying out a constant rate of penetration test on the pile so as to determine its ultimate carrying capacity. The full line in **Figure 56.34(a)** shows the load-settlement behaviour of the pile as a whole.

The broken line labelled 'shaft' in **Figures 56.34(a)** and (b) shows the measured load-settlement behaviour of the shaft. It is clear that the carrying capacity of the shaft is fully mobilised at very small settlements – less than 10 mm. The jagged 'saw tooth' shape of the load settlement curve is of interest. When an additional increment of load is applied to the pile, the load carried by the shaft initially increases and then reduces as settlement takes place, as can be seen from **Figure 56.34(b)**. The temporary increase of load carried by the shaft is a 'rate effect' due to the initial high rate of settlement with time (Burland and Twine, 1988). As the rate of settlement steadily reduces so the load carried by the shaft reduces back to its previous value with the reduction in shaft load being shed to the base. It can be seen that the equilibrium fully mobilised carrying capacity of the shaft remains remarkably constant with increasing settlement, confirming the ductile nature of the shaft behaviour. All of the pile test results reported by Whitaker and Cooke show similar behaviour. As pointed out in Chapter 22 *Behaviour of single piles under vertical loads*, it is important to note in **Figure 56.34(a)** that, for working loads in excess of about 170 tons, the pile is operating with full mobilisation of the shaft carrying capacity. There must be tens of thousands of large-diameter bored piles operating satisfactorily in this way worldwide.

Figure 56.35(a) shows the load-settlement behaviour of a 762-mm-diameter 55-m-long open-ended tubular pile driven into a soft low-plasticity clay at Pentre, England (Cox *et al.,* 1992). The dotted line shows the measured elastic shortening of the pile. It can be seen that the peak resistance develops at a relative settlement of about 10 mm (total settlement minus elastic shortening). Thereafter there is a small reduction in capacity for an additional settlement of about 60 mm. The reduction in capacity is less than 10%.

Figure 56.35(b) shows the load-settlement behaviour of a 762-mm-diameter 33.5-m-long open-ended tubular pile driven into hard till overlying Oxford Clay at Tilbrook Grange, England (Cox *et al.,* 1992). The behaviour is remarkably similar to the pile in the soft Pentre clay. Peak resistance develops at a relative settlement of about 10 mm and thereafter there is a small reduction of carrying capacity with increasing settlement up to more than 70 mm.

In conclusion, shaft resistance is essentially a frictional phenomenon and as such it is permanent and reliable provided the effective stresses acting normal to the pile shaft do not change significantly. Numerous test results on straight-shafted driven and bored piles in clayey soils show that full mobilisation takes place at settlements of between about 5 mm and 10 mm and

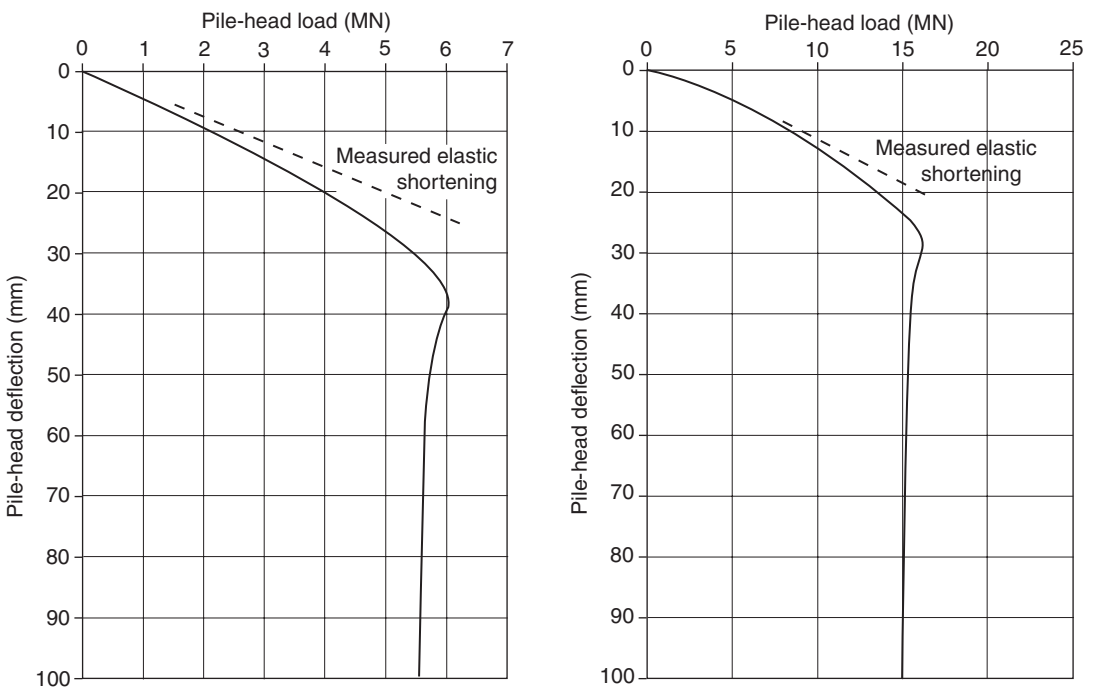


Figure 56.35 (a) Load test on 762-mm-diameter, 55-m-long open-ended pipe pile driven into normally consolidated low PI clay at Pentre; (b) load test on 762-mm-diameter, 33.5-m-long open-ended pipe pile driven into hard overconsolidated Oxford Clay at Tilbrook Grange

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that at larger settlements little or no reduction or increase in shaft resistance takes place. If a pile-enhanced raft is proposed it is prudent to carry out loading tests to large settlements on the proposed type of pile to confirm that the load-settlement behaviour is sufficiently ductile.

56.7.3 Basic mechanisms of behaviour of pile-enhanced raft foundations

We return to the question posed previously but expand it a little:

> *How many piles are required, together with their arrangement, to reduce the settlement (both total and differential) or the stresses in the raft to an acceptable level*?

This question lies at the heart of designing a pile-enhanced raft.

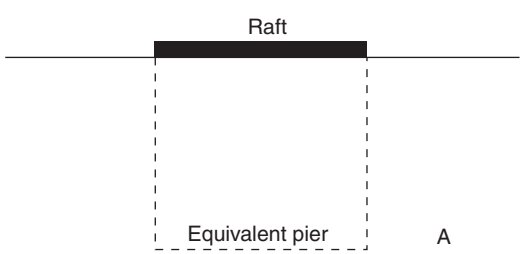
We deal first with the question of the number of piles required to limit the settlement to an acceptable amount. **Figures 56.36(a)** and (b) show, respectively, a raft foundation on a deep layer of clay and a graph of settlement vs the number of piles. With no piles the calculated maximum final long-term settlement is given by point *A* in **Figure 56.36(b)**. After a consideration of the structural and architectural requirements of the building to be supported by the raft, it is concluded that the settlement (or, usually, the differential settlement) is unacceptably large so that piles are required to reduce the settlement. In this situation a full pile group designed by conventional methods can be represented by an equivalent pier as shown in **Figure 56.36(a)** – see Chapter 55 *Pile-group design* (section 4.5.5). A settlement calculation of the equivalent pier will give a much reduced settlement represented by point *B* in **Figure 56.36(b)** but at considerable cost.

If, instead of the conventional approach, the decision is taken to adopt a pile-enhanced raft solution, it will first be necessary to decide on an acceptable maximum settlement – see Chapter 52 *Foundation types and conceptual design principles* (section 1.3). Point *C* in **Figure 56.36(b)** represents the chosen maximum allowable settlement for this case. Straight-shafted (ductile) piles are used and the load required to fully mobilise the shaft resistance of each is calculated. If a small number of piles are used, the settlement of the raft will be sufficient to fully mobilise their shaft resistances. The upward resistance of each pile acting on the raft foundation will reduce the load transmitted to the ground through the raft by the mobilised shaft resistance. If we proceed to add one pile at a time it will be found that the settlement of the raft will initially reduce in inverse proportion to the number of piles as shown in **Figure 56.36(b)**. This is because the spacing between the piles is large and there is little or no interaction between them. As the number of piles is increased the spacing between them reduces until there comes a point where the interaction between the piles and the raft foundation begins to become significant. The relationship between the settlement and the number of piles then becomes nonlinear and approaches the settlement of the equivalent pier foundation asymptotically. Provided the chosen allowable total settlement is large enough to be consistent with the guidance summarised in Chapter 52 *Foundation types and conceptual design principles* then the number of piles is likely to be small enough (and the spacing between them large enough) for there to be negligible interaction between them.

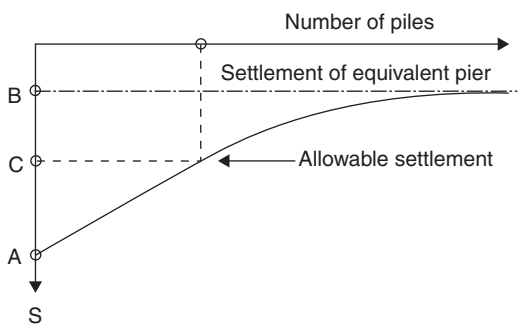
Burland (1995) drew the following conclusions from a study of pile–raft interaction (described in 56.6.3 above):

1. The raft–soil–pile interaction results in the pile beneath the raft mobilising its shaft resistance less rapidly with settlement than a free-standing pile. However, for raft settlements greater than 20 mm, more than 90% of the total shaft resistance of the equivalent free-standing pile is mobilised.
2. Computed contours of vertical shear stress around the pile beneath the raft show that the pile–soil–pile interaction can be ignored for spacing factors greater than about 6, where the soil stiffness increases rapidly with depth.
3. The pile–soil–raft interaction and the raft–soil–pile interaction can be simply accounted for in design by reducing the fully mobilised capacity of the piles by about 10%.

Observations of the pile–soil–pile interaction, by Mandolini *et al*. (2005) for a broad range of soil types, indicates that



(a) Raft, piled raft and conventional pile group geometry



(b) Raft, piled raft and conventional group settlement

Figure 56.36 Relationship between number of piles and settlement of a piled raft

interaction is negligible beyond about 8*d*. The critical spacing factor for the pile–soil–pile interaction to be negligible, depends on the variation of ground stiffness with depth. If the soil stiffness is relatively uniform a spacing factor of 8 will be an appropriate assumption, whereas if the soil stiffness increases rapidly with depth, then a spacing factor of 6 will be appropriate.

56.7.4 A simple design approach

The practical importance of these conclusions is that the analysis of a pile-enhanced raft can be carried out by a standard raft computer program (see section 56.2) in which the raft is modelled as plate elements and the deflections of the ground are derived from elastic half-space theory or the 1D method (see Chapter 53 *Shallow foundations*). The actions of the fully mobilised piles are simply modelled as appropriate upward forces applied at the chosen pile locations. For a given raft geometry and loading distribution, the locations of the piles can be successively adjusted to minimise the differential settlements, bending moments and shear stresses in the raft.

The above approach is illustrated in **Figure 56.37(a)**, which shows a raft foundation on a relatively uniform deep layer of clay and subjected to local high column loads. As shown in **Figure 56.37(b)** the raft foundation will undergo significant settlement in a sagging mode and there will be high local bending moments beneath the columns as shown in **Figure 56.37(c)**. By placing a suitably designed ductile pile under each column it is possible not only to reduce the bending moments in the raft as shown in **Figure 56.37(c)** but also the total and, in particular, the differential settlement as shown in **Figure 56.37(b)**.

Figures 56.32 and **56.33** illustrate how pile locations can be optimised to minimise differential settlements for raft-enhanced pile groups. Exactly similar approaches can be adopted for pile-enhanced rafts but the analysis will be much simpler, since pile–raft and pile–pile interactions can be ignored. If the pile spacing ratios turn out to be less than about 6 to 8 (depending on the variation of soil stiffness with depth) then a pile-enhanced raft will not be appropriate. However, it is worth revisiting the reasons for the choice of the maximum allowable settlement. Increasing the allowable total settlement will reduce the number of piles but this does not necessarily lead to increases in differential settlement or raft stresses provided the piles are located appropriately.

Love (2003) describes a case history of the design of a pile-enhanced raft foundation on London clay for which there was a considerable saving both in the number of piles and in the raft thickness when compared with a conventional design of a fully piled rigid raft. The building is a reinforced-concrete framed structure, seven storeys high, with a basement. Love stresses the following design requirements:

1. The load-displacement behaviour after full mobilisation should be as near as possible constant.
2. In practice the actual load deflection relationship will not be perfectly constant, see for example **Figure 56.35**. Unlike conventional pile designs, overcapacity can be as much as a problem as undercapacity; see **Figure 56.38**.
3. Raft settlement should always be greater than the amount required to mobilise full shaft friction.
4. Reasonable upper and lower bounds should be set for the fully mobilised pile loads, **Figure 56.38**, and the raft design checked for both extremes. **Figure 56.39** is taken from Love's paper and illustrates the designed upper and lower bound settlement profiles across the raft at various sections. It should be noted that a pile was not needed beneath every column.
5. In order to reduce the potential variation between the lower and upper bound pile capacities, the end-bearing resistance was eliminated. A variety of different methods can be used to eliminate, or reduce, end bearing if thought necessary.
6. Three pile tests were carried out and the tests were required to pass the design upper and lower bound load-settlement

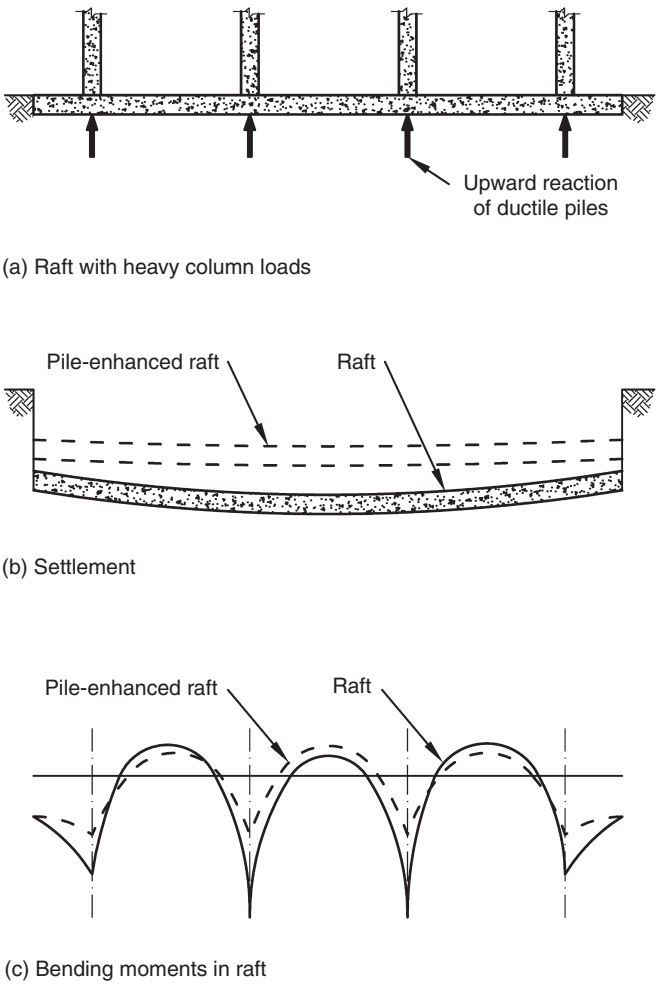


Figure 56.37 Illustrative example of a pile-enhanced raft foundation showing how the use of ductile piles beneath heavily loaded columns can reduce the total and differential settlement and the bending moments

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criteria set for the design, points 1 and 2 in **Figure 56.38**. In order to capture the full load-settlement behaviour of the pile, the load tests were run as load controlled up to yield and as displacement controlled thereafter. As mentioned in section 56.6.2, it should be noted that the shaft resistance is rate dependent and a constant rate of displacement of 2 mm/min is likely to give shaft resistances about 15% higher than the static resistance (Burland and Twine, 1988).

56.7.5 Lateral loads on pile-enhanced rafts

The use of pile enhancement for rafts and footings carrying lateral loads has the potential for achieving very significant economies. In the traditional approach to pile-group design most of the vertical load is assumed to be carried by the piles. This leads to difficulties in carrying lateral loads since no reliance can be placed on frictional resistance between the base of the raft and the underlying ground, as would be the case for an unpiled foundation. Therefore, either the pile group has to be designed to carry the horizontal loads or raking piles must be introduced. Both these solutions are expensive and unsatisfactory for a variety of reasons. It is evident that a 'vicious circle' has developed in which an excessive number of vertical piles inhibits the ability of the foundation to transfer horizontal loads into the underlying ground.

However, if a pile-enhanced raft design is adopted, the number of piles can be substantially reduced and a significant contact stress between the raft and the ground is retained. Hence the frictional resistance between the underside of the foundation and the ground can be utilised to transmit the horizontal shear stresses into the ground and the need for additional piles, either vertical or raking, is eliminated. This approach requires a careful assessment of the likely horizontal displacements of the raft together with the associated induced bending moments at the top of the piles. It is important to note that the relative stiffness of the piles and raft under lateral loads needs to be considered (and *not* just their ultimate capacity), in order to assess the load share between the raft and piles. The reader is referred to Chapter 54 *Single piles* for an introduction to the design of single piles subject to lateral loads.

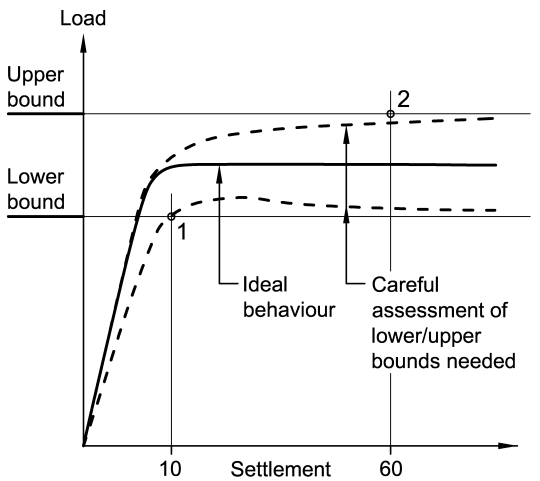


Figure 56.38 Pile-enhanced rafts – pile load-settlement response

56.8 A case history of a pile-enhanced raft – the Queen Elizabeth II Conference Centre

An early application of pile-enhanced rafts was the design of the foundations of the Queen Elizabeth II Conference Centre in Westminster, London (Burland and Kalra, 1986) – see **Figure 56.40**. The conference centre is founded on a 2-m-thick raft in a relatively shallow excavation. The calculated maximum total settlement was in excess of about 20 mm. Over half of the total weight of the superstructure is transmitted to the raft by means of a few columns, which are located at the edges of the raft and carry loads of up to 26 MN. The preliminary analysis of the raft identified local areas of high bending moments and shear forces and concern was expressed that the bearing pressures under some of these columns might induce local yielding and excessive settlement in the underlying London Clay.

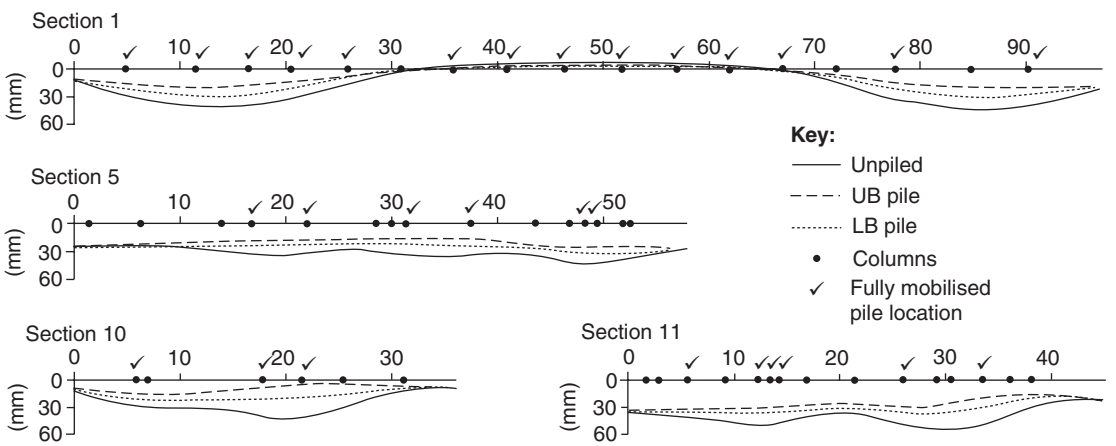


Figure 56.39 Upper and lower bound settlement profiles for a pile-enhanced raft compared to an unpiled raft (Love, 2003)

One method of dealing with the problem would have been to thicken the raft locally. This would have been a costly and time-consuming approach with the potential for causing delays and difficulties in the construction of the major raft foundation. The local thickenings would have been at the edges of the raft and there were significant 'knock-on' costs in the design of the retaining walls.

A pile-enhanced raft appeared to be a suitable solution. Single straight-shafted piles were placed directly beneath the most heavily loaded columns in the knowledge that the settlement of the raft would be sufficient to fully mobilise the shaft resistance of the piles. The effect was to apply a constant upward force beneath each column. By this means the resultant loads transferred from the columns into the raft were significantly reduced. The piles are each 16 m long and 1.8 m in diameter. The calculated ultimate shaft resistance was 6.4 MN giving effective reductions in the resultant column loads of between 25% and 46%.



Figure 56.40 Queen Elizabeth II Conference Centre

Figure 56.41 shows the calculated bending moments along a 3-m-wide edge strip with the locations of the columns indicated by arrows. The bending moment labelled 1 relates to the undrained case without piles. The crosses refer to the worst values for some other cases considered, including the fully drained long-term case and partially drained cases involving some heave. The bending moment labelled 2 is the undrained case with stress-reducing ductile piles in place beneath the columns. This proved to be the worst case considered using such piles. It can be seen that the presence of the ductile stress-reducing piles beneath each column results in a considerable reduction in the bending moments, particularly beneath the columns.

In view of the novelty of this approach the Building Research Establishment was invited to instrument one of the piles to measure the magnitude of the load transferred to it through the raft foundation. A full description of the investigation is given by Price and Wardle (1986).

Figure 56.42 shows the results of the measurement. The full line and the broken line represent the measured pile-head loads and the calculated column loads, respectively, noting that they are plotted to different scales. Early on in construction the pile-head loads showed some fluctuations due to intermittent load applications with a tendency for the underlying clay to swell during intervening periods. The main construction began in the middle of 1982 as can be seen by the rapid increase in the column loads. The measured pile-head loads reflect the loads on the columns. Construction was completed during 1984. It can be seen that by 1986 the measured pile-head load was in good agreement with the calculated value of 6.4 MN for the fully mobilised shaft resistance. Measurements continued through to 1996 and it is clear that no significant change in the load carried by the pile has taken place over a period of about ten years.

56.9 Key points

(1) Rafts – the differential settlement and bending moment induced in a raft is fundamentally affected by its stiffness. Raft stiffness is a function of soil and concrete stiffness, and, particularly, the ratio of raft thickness to raft width (or diameter).

(2) Raft on springs vs soil continuum – a structural design often assumes the soil beneath a raft acts as a bed of

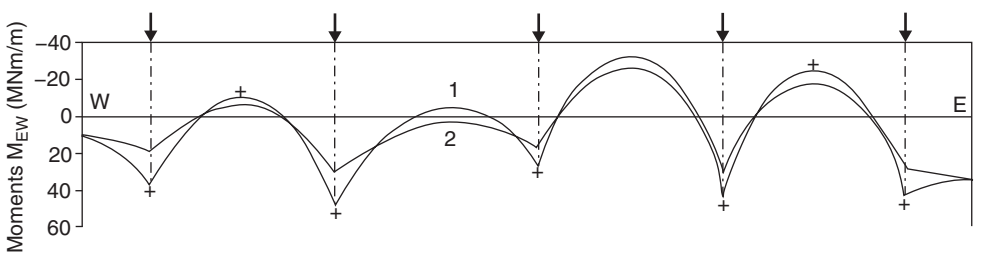


Figure 56.41 Queen Elizabeth II Conference Centre: calculated bending moments on a 3-m-wide strip at the edge of the raft: (1) is for the raft alone and (2) shows the effects of the ductile piles
Modified from Burland and Kalra (1986)

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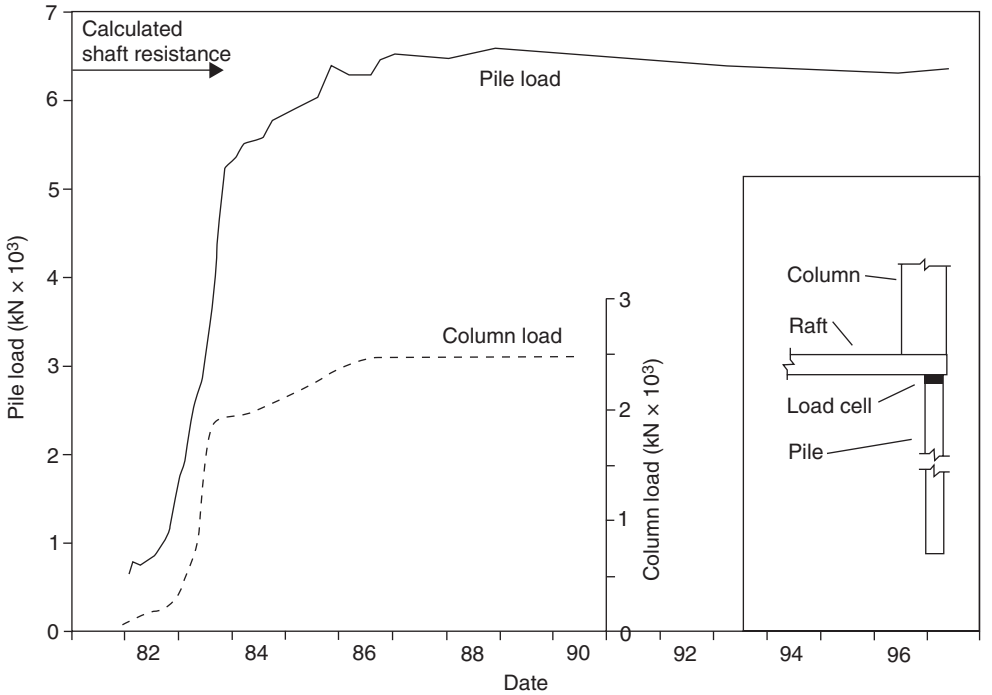


Figure 56.42 Queen Elizabeth II Conference Centre: load versus time response of instrumented pile beneath edge of raft foundation carrying a heavily loaded column
Data taken from Burland and Kalra (1986)

uniform springs. There are many pitfalls and associated errors with this approach and it is *not* recommended. A practical and more realistic approach is to have 'interactive' spring stiffnesses, with a structural spring model run in tandem with a soil continuum model. This will result in non-uniform springs, typically stiffer towards the raft edge and softer at the centre.

(3) Piled rafts – compared with conventionally designed rafts and pile groups, a piled raft can provide many benefits when used in suitable ground conditions, e.g. reduced costs and easier and safer construction. There are two main types of piled raft: the raft-enhanced pile group and the pile-enhanced raft. The design concepts are very different and must *not* be mixed.

(4) Raft-enhanced pile group – both the raft and pile group behave in a pseudo-elastic manner. A key issue is to assess the load sharing between the pile group and raft, which is dependent on the relative stiffness of the pile group and raft. The upper and lower bound stiffnesses of pile group and raft need to be assessed. This type of piled raft can be thought of as a 'value-engineered pile group', with piles omitted to the extent that raft resistance can be relied upon.

(5) Pile-enhanced raft – the piles will have fully mobilised their capacity. It is essential that the pile load-settlement behaviour is ductile. A key issue is to assess the upper and lower bound pile capacities. This type of piled raft acts predominantly as a raft, with additional local support by piles, beneath heavily loaded columns. The term 'pile-enhanced raft' is considered more appropriate than the term 'settlement-reducing piles', because the main benefits may be broader than settlement reduction alone. A pile-enhanced raft is generally chosen when a raft by itself just fails to work.

56.10 References

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It is recommended this chapter is read in conjunction with

- Chapter 19 *Settlement and stress distributions*
- Chapter 21 *Bearing capacity theory*
- Chapter 22 *Behaviour of single piles under vertical loads*
- Chapter 53 *Shallow foundations*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

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Chapter 57

Global ground movements and their effects on piles

Edward Ellis University of Plymouth, UK
Anthony S. O'Brien Mott MacDonald, Croydon, UK

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Global ground movements may occur for a variety of reasons, but are most commonly associated with cut-and-fill operations, particularly in clay soils. The impact on structural elements such as piles depends strongly on construction sequence, and normally the presence of soft clay or peat is most problematic, since large movements may occur over several months or years. The chapter considers negative skin friction, heave-induced tension and lateral interaction. Lateral ground movements are of particular concern since they have led to the structural failure of piles and pile groups on many occasions.

57.1 Introduction

Global ground movements may occur for a variety of reasons, but are most commonly associated with cut-and-fill operations, particularly in clay soils. The impact on structural elements such as piles depends strongly on construction sequence, and normally the presence of soft clay or peat is most problematic, since large movements may occur over several months or years. The chapter considers the following aspects:

1. Negative skin friction (in soil which is settling, e.g. beneath an embankment). This increases the axial load on piles, and will tend to increase pile settlement significantly.
2. Heave-induced tension (in soil which is heaving upwards, e.g. beneath an excavation). This may cause tensile failure of the piles.
3. Lateral interaction (in soil which is moving horizontally, e.g. adjacent to an embankment or excavation, or in an unstable slope). This can be particularly damaging since the piles may fail in bending or shear. This is a common cause of pile or pile group failure if this mode of ground movement is overlooked.

In each section the response of a single (individual) pile is initially considered, before proceeding to additional considerations for the group response. Generally speaking piles at the centre of a group tend to be 'sheltered' from interaction effects and therefore the assessment of a single pile response tends to be conservative for pile groups. When lateral interaction develops, progressive structural failure may be an issue (because the structural failure mode is often brittle, rather than ductile) and then a cautious approach is necessary.

Owing to the importance of construction sequence, close communication and coordination is likely to be required between the contractor or project manager and the designer. Close control of activities on site, particularly where sub-contractors may not be aware of or prefer to ignore the implications of their actions, is also important.

Owing to the complexity of many of the problems and their relation to subtle aspects of soil behaviour, input from a geotechnical specialist will be essential. The text below highlights the main points and indicates key references for more detailed information. However, most of the references are published in specialist geotechnical journals, which will assume prior knowledge of geotechnics.

Often the most pragmatic approach may be a practical 'engineering solution', which substantially 'removes' the problem. For instance:

1. inherently reduce soil movement, e.g. by carrying out ground improvement;
2. wait for significant soil movement to be substantially complete before constructing the piles;
3. isolate the piles from the soil movement.

Point 1 above normally requires that the cause of movement is largely removed, for instance by constructing an embankment from lightweight fill or by carrying out ground improvement.

Point 2 highlights the difficulty of low-permeability clay soils, which are likely to require months or years to fully consolidate. It may be undesirable to wait this long. Time-dependent ground movements tend to be predominantly vertical, whereas significant lateral ground movements mainly occur during construction (unless there is ongoing instability). Hence, when lateral ground movements are the main concern point 2 can be effective.

An example of point 3 is the use of double casings for bored cast *in situ* piles. The annulus between the two casings can isolate the pile from the outer casing, which moves with the soil.

Figure 57.1 shows an example of the extreme consequences of global ground movements on foundations. In this situation it appears that lateral interaction (due to excavation on one

side and stockpiling on the other) was a significant contributory factor, which led to a catastrophic collapse (China Daily, 2009). This failure illustrates a few key principles:

(i) it is vital to have an awareness of all the construction processes and their relationships with site topography, existing structures and infrastructures, both across a site and beyond the site boundaries;

(ii) the sequence and timing of activities can be important;

(iii) piles which are designed solely for vertical loads (common for buildings) can be extremely vulnerable to lateral ground movements.

The above means that temporary works can have a significant impact on the permanent works. Temporary and permanent works are often designed by separate organisations and the commercial and procurement environment will also be a factor, as well as technical issues. The avoidance of these major hazards, although often obvious with the benefit of hindsight, can require a major coordination effort both during design and construction.

57.2 Negative skin friction

57.2.1 Introduction

As shown in **Figure 57.2(a)**, ordinarily for piles the vertical load applied at the head (Q_h) is resisted by a combination of the resistance on the shaft (Q_s) and the base (Q_b); see Chapter 10 *Codes and standards and their relevance* and Chapter 54 *Single piles* for further details. Hence:

$$Q_h = Q_s + Q_b. \tag{57.1}$$

The maximum axial load in the pile is Q_h.

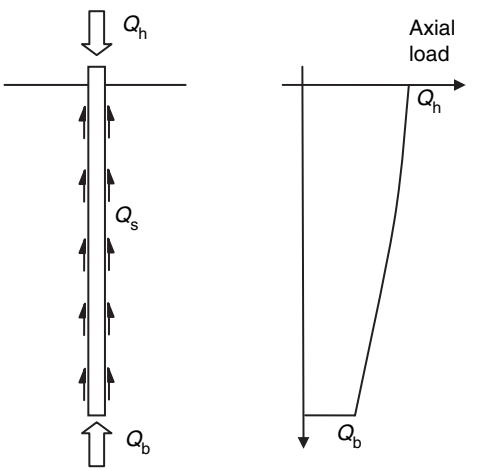
It is assumed that the pile displaces downwards, whilst the surrounding soil is static except near the pile, where it will tend to move with it. However, as shown in **Figure 57.2(b)**, if the soil inherently tends to settle more than the pile over at least part of the length, then there will be both 'positive' and 'negative' skin friction (Q_{s+} and Q_{s-} respectively). The transition from positive to negative skin friction is the point where settlement of the pile and soil are equal, and will be referred to as the neutral point. When there is negative skin friction (NSF) the maximum axial load in the pile (Q_{max}) occurs at the neutral point and exceeds Q_h:

$$Q_{max} = Q_h + Q_{s-} = Q_{s+} + Q_b. \tag{57.2}$$

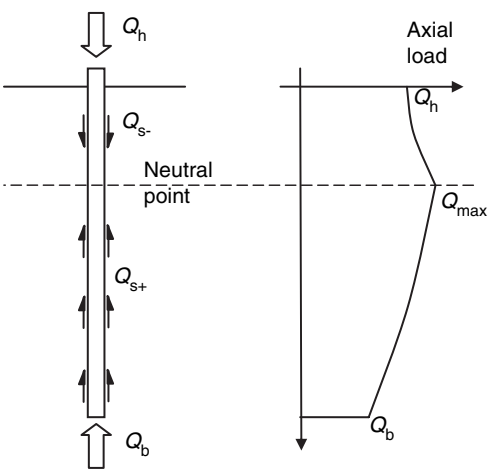
Thus, any NSF on the pile increases the maximum axial load, which may have implications for the structural design of the pile. However, settlement of the pile will also be increased, and this is likely to be the most significant practical issue that will govern design (Poulos, 2008).



Figure 57.1 Building collapse due to failure of the piles (Shanghai, 2009).
Source: China Daily www.chinadaily.com.cn



(a) Normal shaft and base resistance



(b) Negative skin friction

Figure 57.2 Effect of negative skin friction compared to normal pile action

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Settlement of the soil layer is likely to be due to imposition of a surcharge (e.g. fill material) at the surface, although lowering of the water table may also cause significant settlement without any change in load at the surface (especially for soft clays or peat). Furthermore, vegetation-induced changes in effective stress may cause both settlement and heave; refer to Chapter 53 *Shallow foundations*. The settling material may actually be fill material, which has been recently placed and is settling with the underlying material that it has loaded, as well as potentially settling within its own thickness. Where the soil is clay it will take significant time for the settlement to occur. If the thickness of compressible soil is variable, say soft clay on a steeply dipping rockhead, then lateral movement may also develop in addition to settlement. This would then require a consideration of lateral interaction effects; refer to section 57.4 below.

57.2.2 Single piles: end-bearing and 'floating'

57.2.2.1 End-bearing piles

As shown in **Figure 57.3(a)**, the simplest form of NSF loading is for a pile that derives nearly all of its capacity from end-bearing in an underlying stratum, which is significantly stiffer or stronger than the overlying soil (e.g. rock). The neutral point can then be assumed to be close to the bottom of the settling soil layer, and it will be slightly conservative to assume that negative skin friction exists over the entire length of the shaft ($Q_{s-} = Q_s$, $Q_{s+} = 0$). Equation (57.2) then becomes:

$$Q_{max} = Q_h + Q_{s-} = Q_b. \tag{57.3}$$

If the pile also derives a significant shaft capacity in the underlying stratum (i.e. $Q_{s+} > 0$), then the original version of equation (57.2) may be appropriate. However, the neutral point is still assumed to occur at the boundary between the two strata and, hence, determination of Q_{s-} and Q_{s+} is relatively straightforward.

When NSF is induced by soft clays or peats it is common to assess this by using total-stress methods. However, there are a number of problems: firstly the empirical alpha values quoted in the literature are not directly relevant for NSF (since they have only been derived from conventional load tests to measure positive shaft friction); secondly, there is usually a wide spread of alpha and c_u values, so selection of appropriate design values is problematic; and finally, depending upon the magnitude of the surcharge, the total-stress method may be either unsafe or overconservative.

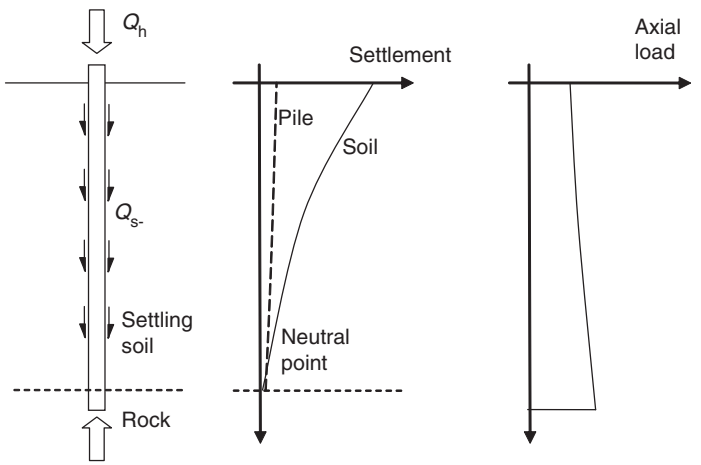
For soft clays or peats, it is recommended that NSF is calculated by using an effective-stress (β) approach (refer to Chapter 22 *Behaviour of single piles under vertical loads* and Chapter 54 *Single piles*).

57.2.2.2 'Floating' piles

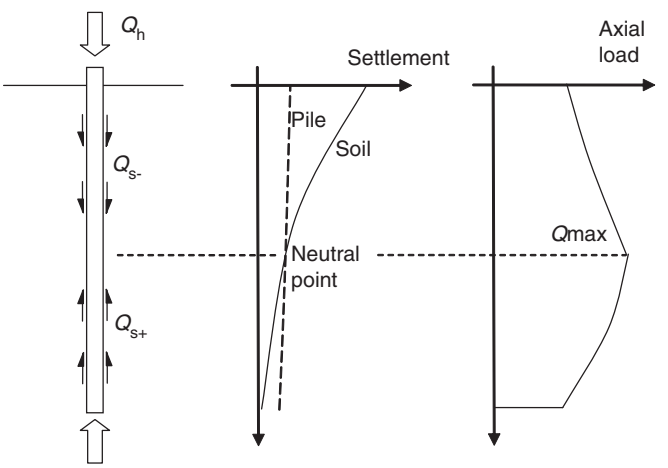
As shown in **Figure 57.3(b)**, where there is a more gradual increase of soil strength or stiffness with depth, the situation becomes more complicated, since the depth of the neutral point will not be immediately apparent. Rather, it depends upon a comparison of estimates of soil and pile settlement with depth.

57.2.2.3 End-bearing piles and brittle response

The design of end-bearing piles is generally more straightforward, since the location of the neutral point can be readily assumed. However, despite the complication of establishing the depth of the neutral point for floating piles, the response will be relatively ductile (or 'self-correcting'), since if Q_{s-} is larger than anticipated, the pile settlement will be increased, but this will in turn move the neutral point upward, tending to reduce Q_{s-}. For end-bearing piles this will not be the case, and the base resistance must have sufficient reserve capacity to cope with uncertainties in the assessment of negative skin friction. For piles which are socketed into rock, particular care may be required if they are subjected to negative skin friction. The shaft friction that develops in some rocks (sandstones,



(a) End-bearing pile – substantially stiffer/stronger substratum



(b) Floating pile – gradual increase in soil strength/stiffness with depth

Figure 57.3 End-bearing and floating piles

limestones) can exhibit brittle behaviour (i.e. the shaft friction reduces after a limiting displacement has developed). Hence, the value of any 'factor of safety' (FoS), which is applied, would need to vary depending upon which of the above three scenarios was relevant (i.e. from relatively low values for floating piles, intermediate for end-bearing piles and relatively high for piles with 'brittle' resistance characteristics).

57.2.3 Design considerations for single piles

Even for a large pile group, a consideration of the behaviour of a single pile is likely to be a logical starting point for design.

57.2.3.1 Determination of skin friction load

Determination of Q_{s+} and Q_{s-} (and Q_b) is likely to be based on routine methods as outlined in Chapter 54 *Single piles*. The shaft capacity will normally be assessed using effective-stress methods in free-draining soils or a total-stress approach in clay. However, an effective-stress approach may be used in clay, potentially leading to a reduced estimate of Q_{s-} (Cheong, 2007). Where an effective-stress approach is used in an overconsolidated clay, determination of the earth pressure coefficient near the soil surface, and the influence of pile construction-related effects will require careful consideration (Bown and O'Brien, 2008).

Since skin friction is mobilised at relatively small pile–soil displacement it may normally, and conservatively, be assumed that the 'switch' from Q_{s+} to Q_{s-} at the neutral point occurs instantaneously. The estimation of base capacity as settlement of the pile increases is likely to be more important, as discussed below.

57.2.3.2 Determination of the soil settlement profile with depth

The profile of soil settlement is an important element of any NSF design. It will require knowledge of the increment in vertical effective stress and the one-dimensional stiffness with depth (see Chapter 19 *Settlement and stress distributions* and Chapter 53 *Shallow foundations*). The distribution of strain and, hence, settlement with depth can then be derived. The resulting profile is likely to have a curvature as shown in **Figure 57.3(b)**, representing reducing strain with depth. The increment of vertical stress will reduce with depth to some extent, particularly as the depth becomes large compared to the extent of the load. Furthermore, the stiffness of the soil is likely to inherently increase with depth. This effect is compounded since soil stiffness also reduces significantly as strain increases (see Chapter 53 *Shallow foundations*), so that the relevant 'operational' stiffness may increase very significantly as strains reduce with depth. Ignoring the curvature of the settlement profile (**Figure 57.3(b)**) can lead to significant overestimation of the thickness and, hence, magnitude of NSF loading (Cheong, 2007).

The interdependence of strain and stiffness can be accounted for in a finite element analysis with a suitably sophisticated soil constitutive model, but care will also be required that the non-linearity of the response is correctly modelled by conducting the analysis in sufficiently small increments (Potts, 2003). An iterative analytical method, which can be undertaken using a spreadsheet, is described by O'Brien and Sharp (2001a and 2001b).

Once the 'free-field' soil settlement profile (i.e. not specifically accounting for the pile) has been determined, it is likely to form an input to the subsequent analysis, and can be assumed to remain constant. This will be conservative, particularly for large pile groups (see below), since the presence of the piles will tend to locally reduce settlement of the soil. A finite element analysis including the piles can potentially be used to overcome this assumption, inherently incorporating the soil–pile interaction. However, implementation of the analysis will require an experienced user, and a software package suitable for geotechnical applications. Furthermore, an appropriate soil constitutive model will be required, and modelling of the pile–soil interface properties will require careful consideration.

57.2.3.3 Determination of the pile settlement profile with depth

Generally speaking, the vertical strain of concrete in the pile will be one or two orders of magnitude lower than the strain in the surrounding soil if the soil is settling significantly. Hence, an assumption that the pile is rigid will often be appropriate, leading to a profile of settlement with depth, which is a vertical line on a plot such as **Figure 57.3**.

Settlement at the toe of the pile is likely to be significant except where there is a very stiff bearing layer. Fleming (1992) presents a relatively simple method based on hyperbolic functions, which can be used for estimation of the base response. Alternatively a suitable factor on the ultimate bearing capacity can be used to limit settlement at the toe of the pile (see Chapter 54 *Single piles*).

57.2.3.4 Determination of pile length

Routine design of a pile (**Figure 57.2(a)**) is likely to require some iteration in the sense that even for a given pile diameter the required pile length will initially be unknown, and will be dictated either by the FoS against ULS failure, or the allowable settlement at the head. Normally the pile will be 'made longer' until these criteria are achieved.

As described above, for an end-bearing pile (**Figure 57.2(a)**) the design is likely to follow this approach. The total load (Q_h + Q_{s-}) must be resisted in the stronger substratum. Note that the pile will need to extend a few diameters (approximately three to six) into the stronger substratum to develop the full bearing capacity of this layer in any case (see Chapter 54 *Single piles* and Chapter 55 *Pile-group design*). Any weathering of this material near the top of the layer will also be an issue. It may be decided to extend the pile a significant distance into the stronger layer so that a significant Q_{s+} can also be exploited. However, the ability of piling plant to extend (drive or bore) the pile into this layer may be a constraint, particularly as diameter increases.

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For a floating pile the design will be governed by serviceability. A practical starting point for design is to assume that the pile is rigid and settles uniformly by an amount corresponding to the maximum allowable settlement at the pile head. Using the soil settlement profile this then allows determination of Q_{s-}, and hence ($Q_{s+} + Q_b$) required for equilibrium. Since the calculation is a serviceability calculation a relatively small factor can be applied to Q_{s+}; however, as mentioned above Q_b will require more careful consideration or a larger factor to account for potentially significant settlement at the toe. For piles in stiff clay, Q_b is typically quite small compared with Q_{s+}; hence, it could be conservatively ignored for a simple assessment. The importance of an accurate estimation of the distribution of settlement with depth is stressed, since this can have a significant impact on Q_{s-} for a given settlement.

Poulos (2008) gives guidance on the factor of safety required in the 'stable' (non-settling zone), which should limit settlements to acceptable values. In the discussion of that paper Bourne-Webb (2009) highlights the potential importance of settlement at the toe, arguing that this approach should not rely heavily on Q_b unless this can be justified. The value of Q_b will be very important if the pile is toed into cohesionless soil or rock (when Q_b will be relatively large compared with Q_{s+}).

57.2.4 Pile groups

Lee *et al*. (2002) report finite element modelling of pile group effects on NSF loading. As might be expected, they found that piles near the centre of a large group are 'sheltered' from the effect of NSF. Thus, the estimation of the effect on a single pile as above is conservative.

Lee *et al*. (2006) furthermore consider the effect of a pile cap on group behaviour. Again, piles at the edge of the group are most prone to NSF, and tend to cause settlement of the group, which is resisted by piles in the centre. In the absence of a significant external vertical load on the pile group this can lead to tension at the head of the outer piles as they pull the cap down. This would need to be carefully considered in the structural design of the connection of the pile head to the pile cap.

Global ground movements will affect the distribution of axial forces in a pile group. For example, observations of axial forces in a piled bridge abutment at Newhaven (Reddaway and Elson, 1982) indicated a relatively uniform load distribution across the group; whereas a conventional linear elastic analysis (which ignored the influence of adjacent embankment settlement) gave axial forces in the front row of piles that were nearly three times larger than the rear row.

57.2.5 Methods to reduce NSF

For pre-cast driven piles the pile may be coated with a low-friction material (e.g. bitumen) over the depth where NSF is anticipated, thus reducing Q_{s-}. However, there is a risk that the coating will be removed as the pile is driven. Even if the coating is undamaged, the NSF over this length will not be zero.

For bored piles a double casing can be used over the upper portion of the pile. The outer casing is directly in contact with the settling soil, but not the inner casing or pile. The inner casing allows the pile to be cast *in situ* through the full depth. However, the piles are then laterally unsupported over this length, which may have implications for buckling under an axial load or there may be a significant loss of capacity to carry a lateral load.

57.3 Heave-induced tension

57.3.1 Single piles

Heave-induced tension (HIT) occurs when piles are constructed in clay soil beneath a deep excavation, where upward movement is ongoing, or in an expansive swelling soil (e.g. due to changes in the water table or saturation) – see section 57.3.3 below.

Here, the main issue is that if there is insufficient load at the head of the pile tension will occur, with consequent cracking of the concrete if there is insufficient tensile reinforcement. **Figure 57.4(a)** shows the situation with no load at the pile head. The top portion of the pile is 'pulled 'upwards by what would conventionally be 'positive' skin friction, and vertical equilibrium is preserved by an equal amount of 'negative' skin friction anchoring the pile into the soil at greater depth, which is heaving less. There is no load at the base of the pile which is being 'lifted'. Since soil strength tends to increase with depth the neutral point is likely to be approximately at a depth of between half and two-thirds of the pile length. For the situation shown in **Figure 57.4(a)**:

$$Q_{s+} = Q_{s-} = \frac{Q_s}{2}. \tag{57.4}$$

Therefore a pragmatic safeguard against tensile failure of piles due to HIT is to provide sufficient tensile reinforcement to carry a load equal to half the shaft capacity through the full length.

As shown in **Figure 57.4(b)**, if there is some compressive load Q_h at the head of the pile, tension will still occur near the bottom of the pile unless $Q_h > Q_s$. However, the position of the neutral point is lower and the heave of the pile is reduced.

For clay soils beneath an excavation, the strength and effective stresses will reduce as swelling occurs. Hence, if the pile is installed before excavation then there should be a check on the heave-induced tension for both short- and long-term conditions to ensure the critical condition is identified. Typically, short-term conditions will be critical for structural design, and either short- or long-term conditions may be critical for serviceability (i.e. upward movement) of the pile, depending upon the magnitude of the compressive load at the pile head. If the pile is installed after excavation, the structural design checks would be similar to those above; however, upward movement would solely be due to time-dependent movement (the total minus undrained movement).

A free-field profile of soil heave versus depth can be calculated, as in section 57.2, based on a reduction in the effective

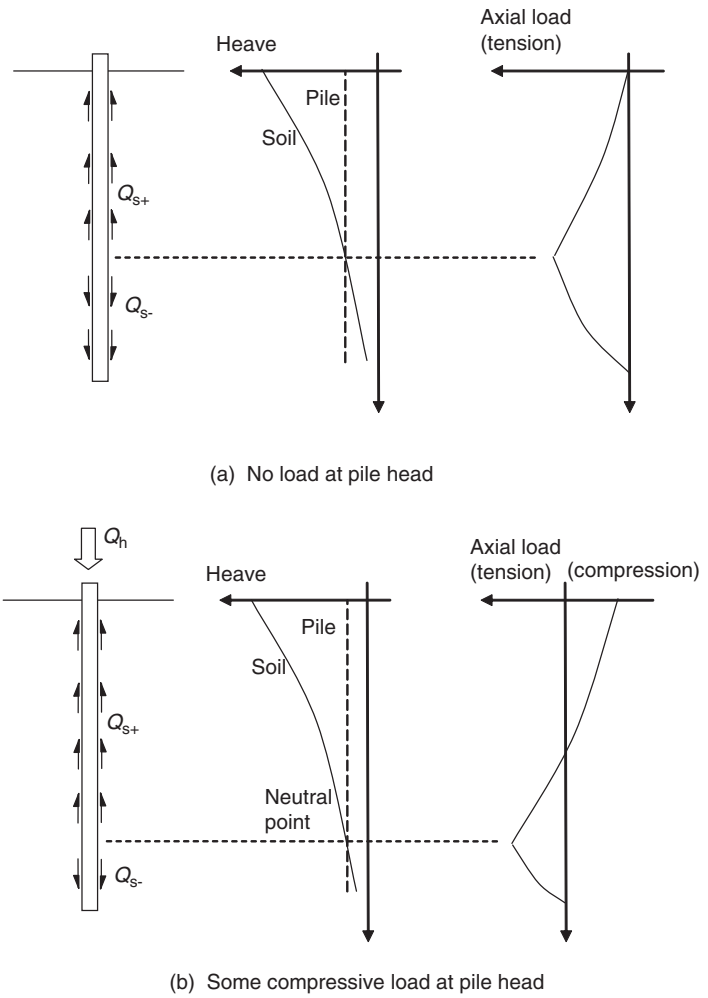


Figure 57.4 Heave-induced tension

stress. The soil swelling curve will tend to be strongly nonlinear, and this is an important consideration for making reasonable estimates of the soil–structure interaction.

57.3.2 Pile groups and floor slabs

Where a floor slab will be cast near ground level, it is most likely that it will be designed to be suspended from the pile caps, and cast onto a void former, which collapses at a small load on the underside of the floor slab as the soil heaves beneath. Where the slab is not isolated from movement and uplift pressure of the soil beneath, the connection to the piles would be required to carry tension, and the slab would need to be designed for large hogging bending moments induced by the soil uplift pressure.

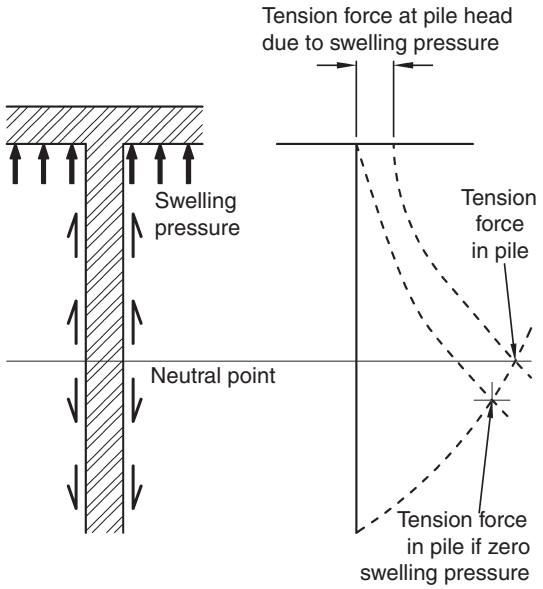
Where a pile cap is used for a pile group, the underside of the pile cap will also be subject to uplift pressures unless it is isolated from the underlying soil using a void former. Piles near the centre of the group will probably be 'sheltered' from the effect of soil heave to some extent, whereas HIT on the outer piles will tend to cause uplift of the pile cap. If there is an insufficient external load on the pile cap (including its weight), there may be significant tension acting at the heads of the central piles as the cap is lifted by HIT on the outer piles. This is essentially a reversal of the situation described above for pile groups subject to NSF, where the outer piles are potentially subject to tension. An important difference between piles subject to unrestrained swelling (i.e. a suspended base slab) and restrained swelling (a base slab or pile cap in contact with the ground) is the development of shaft friction with depth. For unrestrained swelling, the relative pile–soil displacement (and shaft resistance) can develop near the pile head and move progressively downwards. In contrast, for restrained swelling, shaft resistance will develop at depth and move upwards, since the base slab or pile cap will inhibit the relative soil–pile displacement near the pile head.

Soil–structure interaction will develop between the slab, piles and swelling soil in contact with the floor or base slab. The swelling pressure will have two components: the soil 'effective' pressure plus the groundwater pressure. The magnitude of the soil 'effective' swelling pressure, and the tensile forces and displacement of the piles will depend on the stiffness of the structure, the pile length and the swelling characteristics of the soil. **Figure 57.5** illustrates the inter-relationship between some of these factors. The swelling pressure will lead to an increase in the tensile force in the pile, **Figure 57.5(a)**, the neutral point will move upwards and the upward displacement of the pile will increase. As the upward displacement of the structure increases, the swelling pressure on the slab will reduce, **Figure 57.5(b)**. Therefore, a relatively stiff structure (B) will move less, S_B, but larger swelling pressures P_B, will be 'locked in', compared with a more flexible structure (A), **Figure 57.5(b)**. Hence, an interactive analysis is needed to assess the equilibrium values of the swelling pressure, the ground and structure displacements and the forces induced in the piles and structure. Irrespective of the 'internal' structural stiffness of the structure, some global upward displacement, *Sg*, will develop as friction develops on the basement walls and piles. Typically this situation will occur for deep basement structures and the 'global' stiffness and displacement of the whole structure needs to be considered, as well as the local interactions between the pile, slab and soil, **Figure 57.5(c)**. It is particularly important to check that the structural strength of connections between the pile, slab and basement walls is adequate. The groundwater pressure is unaffected by the soil–structure interaction, and needs to be added to the effective swelling pressure from the soil. The incorporation of drainage beneath the slab can eliminate the groundwater pressure acting directly on the underside of the slab and reduce the soil swelling displacement, swelling pressures and forces induced in the structure. However, the drainage must be easy to maintain, in order to remain effective throughout the structure's design life.

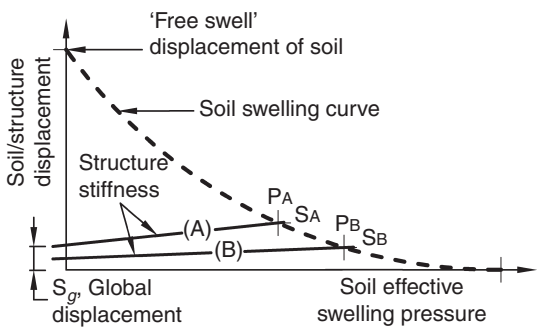
57.3.3 Expansive and collapsible soils

Collapsible and expansive soils are considered in Chapter 32 *Collapsible soils* and Chapter 33 *Expansive soils*, respectively.

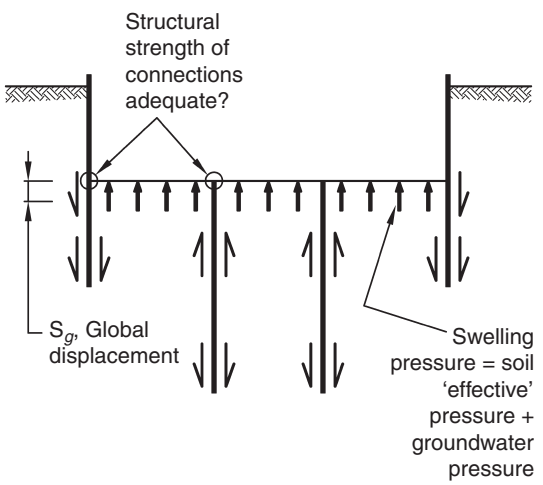
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(a) Swelling pressure and tensile force in pile



(b) Interaction between structure stiffness, soil swelling and swelling pressure



(c) Deep basement, global interaction – swelling pressure/head induced tension

Figure 57.5 Heave-induced tension, floor slab in contact with swelling soil

The behaviour of such soils is very complex, and in certain scenarios it may not be clear which situation will arise (i.e. negative skin friction or heave-induced tension) – see Chapter 53 *Shallow foundations*. It may then be necessary to adopt a robust structural design for the piles, which will cope with both eventualities (NSF and HIT, respectively). Furthermore, the piles will be required to penetrate to sufficient depth where the soil is known to be unaffected by tendencies for expansion or collapse.

57.4 Piles subject to lateral ground movements

57.4.1 Single piles

This form of interaction is most often considered using a horizontal plane normal to the axis of the pile (which is normally vertical), as shown in **Figure 57.6(a)**. The simplest situation is for an 'isolated' single pile where the boundaries are at a large distance compared with the diameter of the pile. The relative pile–soil displacement in the horizontal plane will be referred to as δ. Relative movement most commonly occurs when the pile carries a horizontal load applied at the head and it moves relative to the stationary soil, which resists this movement – see Chapter 54 *Single piles*. However, this chapter considers the reverse situation – where the soil moves and loads the pile. Examples of this type of interaction are considered in section 57.4.2 below.

In the most fundamental consideration of this interaction, it is immaterial how Δ originates, and a horizontal load acting on the pile (often expressed as an equivalent pressure), p, will result as shown in **Figure 57.6(b)**. As is often the case the interaction may initially be idealised as elastic, ultimately reaching a value of plastic yield. However, the actual response shows a more gradual transition between these extremes of behaviour.

Such loading can be particularly damaging, since it generates bending moments and shear forces in the pile, which will therefore require adequate bending capacity. Historically semi-empirical methods have been used to allow estimation of the soil loading or maximum moment directly. However, such methods tend to rely on a very general consideration of behaviour and are inherently limited in their ability to accurately represent specific circumstances. Finite element analysis is now likely to be considered appropriate where a complex and potentially damaging soil–structure interaction problem is identified. However, the complexities of such an approach should not be underestimated (see section 57.4.3 below). A preliminary set of analyses using one, or preferably two, of the available simplified methods should always be carried out first before undertaking complex finite element analyses.

The text below will first consider situations where this type of interaction may occur. As described in section 57.1 the construction sequence, etc. may have a significant impact and 'engineering solutions' may exist to avoid significant lateral interaction effects. This is generally the best course of action, rather than attempting complex analysis. Key references are listed, which may be of use in making simplified preliminary calculations. The incorporation of this type of interaction in a finite element analysis is then briefly considered.

The piles will often be in rows or groups. Some of the references below specifically consider group interaction effects. Progressive structural failure may be an issue for multiple pile rows exposed to an interaction; if the structural failure mechanism (which often involves combined bending and shear) is brittle, then this must be carefully considered and avoided.

57.4.2 Examples of piles subjected to lateral ground movements

57.4.2.1 Piles adjacent to embankments on soft clay

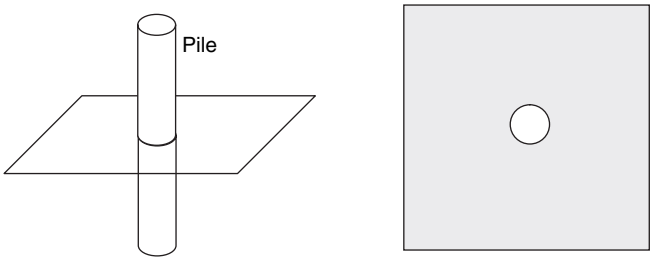
When an embankment is constructed on soft clay or peat the soil beneath tends to move horizontally outwards, rather like toothpaste being squeezed from a tube. As shown in **Figure 57.7**, where piles are situated at the edge of an embankment they will be horizontally loaded by the soft clay, in addition to any structural loading at their head. The effect of a pile-head restraint should be considered – where the pile head is restrained against rotation this reduces the displacement of the pile but, hence, increases the interaction loading from the soft clay. Where an abutment structure is supported by the piles, there may be additional interaction issues associated with the embankment fill 'arching' onto the abutment due to loss of support from the soft clay (Ellis and Springman, 2001).

In this situation there will be an immediate undrained movement, which is specifically associated with construction of the embankment and, thus, the construction sequence is of significant impact. Using a lightweight fill (e.g. expanded polystyrene blocks), or supporting the embankment itself on piles or improved ground (refer to Chapter 59 *Design principles for ground improvement*) may be considered to reduce soil movement. It may be possible to isolate the piles from the moving soil, e.g. using an annulus of soft bentonite clay, or inside an oversized liner. However, the structural design of the pile must then consider the pile to be laterally unsupported by the soil over this height. In general it is preferable to reduce the problem at source, i.e. construct the piles after the embankment has been constructed and allowed to settle, or use lightweight fill or ground improvement of the underlying soil.

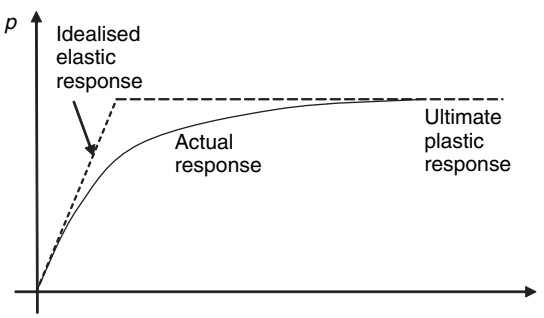
Key references

DeBeer and Wallays (1972) (also summarised in Fleming *et al.*, 2008) give an example of a 'pressure-based method' for the direct estimation of the lateral pressure on piles near an embankment as a fraction of the nominal increase in vertical stress from embankment loading (q). The ratio of q to the undrained shear strength of the soft soil (c_u) is also considered, and many other later methods, see below, also use this ratio. Ultimate bearing failure occurs at a value of (q/c_u) = 5, but the risk of damage to the piles is generally considered to become significant when a value of about 3 is reached.

Springman and Bolton (1990) proposed a method for the simple calculation of soil deformation and, hence, pressure

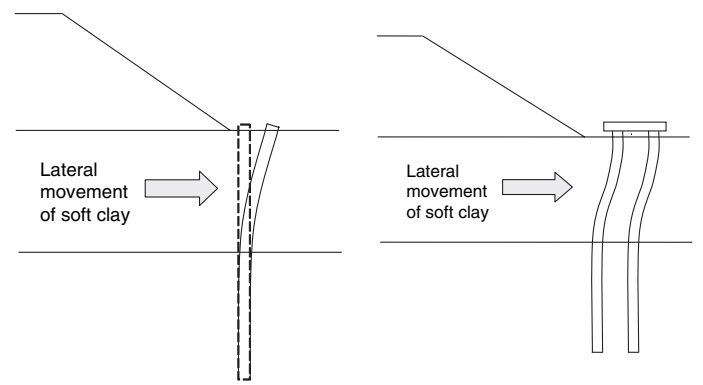


(a) Soil–structure interaction in horizontal plane normal to vertical pile axis

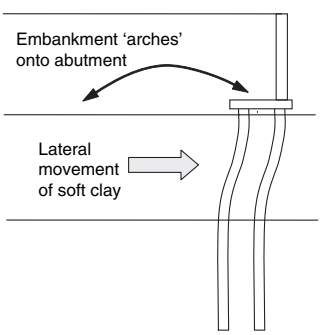


(b) Increase in pile load (p) with pile–soil relative displacement (δ)

Figure 57.6 Fundamental concepts of lateral pile–soil interaction



(a) Free-headed pile (b) Pile head fixed against rotation



(c) Piled bridge abutment

Figure 57.7 Piles adjacent to an embankment on soft clay

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loading due to adjacent loading on a soft clay layer. The method also considers the resulting bending moment distribution in the piles.

Stewart *et al*. (1994) give a useful summary of existing analytical methods at that date. A method for direct but approximate estimation of the maximum pile moment loading is proposed, and the method proposed by Springman and Bolton (1990) is reviewed and modifications are proposed.

Goh *et al*. (1997) give a numerical procedure for the determination of pile pressure and moment loading resulting from known 'free-field' soil movement (which would occur in the absence of the piles). They compare the results with some case studies, and propose an equation for the simple direct estimation of the maximum moment in the piles.

Chen and Poulos (1997) give a boundary element procedure for consideration of pile loading again from known 'free-field' soil movement. They use this approach to produce design charts, which potentially can be used for the 'preliminary design' of piles near embankments or piles used to stabilise slopes (see below). Leroueil *et al*. (1985) present case study information regarding the lateral displacement associated with the construction of an embankment on soft clay.

Ellis and Springman (2001) describe physical model testing and plane strain finite element analyses of full-height piled bridge abutments on soft clay. A mechanism of arching onto the pile cap (**Figure 57.7(c)**) was highlighted for this type of structure. The authors conclude that empirical methods which do not account for specific boundary conditions of the piles are likely to be inadequate for a direct estimate of the pile moment loading in complex situations. However, a simple structural analysis of the pile under the actual boundary conditions and pile pressure load obtained via one of the methods above may be adequate for preliminary design. For more detailed design a finite element approach should be used, and the paper demonstrates that a plane strain analysis can be used provided there is some facility for incorporating lateral interaction between the soil and pile row (see section 57.4.3).

57.4.2.2 Piles near excavations

Where an excavation (e.g. an open excavation or a cantilevered retaining wall) is constructed near a pile, the ground will tend to move towards the excavation, causing interaction loads on the pile and a corresponding deformation (**Figure 57.8**). Again the deformation and load in the pile will be affected if the pile is restrained against rotation or movement at the head.

For a simple cantilevered wall the zone of deformation may be assumed to be an 'active wedge' (**Figure 57.8**). The amount of movement will be reduced (perhaps considerably) if the excavation is braced, and the zone of influence will also be affected.

A plane strain finite element analysis of the wall and pile row can be undertaken (see section 57.4.3). Alternatively, if 'free-field' soil deformations have been estimated, the load on a pile can be established and the bending moment in the pile can be estimated. Long (2001) and Clough and O'Rourke (1990) have published comprehensive summaries of ground movements around deep excavations in a variety of soil types. For excavations in soft clays, the magnitude of the ground movement is very sensitive to the factor of safety against basal instability. Movement of the pile tends to reduce the relative pile–soil displacement and hence the interaction pressure, thus, it is conservative if it is assumed that the pile does not move.

It may be possible to isolate the piles from the moving soil, e.g. inside an oversized liner. However, the structural design of the pile must then consider it to be laterally unsupported by the soil over this height.

Key references

Poulos and Chen (1997) present a method for analysing the response of piles to a nearby excavation, focusing on braced excavations in clay. Design charts for maximum bending moment and deflection of the piles are presented.

57.4.2.3 Piles in unstable slopes

Piles are often deliberately constructed in unstable slopes in an effort to 'dowel' the potential slip (**Figure 57.9(a)**). For reasons of economy and to minimise the use of resources 'discrete' piles are spaced at a distance s along the row, typically 2 to 5 times the pile diameter (d). However, it is critical that the unstable soil loading the piles 'arches' across the gap between adjacent piles and does not 'flow' through it (**Figure 57.9(b)**). Piles constructed in an unstable slope for any other reason will attract similar loading.

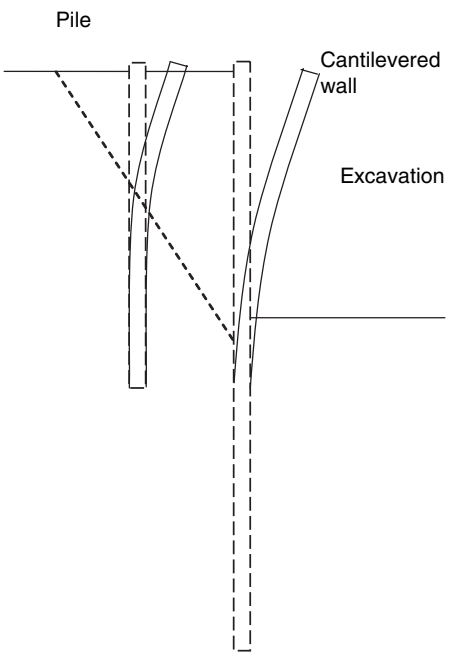


Figure 57.8 Piles near an excavation

There are a number of aspects of any design:

- Establishing the critical slip (where there is not a well-defined existing slip), and required stabilising force to give the desired factor of safety for the slope.
- The location of the piles in the slope, which is likely to be affected by the practicality of construction and the desire to protect infrastructure either at the crest or toe of the slope.
- Choice of spacing ratio (s/d).
- Provision of adequate moment capacity in the piles (noting that the load per pile increases in proportion with s).
- Sufficient embedment of the piles in the underlying stable material to give the required resistance, and so that a deep failure cannot pass beneath the piles.
- Horizontal restraint may be provided at the pile head using a capping beam, which is itself restrained by ground anchors or raking piles. This reduces displacement of the pile head and the maximum moment in the pile.

Note that the use of a 'free-field' soil displacement to determine the loading on the piles (e.g. Chen and Poulos, 1997) is not that common in design. Instead the required stabilising force is normally used to determine the pressure on the piles above the slip.

Key references

Viggiani (1981) gives a simple method for the derivation of the moment in piles by assuming that the loading from unstable and underlying stable material is uniform with depth.

Chen and Poulos (1997) give a boundary element procedure for consideration of pile loading from known 'free-field' soil movement. They use this approach to produce design charts, which potentially can be used for the 'preliminary design' of piles in unstable slopes.

Hayward *et al*. (2000) examine the effect of pile spacing for the stabilisation of a clay cutting using the results of geotechnical centrifuge tests.

Carder and Temporal (2000) give a good overview of the topic and associated references at the time of publication.

Smethurst and Powrie (2007) give a recent case study with monitoring data.

Ellis *et al*. (2010) refer to many recent numerical studies, and give guidance on various generic aspects of design including critical pile spacing.

57.4.2.4 Piles near tunnels

Tunnelling can have a significant effect on the surrounding ground, depending on the tunnelling method and increasing with volume loss. Where existing piled foundations are in the vicinity of a tunnel that is to be constructed there should be an assessment of the effect on the pile (**Figure 57.10**).

Generally speaking construction of a tunnel will cause movement of soil towards the tunnel with consequent relaxation of both vertical and horizontal stresses in the soil (**Figure 57.10**). This will have a negative impact on the pile capacity,

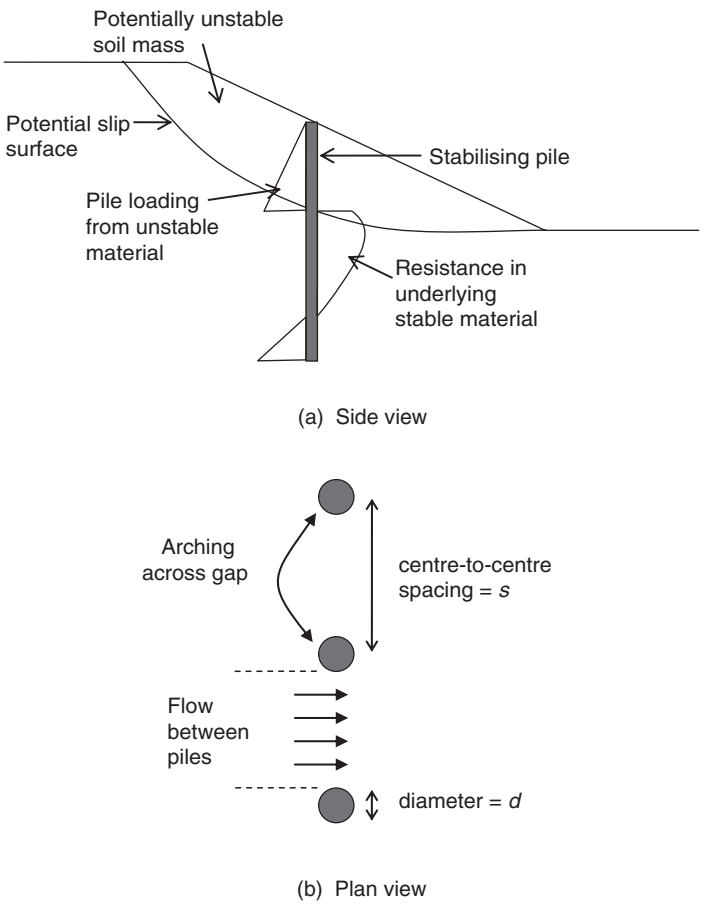


Figure 57.9 Discrete pile row stabilising slope

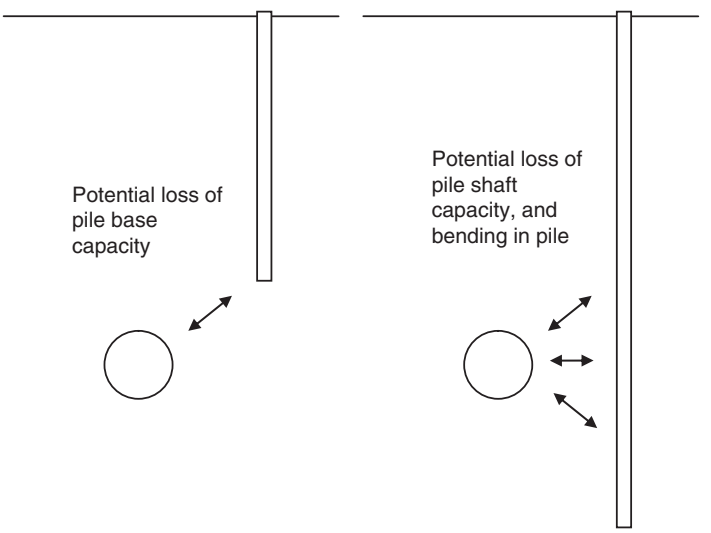


Figure 57.10 Tunnelling near piles

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particularly if the pile relies significantly on the end-bearing capacity and the tunnel is near the toe of the pile. Assuming that the pile is subject to a constant vertical load at the head the reduction in capacity will cause increased settlement. In the extreme this may cause failure of the pile.

Furthermore, lateral movement of the soil towards the pile may cause bending and shear forces in the pile.

Key references

Chen *et al*. (1999) consider the effect of tunnelling-related movement on piles. The effect on the lateral and axial pile response is then used to produce simple design charts (e.g. for settlement at the pile head).

Jacobsz *et al*. (2003) consider the effect of tunnelling on driven piles. Selemetas *et al*. (2005) give a study of the response of full-scale piles to tunnelling, whilst Chung *et al*. (2006) present physical modelling of the pile–tunnel interaction.

57.4.3 Numerical analysis

Early computational methods used bespoke software based on specific analytical approaches. Such methods often rely upon an estimate of the 'free-field' ground movement (in the absence of piles) to derive the interaction pressure on the pile, or estimate this directly. Once this has been done it is still necessary to combine this loading with other conditions of restraint on the pile (both structural and from soil that is not moving) to derive a bending moment distribution. Consequent deformation of the pile tends to reduce the interaction, and thus iteration may be required, or it is conservative to assume that the pile does not move.

More recently finite element analysis is likely to be considered appropriate, and can inherently consider the simultaneous compatibility of pile–soil relative displacement and restraint and the structural loading of the pile. Plane strain modelling will require the use of 'link' elements to model the interaction, and there are other considerations for 'scaling' the three-dimensional pile row to the plane strain equivalent (e.g. Ellis and Springman, 2001). Three-dimensional analysis offers the benefit of removing these complications. However, other significant complications are introduced beyond the increased computation effort required to run the analysis (e.g. Ellis *et al.*, 2010).

It should be emphasised that this type of soil–structure interaction analysis requires extremely complex numerical modelling, and it should not be attempted in normal geotechnical practice.

57.5 Conclusions

Pile-supported structures are often considered to be 'safe', and therefore may not receive the attention they deserve. When piles fail, it is often because they were loaded in unexpected ways. The impact of global ground movements can be severe and numerous failures have occurred, including the recent catastrophic collapse of a building in Shanghai, **Figure 57.1**. Usually these failures are due to a lack of awareness, on the part of designers and constructors, that global ground movements could occur. Fragmentation across different design and construction teams can exacerbate the technical challenges, and therefore the procurement process, managed by client organisations, needs to facilitate proper coordination across any design or construction interfaces. Therefore, the following are particularly important:

(i) Awareness of the situations when significant global ground movements may develop, either during construction or in the long term. This will usually require input by a senior geotechnical engineering specialist at an early stage of the project.

(ii) An appreciation of the interaction that may develop between temporary and permanent works; this may need the temporary works to be carefully controlled or the permanent works design to be modified. Hence, the loading scenarios for design or specifications may need to be tailored to suit specific circumstances.

(iii) An appreciation of the interactions that may develop between existing and new structures or which may develop due to intrinsic site hazards (e.g. non-engineered fills, Chapter 58 *Building on fills,* or due to the site's history or geology).

(iv) As stated earlier, it is usually wise to avoid adverse interactions developing due to global ground movements, e.g. by selecting an appropriate construction sequence (which would then need specifying on drawings, CDM registers, etc.) rather than attempting a complex analysis.

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It is recommended this chapter is read in conjunction with

- Chapter 53 *Shallow foundations*
- Chapter 55 *Pile-group design*
- Chapter 94 *Principles of geotechnical monitoring*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

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Chapter 58
Building on fills

Hilary D. Skinner Donaldson Associates Ltd, London, UK

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This chapter will outline the main issues to consider when building on filled ground, and the references includes further reading relevant to each topic. Successful construction depends not only on the load-carrying characteristics of the fill, but also on the sensitivity of the structure to ground movement. Due to the variability inherent in filled ground, the application of the geotechnical triangle (Chapter 4 *The geotechnical triangle*) is particularly important.

58.1 Introduction

It is often necessary to construct buildings on areas covered by fill, for which the engineering properties are almost always poorer than the parent material. Whilst this chapter will consider the issues related to construction on fills, other issues considered in the design such as sloping ground, should not be neglected (see Chapter 9 *Foundation design decisions*).

Generally, little or no control was exercised over the disposal of non-engineered fills and so they may be highly variable in composition, placement and geometry. All of these provide potential risks to development. This means that the engineering behaviour of non-engineered fills is difficult to predict and that they are likely to afford very variable support to any building or structure constructed upon them.

When construction is carried out on an engineered fill placed for a different development, the investigation must identify whether the fill properties and engineering behaviour are suitable for the development being undertaken. If an engineered fill is to be placed for the development, the choice of materials and placement methodology should take into consideration the requirements of the structure to be supported. The behaviour of the materials below the fill should also be considered.

Because the settlement distribution over the loaded area of a fill, even under uniform loading conditions, may be very irregular, it is often necessary to design so that the total, as well as the differential, movements are restricted or can be withstood without damage. Hence, the structure should either be sufficiently rigid to redistribute the loads and thereby reduce relative settlement, or should be relatively flexible to accommodate them without cracks appearing in the structure. Foundation design, therefore, has to consider the predicted fill–structure interaction. Long continuous structures should be avoided by dividing them into sections with flexible joints passing completely through the structure and foundation. Ordinary pad and strip footings are rarely adequate. Wide reinforced strip footings may be adequate otherwise a reinforced raft is necessary. If a fill is relatively thin and it can be penetrated and overlies a suitable stratum, then a beam and pier or beam and pile foundation may prove an alternative.

This chapter will outline the main issues to consider when building on filled ground, from investigation, characterisation of fill properties, causes of volume change and foundation design.

All construction requires appropriate investigation allied to careful design. In the subsequent sections, only those issues which are different, or require a different approach or interpretation from the 'normal' design process are highlighted.

Section 3 *Problematic soils and their issues* discusses the behaviour and characterisation of fills in depth. A summary is given here.

58.2 Engineering characteristics of fill deposits

Some of the characteristics of a fill relevant to the design of foundations are related to the nature and extent of the deposit, and not just to the material of which it is composed. These should be identified as far as possible by desk studies and confirmed by investigation (See Section 4 *Site investigation* for more on this).

58.2.1 Surface extent and depth

The boundaries of the filled area need to be established reliably. With well-defined infilled excavations, such as old docks, this may be simple, but with other infilled excavations it may be more difficult. The depth of the fill is important in assessing its behaviour, and in evaluating the applicability of various ground treatment techniques and foundation designs.

Abrupt changes in depth may lead to differential settlement. Some old excavations such as docks have vertical sides, and it may be necessary to excavate the sides to a flatter slope to reduce subsequent differential settlement.

58.2.2 Age

The age of a fill deposit can have several different effects on its engineering behaviour:

- *Environment*. An old fill may have already been subjected to many changes in moisture content, water level and loading conditions, which may make it less vulnerable to large movements under future changes.
- *Fill material*. The length of time that has elapsed since fill placement is of major importance for fills containing a significant proportion of biodegradable materials where settlement is largely related to decomposition.
- *Deposition*. The rate of long-term settlement under self-weight is related to the time that has elapsed since fill placement. Other time-related phenomena affecting a fill deposit include desiccation and weathering leading to the formation of a surface crust.

58.2.3 Method of placement

The mode of formation will have a major influence on subsequent behaviour. Fills may have been placed in thin layers and heavily compacted or they may have been end-tipped in high lifts in dry conditions or into standing water. The method of placement affects the density of the fill and the homogeneity of the deposit. Many of the problems with non-engineered fills are related to their heterogeneity. Depending on the method of placement and the degree of control exercised during placement, there may be variability in materials, density and age. Much of the usefulness of ground treatment (see Chapter 59 *Design principles for ground improvement*) techniques is related to reducing this variability. **Figure 58.1** shows an example of some control exercised in fill placement.

Hydraulic placement of materials generally produces fills with a high moisture content in a relatively loose or soft condition. Segregation of different particle sizes is likely.

58.2.4 Groundwater level

The groundwater level within a fill is controlled by a variety of factors such as the surrounding hydrogeology, whether there is an impermeable crust on the surface of the fill, the permeability of the fill or whether there is fill drainage.

The current level of the water table, the amount by which it has fluctuated in the past and the possibility of future fluctuations may have important consequences. A rise in the water level that inundates a fill for the first time may cause collapse compression in the fill. Lowering the water table can cause settlement due to the increase in effective stress within the fill.



Figure 58.1 Placement of fill in deep lifts

58.3 Investigation of fills

Non-engineered fills are difficult to investigate – their variability is a key characteristic that can be difficult to quantify. Engineered fills should have lesser variability, but this should be confirmed by records and intrusive investigation. In addition, even when fills can be penetrated, fill properties are often altered dramatically by the investigation process and standard correlations based on natural soils are rarely appropriate. Given the challenges in identification and characterisation, it is vital that a good desk study identifies what might be present and that the engineer sees the resulting material, as descriptions may not convey the characteristic of the material. As investigation necessarily only tests properties in the short term, it does not give a great deal of indication of how properties or settlements may change over time (see also Section 4 *Site investigation*).

58.3.1 Desk studies and walkovers

A key component in the investigation of fills is the desk study. Possible areas of filled ground can be identified by previous site or local land use and from British Geological Survey (BGS) mapping (see Chapter 32 *Collapsible soils*). As it is unlikely that fill has been transported a significant distance, local arisings can be identified. Types of fill generated by different processes are discussed in Chapter 34 *Non-engineered fills*. Possible contamination arising from processes should also be identified; a good source is the Department of the Environment (DoE) industry profiles available online at the Environment Agency website. The depth of infilled ground may be discerned from the evolution of the site, and also from knowledge of the processes that may result in a large depth of non-engineered ground (or from deep holes!), i.e. opencast mining, deep mining, quarrying or landfilling. Processes such as landscaping usually result in shallower depths of fill.

A walkover may identify regular patterns of hummocks or hills indicating a man-made origin, variations in vegetation or significant areas of ‘empty space’ near built-up areas. A combination of desk study and walkover may also help identify areas of infilled basements.

The investigation of the site and local area is a key component in identifying the likely presence, type and history of the fill as well as other site issues requiring investigation and design. This allows key risks to the development to be identified early and the scope of intrusive investigation defined.

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58.3.2 Non-intrusive investigation

Some geophysical techniques may aid the identification of the depth of fill where significant differences in stiffness, density or water content of the filled and natural ground exist (see Chapter 45 *Geophysical exploration and remote sensing*). Before specifying the technique, it is worth noting the change in properties that is detectable by each – a change in stiffness for example and finding out whether there may be a sufficient change in the property to enable the technique to locate the features required. In each case some form of intrusive confirmation is recommended.

58.3.3 Intrusive investigation

The techniques available to the engineer are likely to be:

58.3.3.1 Trial pits

For shallow fills a key element is simply digging trial pits. This will enable a good visual description (the number of trial pits required depends on how variable the fill may be and the type of construction required). Bulk disturbed samples will enable basic characterisation of the components by description, particle size, moisture content and Atterberg limits as appropriate. The visual characteristics of deposition may also be visible, such as layered or very variable. Some chemical testing may be required.

Depending on construction requirements, shallow *in situ* tests such as plate or skip tests may be a useful way of characterising the bearing capacity and immediate settlement. Care must be exercised that a sufficient volume of fill will be tested, and a number of tests may be necessary to assess variability. *In situ* CBR tests must be analysed with particular care, as the size of test is small. If the fill is amenable to treatment, further testing on samples is helpful; if mixing is a possible treatment then consider compaction testing and chemical testing.

58.3.3.2 Window samples (WS) and dynamic probing (DP)

Dynamically driven window samplers may penetrate fills to a few metres, although they will refuse on large or flat hard obstructions. The samples obtained will be small; hence, they may not be representative of larger particles, and will be disturbed by the driving process. It is likely that a higher density will be retrieved and some particle breakdown will be caused.

Information from such samples, whether descriptive or a classification, should be interpreted with care.

Dynamic probing may be one of the only tests available. A dynamically driven cone is used to penetrate the fill and the number of blows per 100 mm step is recorded. Some indication of density, with a similar caveat on obstructions, can be obtained. A number of test profiles can give an idea of the variability and help refine additional test locations.

58.3.3.3 Deep *in situ* tests

Deep *in situ* tests can be divided into those involving pushing (CPT, CPTU), drilling (self-boring pressure meter) or carried out from a borehole (DMT, PMT, SPT). For more information see Chapter 48 *Geo-environmental testing*.

Few CPT, correlations are relevant for fills and so this form of profiling and testing should be used with care. CPTs are also liable to refuse on or be damaged by obstructions and so are rarely used. Where the fill is similar to a natural soil (particularly some granular fills) it is, however, a useful tool and can readily identify both variability and improvement due to treatments such as dynamic compaction.

Pressure meter and dilatometer testing is also rarely carried out, primarily because the sample size tested is not representative of a highly variable deposit. The cost of a large number of tests required allied to the potential for remoulding caused by the drilling process means they are often inappropriate.

SPT tests are often carried out where boreholes can be drilled, and give some indication of the strength of the fill as altered by the drilling and testing process. Due to the small sample size and lack of appropriate correlations these tests should be analysed with care. However, SPT testing may be the only data that can be readily obtained.

58.3.3.4 Drilling and sampling

Cable percussion (CP) drilling is likely to break down and densify fills, where water is added during the drilling process this will often aid this breakdown. Samples retrieved from CP boreholes should be analysed and described with care and are likely to be disturbed.

Rotary drilling may retrieve little material; where it does these will have been altered by the drilling and the drilling fluid.

Samples are very unlikely to be suitable for testing for strength or stiffness.

58.3.3.5 Monitoring

Monitoring of water levels, gas and also long-term settlement are important tools for understanding the current state and future behaviour of a fill; refer to section 58.6.4 of this chapter.

58.3.3.6 Trials

Where treatment is to be carried out that addresses deep-fill behaviour an initial trial area in which a greater density of monitoring is carried out is worthwhile. This may well lead to optimisation in the treatment method or increased confidence in its efficacy.

In summary, when investigating a fill, routine methods for drilling and testing are often not useful in determining the behaviour or design parameters. A trial pit to determine the nature of the material (and care is needed to ensure that the material sampled and viewed is representative), backed up by a good desk study and monitoring information is often the most useful source of information. Some information on boundaries can usually be supplemented by direct investigation. The properties that can be identified by intrusive investigation are listed in **Table 58.1**.

Information from case histories on similar fills or large-scale tests may be very useful. Again, care should be taken that the test is representative of the scale and type of behaviour anticipated. A small short-term load test on a fill where self-weight settlement or creep is still occurring is not likely to reveal the

	Disturbed Samples				*In situ* testing		Shallow load testing
Property	**Trial pits**	**WS**	**CP**	**Rotary**	**DP**	**SPT**	**Plate**
Density	x	x	x	x		Some indication	Inferred
Compactive state	Visual indication	x	x	x	x	x	Inferred
Water content	y	y	May be altered by drilling		x	x	x
Particle size	y	y			x	x	x
Plasticity	y	y – fines content may be altered by drilling			x	x	x
Strength	x	x	x	x		Some indication	y depending on load
Stiffness	x	x	x	x	x	x	y
Over-consolidation	x	x	x	x	x	x	x
Variability	Depends on number of locations tested						

y – technique likely to give some indication of the property
x – property unlikely to be determined by the technique

Table 58.1 Properties likely to be identified by intrusive investigation

performance that a large structure built on the fill may experience over a number of years!

58.4 Fill properties

As fills may be difficult to investigate, information from tests on similar materials is vital for the designer.

58.4.1 Index and classification properties

Simple index and classification of properties enable fills to be classified in ways that are relevant to their engineering characteristics. BS 1377-2:1990 (British Standards Institution, 1990) describes classification tests for soils.

58.4.1.1 Particle-size distribution

Particle size has a strong effect on soil behaviour and provides a useful method of classification. Sieving and sedimentation can be used to determine the distribution by particle size. There are basic differences in behaviour between coarse 'granular' soils and fine 'cohesive' soils. Coarse soils tend to have high shear strength and permeability whereas fine soils generally have lower strength and permeability. The percentage of silt- and clay-sized particles (i.e. finer than 0.06 mm) is important; when this percentage is high (usually taken as $Fc > 35\%$), the soil will cease to behave as a coarse soil. In reality, the distinction between 'granular' and 'cohesive' behaviour is a function of other properties in addition to particle size. With a well-graded soil, it may not be immediately obvious whether the behaviour will be that of a coarse or a fine soil. The *BSI Code of Practice* (British Standards Institution, 1999) for site investigations gives helpful advice on this transition.

An uncompacted clay fill composed of lumps of stiff clay may behave more like a granular fill than a cohesive fill when loaded in the absence of water, but if inundated it may undergo collapse compression and subsequently behave like soft, saturated clay fill.

58.4.1.2 Moisture content and degree of saturation

Water is invariably present in fill materials. The moisture content for saturated fills is typically from 20% to 80%. The placement moisture content affects the properties of fills, and the behaviour of clayey fills is very moisture dependent. Moisture content changes can cause volume changes in fills.

58.4.1.3 Density

The density or 'compactness' of a fill has a major influence on its behaviour. This is a function of the method of placement of the fill and its subsequent stress history. The term 'compaction' is used to describe processes in which equipment is used to compress the fill into a smaller volume, increasing its density. Usually, engineering properties are improved with increased density. As solid soil particles and water are virtually incompressible, compaction reduces the percentage of air voids.

58.4.1.4 Coarse fills

For a fairly clean coarse hydraulic fill, the dry density is typically 1.6 Mg/m^3. A well-compacted rockfill might be used in an embankment dam and its porosity will typically be in the range 20% to 25%. In contrast, an uncompacted opencast backfill of similar material has a much higher porosity of about 40%.

58.4.1.5 Fine fills

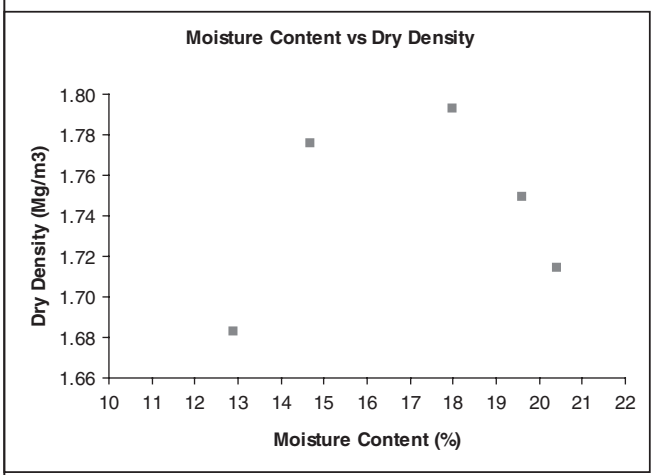
The amount of densification that can be achieved for a given compactive effort on a clayey soil is a function of moisture content. Laboratory compaction test results are usually plotted as the variation of dry density with moisture content. Such plots are a useful way of representing the condition of a partially saturated fill. The density that can be achieved by a particular compactive effort on a clay fill depends on the moisture content of the fill. For a given compactive effort there is a maximum dry density, which is achieved at the optimum

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Moisture Content and Dry Density Relationship Test

In accordance with BS 1377-4:1990

Hole No/Sample Ref:	R2	Test Procedure:	2.5 kg hand hammer
Depth (m):	1.00	Mould Type:	1 litre
Soil Visual Description:	Firm to stiff (medium to high strength) orange patched grey slightly sandy CLAY with frequent flint gravel (8.3% retained on 20mm sieve)		



Natural Moisture Content (%)	18.50

Moisture Content (%)	Dry Density (Mg/m^3)
12.90	1.683
14.70	1.776
18.00	1.793
19.60	1.749
20.40	1.714

Optimum Moisture Content (%)	Maximum Dry Density (Mg/m^3)
16.41	1.794

Note:

Figure 58.2 Proctor test results for a fine-grained fill

moisture content. The expressions 'maximum dry density' and 'optimum moisture content' refer to a specified compaction procedure and can be misleading if taken out of the context of that procedure. With a moisture content that is less dry than the optimum, the specified compaction procedure will result in a fill with larger air voids. For a moisture content that is significantly wetter than the optimum, the specified compaction procedure will produce a fill with a minimum of air voids, typically between 2% and 4% (see **Figure 58.2**).

The fundamental factors listed above demonstrate that the moisture content during placement is key to the density and, hence, engineering performance to be expected.

58.4.1.6 Liquidity and plasticity indices

In this context, the term plasticity describes the response of fine fills (or the finer fractions) to changes in moisture content and is the same property as for natural soils.

Care should be taken when assessing index tests related to the matrix, as the impact of this will be differerent for fills compared with natural materials.

58.4.1.7 Compressibility and stiffness

A wide range of compressibility is possible. Different relationships can be used below and above the optimum moisture content, and the relationships will also depend on the nature of the fill material and whether the grading envelope means that the voids are 'filled' or 'unfilled'. When fills are not close to failure, it is often convenient to describe their behaviour using elastic parameters even though fills generally exhibit nonlinear non-recoverable stress-dependent behaviour (see Charles and Skinner, 2001b).

Conventionally Young's modulus, E, and Poisson's ratio, ν, are used to describe linear elastic behaviour. However, these parameters have little direct relevance to the stress conditions that generally pertain to fills. In practice, the settlement for many fills is closely related to one-dimensional compression, and the constrained modulus, D, is of more direct application.

Typically for non-engineered fills it may be assumed that $D = 1.3E$ to $1.8E$.

When a uniform load is applied over a wide area of the surface of a fill, the resulting compression is largely one dimensional. An application of load will produce an immediate compression in a partially saturated fill. Effective stress is not a simple concept for partially saturated soil and, for most practical purposes, compressibility can best be described by a constrained modulus expressed in terms of total stress. The constrained modulus is the ratio of the increment of applied total vertical stress, $\Delta\sigma_v$, to the increment of vertical strain, $\Delta\varepsilon_v$, which it produces under drained conditions:

$$D = \Delta\sigma_v/\Delta\varepsilon_v. \tag{58.1}$$

It should be noted that $D = 1/m_v$.

The constrained modulus can be measured in oedometer tests, but is usually not representative of *in situ* behaviour because of the small test size (**Table 58.2**).

Fill	D_{sec} (MPa)	ε_v (%)
Sandstone rockfill (I_D = 0.8)	12	0.83
Sandstone rockfill (I_D = 0.5)	6	1.67
Colliery spoil (compacted)	6	1.67
Colliery spoil (uncompacted)	3	3.3
Clay fill (I_p = 15%, I_L = 0.1)	5	20

Table 58.2 Typical values of the constrained modulus (D_{sec}) and strain for vertical stress increments of 100 kPa (from 30 kPa)

58.4.2 Time dependency for fine fills

A fundamental difference between the deformation behaviour of fine fills, such as a clay fill, and coarse fills, such as a sand fill or a rockfill, is due to permeability.

Time-dependent movement depends on the nature and thickness of the fill, the method of placing and the nature of the underlying ground, especially the groundwater conditions. The best materials in this respect are obviously well-graded, hard and granular. Fills containing a large proportion of fine material, by contrast, may take a long time to settle. Similarly, old fills and those placed over low-lying areas of compressible or weak strata should be considered unsuitable unless tests demonstrate otherwise or the structure can be designed for low bearing capacity and irregular settlement. Frequently, poorly compacted old fills continue to settle for years due to secondary consolidation. Mixed fills that contain materials liable to decay, which may leave voids or involve a risk of spontaneous combustion, afford very variable support and such sites should generally be avoided. Some materials such as ashes and industrial wastes may contain sulfate and other products that are potentially aggressive to concrete.

Generally, for uncompacted materials, rockfills will settle about 2.5% of their thickness, sandy fills about 5% and cohesive materials around 10%. The rate of settlement decreases with time, but in some cases it may take 10 to 20 years before movements are reduced within tolerable limits for building foundations. In coarse-grained fills most of the movement generally occurs in the early years after the completion of filling and, often, after five years, settlements are small.

58.4.3 Strength

58.4.3.1 Undrained shear strength

The undrained behaviour is of practical importance for saturated clay fills when applied loads change faster than the induced pore pressure can dissipate. The undrained shear strength (c_u) is a function of moisture content, fabric and effective stresses. Plastic silt and clay hydraulic fills are cohesive and can be characterised by a profile of undrained shear strength versus depth. With a high water table, effective stresses in a hydraulic fill will be low. Some desiccation is likely to occur at the surface of the fill, forming a stiff crust.

58.4.3.2 Drained shear strength

Non-engineered fills are likely to be in a loose condition. Consequently, their strength may be close to the constant volume angle of shearing resistance, $\varphi'cv$. For a granular fill this angle of shearing resistance is usually similar to the angle of repose. If instability should occur, loose fill materials may be vulnerable to liquefaction and flow slides.

Box 58.1 Example: Colliery spoil

The shear strength parameters of a coarse discard do not exhibit any systematic variation with depth in a spoil heap, and so are not related to age. This suggests that a coarse discard is not seriously affected by weathering. As far as the effective shear strength of a coarse discard is concerned, the angle of shearing resistance usually varies from 25° to 45°. The angle of shearing resistance and, therefore, the strength, increases in spoil that has been burnt. With an increasing content of fine coal, the angle of shearing resistance is reduced. Also, as the clay mineral content in the spoil increases, so its shear strength decreases.

The shear strength of a discard within a spoil heap and, therefore, its stability, is dependent upon the pore water pressure developed within it. Pore water pressures in spoil heaps may develop as a result of the increasing weight of material added during construction or by seepage through the heap by natural drainage. A high pore water pressure is usually associated with fine-grained materials that have a low permeability and high moisture content. Thus, the relationship between permeability and the build-up of pore water pressure is crucial. In fact, in soils with a coefficient of permeability of less than 5×10^{-9} m s^{-1} there is little significant dissipation of the pore water pressure, whilst above 5×10^{-7} m s^{-1} the pore water pressure is generally completely dissipated. The permeability of a colliery discard depends primarily upon its grading and its degree of compaction. It tends to vary between 1×10^{-4} and 5×10^{-8} m s^{-1}, depending upon the amount of degradation in size that has occurred.

58.4.4 Permeability

A number of important effects and processes are influenced by water movement and permeability, including the following:

- the rate of dissipation of excess pore pressure and the associated primary consolidation of saturated clay fills;
- the influence of undrained or drained behaviour;
- collapse compression of loose unsaturated fills;
- the liquefaction of saturated fills;
- the loss of ground due to erosion of fills.

Uncontrolled seepage can cause the migration of fine particles out of the fill or into the pores of a coarser soil. Where water flows out of a fill, local instability may occur at the exit point. If water is flowing through a fill, which is susceptible to erosion, then erosion can be prevented or controlled by protection of the fill with adequate filters, which will halt the loss of material.

58.5 Volume changes in fills

One of the most important assessments of the suitability of any fill for building purposes is of the likely volume changes within the fill, which could be observed as settlement, heave or lateral movements. Changes can occur as a result of the

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passage of time or in response to changes in stress or environmental conditions.

58.5.1 Self-weight

Stress changes caused by placement of fill over a wide area differ from the stress changes due to building foundations, which usually apply loads to relatively small areas. The wide extent of fill placement has the following consequences:

- loading and compression behaviour of the fill is largely one-dimensional;
- the full depth of the existing fill may be affected by the loading caused by the extra fill;
- underlying natural strata may be significantly loaded by the fill.

Similar conclusions hold for removal of layers of fill, although the stiffness properties may be very different on unloading, and stiffness is often significantly greater.

Movement caused by settlement or heave of the fill occurs in two stages:

- 'immediate' movement as the applied stress increases or decreases;
- time-dependent movement after the change in applied stress.

58.5.1.1 Movement during and shortly after fill placement

Any immediate compression occurring during an earthmoving operation before construction on the fill has no practical effect on the structures.

As extra fill is placed, most of the resulting compression of a coarse fill occurs immediately. The one-dimensional compression behaviour of coarse granular fills can be described in terms of the constrained modulus, D, which depends on the fill type, placement conditions including initial density and moisture content, the stress level and stress history. In contrast, the behaviour of a saturated or almost saturated clay fill, when extra fill is placed, differs considerably. The load will initially induce an excess pore pressure in the clay fill, and there is little immediate settlement. There may be some immediate settlement resulting from compression of any remaining air voids. This is followed by primary consolidation as the pore pressure dissipates. Any investigation should identify whether primary consolidation is still occurring. Often this is identified by changes in pore pressure.

58.5.1.2 Long-term movement

When movements due to the self-weight of the fill occur subsequent to building on the fill, damage to structures may result. The magnitude and rate of movement subsequent to fill placement are therefore of major interest.

For most fills, there is an approximately linear relationship between creep compression and the logarithm of time. This is often expressed as

$$\alpha = \Delta s/[H \log (t_2/t_1)] \qquad (58.2)$$

Fill type	Compaction	α (%)[1]	Case history[2]
Sandy gravel fill	Heavy vibrating roller	$0.04\sigma'_v$	Megget
Mudstone fill	Heavy vibrating roller	$0.12\sigma'_v$	Brianne
Sandstone or mudstone rockfill	Heavy vibrating roller	$0.13\sigma'_v$	Scammonden
Sandstone or mudstone rockfill	No systematic compaction	0.9–1.5	8
Stiff clay fill	Heavy systematic compaction	0.5	2

[1] σ'_v in MPa
[2] The first three examples refer to case histories of embankment dam construction: Megget (Penman and Charles, 1985a), Brianne (Penman and Charles, 1973) and Scammonden (Penman *et al.*, 1971).

Table 58.3 Creep compression rate parameter α for engineered and non-engineered fills
Data reproduced from Charles and Watts (2001)

where compression $\Delta s/H$ of a fill of height H occurs between times t_2 and t_1.

Some of the results reported in Charles and Watts (2001) are given in **Table 58.3**.

58.5.2 Change in applied stress due to weight of buildings

Whereas fill placement is usually over a wide area, the weight of buildings will generally be concentrated onto relatively small foundation areas. Where the foundation size is small, stresses will be significantly increased over only a relatively shallow depth immediately beneath the footing. In any evaluation of ground deformations caused by the weight of a building, it is important to have some knowledge of the stress distribution below the foundation, because in predicting settlement it is necessary to know to what depth the foundation will effectively stress the fill. The stress distribution below foundations is discussed in Chapter 53 *Shallow foundations*. Furthermore, for a foundation load of 50 kN/m, the ground is significantly stressed only to a depth of 2 m to 3 m.

Apart from the immediate settlement of clay fills under undrained conditions, most movement due to the weight of structures is attributable to a volume reduction resulting from an increase in effective stress. Movement occurs both during and after construction. If movements are very large, structural damage can result from movements during construction; however, these can often be built out and may cause little damage. However, significant settlement occurring after construction can seriously damage a structure.

In addition, some time-dependent strains will occur under a constant applied foundation load:

- For coarse fills, settlement will be caused by creep movements occurring at constant effective stress.
- For fine fills, settlement will be caused by primary consolidation as the pore pressure induced by construction loading slowly dissipates, followed by secondary compression at constant effective stress.

Long-term settlement due to constant imposed foundation loads often shows a linear relationship with the logarithm of time since construction, analogous to creep settlement arising from self-weight of the fill.

58.5.3 Change in water content

58.5.3.1 Change in groundwater level

The effective stress within a fill deposit depends strongly on the groundwater level. A rise in the groundwater level reduces effective stress, and a fall in the groundwater level increases effective stress. The smaller the particle density of the fill, the greater the percentage change in effective stress will be caused by a given change in the groundwater level. The magnitude of the movements caused by these changes in effective stress will depend on the stiffness of the fill. The changes will occur over large areas and so deformations will be largely one-dimensional. The movements can be described by a constrained modulus appropriate to the particular stress change. As the change in water level will usually result in unloading or reloading of the fill, the stiffness may be much larger than that operative during the first loading of the fill. The rate at which the groundwater level within a fill adjusts to any changes in external constraints is a function of the fill permeability. When an unsaturated clay fill is subjected to a rise in groundwater, the change may be rapid at first. Following softening of the clay lumps, often accompanied by large settlement, further changes are slowed by the significantly lowered permeability.

58.5.3.2 Swelling and shrinkage

The volume change of saturated clay fill, particularly high-plasticity clay, is moisture-content dependent. Drying may cause considerable shrinkage. As the fill dries and shrinks, the soil moisture exerts more suction. If water comes into contact with the fill, it is absorbed and the clay swells. These types of movement have caused damage to houses with shallow foundations on natural clay soils, and clay fills are vulnerable to the same processes. Shrinkage can occur through:

- movement of moisture near the ground surface, due to evaporation and recharge;
- the action of the roots of vegetation: the demand for moisture in dry weather increases with the amount of vegetation;
- heating of the ground.

58.5.3.3 Collapse compression

Partially saturated fills may undergo a reduction in volume, known as collapse compression, when inundated. Inundation may occur owing to a rise in groundwater level or by downward migration of water. Most types of partially saturated fill are susceptible to collapse compression under a wide range of applied stress when inundated or wetted for the first time, if they have been placed in a sufficiently loose or dry condition. This phenomenon can occur without any change in applied total stress. When construction is about to take place on such a fill, susceptibility to collapse compression may be the most significant hazard because, where collapse compression occurs after construction has taken place, buildings may be seriously damaged (**Figure 58.3**). This most important facet of fill behaviour is examined in some detail in Charles and Watts (2001).



Figure 58.3 Damage to a building caused by collapse compression

58.5.4 Biodegradation

The age of refuse landfill is important in evaluating the potential for further settlement. Not only has older refuse landfill had more time for organic matter to decompose, it is often an inherently better material with a much higher ash content than more recent refuse. Settlement behaviour is also affected by a number of other factors:

- composition of the waste;
- initial bulk density;
- initial moisture content;
- level of leachate within the fill;
- the time over which the fill was placed.

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Long-term movement can occur as biodegradation causes a reduction in volume, which may be in addition to movement caused by physical creep compression (both can be described by logarithmic compression rate parameters, see Watts and Charles, 1999).

58.5.5 Chemical action

Volume changes, usually expansion, may occur due to chemical reactions within the fill. Loss of material due to dissolution of carbonates could occur in some fills. In addition to a volume change of the waste fill, there may be an interaction between the construction materials and chemically aggressive ground.

Chemical reactions may have been occurring slowly since the waste fill was deposited. However, building development on the site could lead to an acceleration of these reactions as a result of effects connected with construction, such as the following:

- earthmoving may lead to mixing of wastes, which brings different substances into contact and may introduce oxygen into the waste fill;
- leaking drains may cause water to infiltrate the waste fill.

Fills that are particularly vulnerable include:

- sulfate-bearing waste material;
- iron and steel slag;
- pyritic shale;
- alkali waste.

For these materials, expert advice will be required.

58.6 Design issues

In foundation design, it is important to distinguish between the settlement attributable to the weight of the building and settlement attributable to other causes such as self-weight of the fill or collapse compression. The concept of bearing capacity is based on the premise that the settlement resulting from the weight of the building is the critical factor. With small structures on deep fills, almost invariably settlement attributable to causes such as self-weight of the fill or collapse compression will dominate and, consequently, the bearing capacity can be a misleading concept. Foundation design should be based on an assessment of the magnitude of differential movements of the fill subsequent to construction. It is necessary to identify first the cause of settlement. The small scale of many developments makes it difficult to ensure that there is an adequate investigation of the fill.

58.6.1 Bearing capacity

Whilst the performance of building foundations on fills is rarely determined by bearing capacity, it is a design issue. For less settlement-sensitive structures (embankments or gravity retaining walls for example) the bearing capacity is of key importance. The maximum stresses that can be applied to and by fills is also of relevance when considering the pressures on walls (compaction pressure, heave), and also in slope stability calculations.

Standard bearing capacity calculations can be used, but care should be taken with the fill properties used and a range of probable values must be considered (refer to Chapter 53 *Shallow foundations*).

58.6.2 Settlement

Settlement under applied loads, or due to fill settlement caused by other factors, is covered earlier in this chapter. The design must assess which type of settlement effects will apply in the situation under consideration. Following this, the anticipated amount and likely variability of settlement and its distribution (or heave) must be calculated.

Calculation approaches are often driven by the quantity and quality of data on fill properties on the site, and often may take the form of simple estimates. These may be based on elastic analogies (Boussinesq-type stress distributions), on semi-empirical approaches (such as Charles, 1996) or be fully empirical (Burland and Burbidge, 1985).

58.6.3 Geometry

Distortion and damage of buildings are related to the magnitude of the differential settlement across the building rather than the total settlement. Variation in the depth of fill under a building can be a cause of differential settlement and may also result in damaging horizontal movements. There are particularly acute problems associated with differential settlement at the edges of a filled area.

In a backfilled excavation, the severity of the differential settlement will depend on the steepness of the face of the excavation, and many different situations are commonly encountered:

- an old dock with vertical sides;
- a quarry in hard rock with a near vertical face;
- gravel and clay pits with slopes that are not so steep;
- buried changes of the gradient of a slope;
- the sloping base of an excavation.

There is a need to define the areas from which buildings should be excluded because the rate of change in fill thickness gives the potential for unacceptable long-term differential settlement. If unsafe decisions are made and exclusion zones are too small, there may be damage to the building and consequently legal action. On the other hand, an overconservative approach will needlessly sterilise large areas of land and could make many sites uneconomic to develop. In determining the area of land where ground surface movements may be unacceptably large owing to variations in the depth of the fill, consideration needs to be given to both the properties of the fill and the geometry of the fill. Further consideration and prediction of the settlement pattern due to changes in fill depth can be found in Charles and Watts (2001) and Charles and Skinner (2001a).

58.6.4 Monitoring

In assessing the condition of a fill, the performance of a treatment or the condition of a structure it is often helpful to carry out monitoring, which may include:

- surface levelling stations to measure the settlement of the fill surface;
- extensometers (magnet or rod) to measure the settlement of incremental depths of fill;
- standpipe piezometers to measure groundwater levels within the fill (local weather observations are also useful);
- load tests for direct estimation of the settlement produced by a load (**Figure 58.4**).

Such measurements can form one of the most important parts of an investigation of a filled site.

Surface settlement can be monitored by precise optical levelling. The settlement of the levelling stations is due to the vertical compression of the fill, in which the station is installed, together with any movement of the underlying natural ground. Surface levelling stations may be very simple, and are easy to install. The levelling stations should be sufficiently robust to resist damage due to construction traffic and where the stations protrude above the ground surface, it is usually necessary to provide protection such as concrete manhole rings.

(a)



(b)

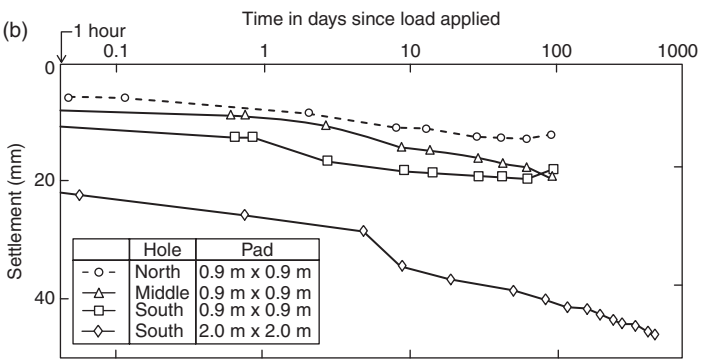


Figure 58.4 (a) Load testing using a skip; (b) load settlement results
Reproduced from Charles and Watts (2001)

Although filled sites can be subject to large settlements, monitoring often takes place over a short period or may be intermittent. Accurate measurements are required to establish the rate of settlement reliably over a relatively short period. Measurements should be made to an accuracy of at least 1 mm. It can be helpful to locate levelling stations in a number of traverses with the stations at quite close intervals. In this way an indication of the likely differential settlement over the area of a building can be obtained.

Settlement at different depths within a deep fill can be measured by installing a magnet extensometer, whereas rod extensometers may be appropriate for shallow fills.

It is also important to measure water levels within a fill. Simple standpipe piezometers can be sealed into boreholes. In low-permeability fills the response time of a standpipe piezometer to a change in piezometric pressure may be too long. Pneumatic or vibrating wire piezometers can be installed in boreholes and will give a much more rapid response.

The period for monitoring depends on the situation. For example, to establish trends in settlement and groundwater levels reliably on a site a minimum period of at least a year will be required, and several years would be preferable. Where a site is being investigated for building development it will rarely be possible to monitor over such long periods. However, even where the period is quite short, it may be very advantageous to perform some monitoring. Monitoring may be used to identify when treatment, or the rate of settlement, has reached some design level. In each case the interval, period and type of monitoring should be linked to the parameter being measured. Further guidance on monitoring is given in Chapters 94 *Principles of geotechnical monitoring* and 95 *Types of geotechnical instrumentation and their usage*.

Post-construction settlement of buildings on fill should be monitored where possible and BRE Digests 343 and 344 (BRE, 1989, BRE and Longworth 1989b) provide guidance on measuring the movements of low-rise buildings.

58.6.5 Shallow foundations

Shallow foundations (see Chapter 53 *Shallow foundations*) are generally less than 3 m deep, so in a shallow fill (<3 m deep) a shallow foundation is all that is required to found the structure on the underlying natural stratum. With medium-depth (3 m to 10 m) and deep (>10 m) fills, either deep foundations into the underlying natural stratum, or a shallow foundation is adopted and the structure is founded on the fill. Where small structures are to be built on medium-depth or deep fills, the only economic solution may involve founding the structure on the fill. It should be ensured that:

- the bearing capacity of the fill is adequate;
- settlement under the working load will not damage the structure;
- settlement due to causes other than building loads will not damage the structure.

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Where fill is vulnerable to significant movement, ground improvement prior to construction on the fill should be considered. Ground treatment methods, when appropriately used should limit and control settlement; they will not normally eliminate settlement (refer to Chapter 59 *Design principles for ground improvement*). Many types of ground treatment will not improve the full depth of a deep (>10 m) fill, but they may stiffen the upper part of the fill sufficiently to prevent excessive differential settlement.

The following basic approaches to shallow foundation design may be adopted:

- Construct shallow foundations on fill that has been sufficiently improved by ground treatment so that deformations are acceptably small.
- Design shallow foundations that are sufficiently stiff to withstand the anticipated deformations in the fill; this may not be difficult with small buildings, but tilt may still be a problem.
- Design shallow foundations and structures that are sufficiently flexible to tolerate the anticipated deformations in the fill.

In practice, solutions may comprise both ground treatment and special foundation design.

Fill that is susceptible to collapse compression on wetting is of particular concern. Several ways of dealing with the problem are listed below and a combination of these approaches may often be the most suitable:

- eliminate susceptibility to collapse by pre-inundation (not generally feasible);
- eliminate or greatly reduce susceptibility to collapse by some other form of ground treatment (depth of effectiveness may be limited);
- prevent inundation of the fill during the life of structure (this often cannot be guaranteed);
- design the foundations and structure to survive collapse movements (difficult if potential movements are large).

Reinforced concrete rafts with edge beams, sometimes termed semi-rafts or stiffened edge rafts, have commonly been used for housing on non-engineered fills. They have usually been designed so that some of the length will act as a cantilever over a potential void. However, where large differential movement may occur, very substantial foundations may be required and tilt may still be unacceptably large. Atkinson (1993) has described typical designs.

The problems associated with shallow foundations for low-rise buildings on filled ground can be minimised by:

- avoiding building across the edges of filled areas where the structure would be partly founded on fill and partly on undisturbed natural ground;
- restricting construction to small units and not building long terraces of houses.

Relative movement between the building and any services entering it, and between various sections of these services, merits careful consideration. If the movements are not likely to be large, the use of short lengths of pipes with flexible connections may be sufficient. In more severe cases it may be necessary to use flexible pipes, or to carry services on piles bearing on a firm stratum beneath the fill. Falls given to drains should be adequate to reduce the risk that settlement will cause backfalls.

58.6.6 Deep foundations

Where a large structure is to be built on shallow or medium-depth fill, the poor load-carrying characteristics of the fill can be circumvented by using piled foundations and a suspended floor. The piles will be designed to transmit the loads applied by the weight of the building down through the fill to a competent stratum underlying the fill (refer to Chapters 54 *Single piles* and 55 *Pile-group design*).

There may be problems where piles hit obstructions. If the depth of fill is not accurately known at a pile location, it may be incorrectly assumed that the pile has reached natural ground.

The piles should be designed for downdrag (negative skin friction) caused by settlement of the fill. The downdrag can be calculated in terms of the effective stress (Burland, 1973) (see Chapter 55 *Pile-group design*). In some situations it may be considered that the downdrag will cause too large a settlement of the pile. The use of a slip coating or a permanent sleeve can reduce its effect, but careful detailing (to ensure effectiveness) is required and complicates pile construction. Often it will be more cost effective to increase the pile length.

Special attention is needed in the design of services that span from the filled ground into buildings founded on piles.

In a fill in which methane gas or other dangerous gases are generated by the decay and decomposition of organic matter, piles could form paths for the escape of the gas.

58.7 Construction on engineered fills

In some cases a fill is placed to support a building or other structure. The characteristics of the site should be considered with regard to the impact of the fill and other applied loads. Ideally the fill should be placed such that simple foundations can be utilised. Sometimes this is not the case, and in these circumstances the nature of the fill should be designed so it has the required properties, e.g. it can be penetrated by piles but will support the ground slab, or can be compacted by vibro after placement.

Where suitable fill material is placed to an appropriate specification under controlled conditions, most of the problems associated with construction on fills should be greatly reduced if not eliminated. The two major considerations are, firstly, the preparation of an appropriate specification, and, secondly, the provision of adequate site supervision to ensure compliance with it.

The required behaviour of the fill should be well defined and it should be possible to make reliable estimates of settlement under working loads including long-term settlement and bearing capacity. However, sometimes problems have been experienced because inappropriate specification or inadequate

control of the placement of engineered fills has led to excessive settlement or heave in service.

While the specified fill may be adequate to support the building, other causes of settlement or ground failure must be considered:

- the stability of the natural ground;
- settlement of the underlying natural ground due to the weight of a fill;
- settlement of an engineered fill due to self-weight;
- movement in an engineered fill due to changes in groundwater level, pore pressure or seasonal changes in moisture content.

The specification of the engineered fill should define acceptable materials for the fill and the method of placement and compaction. Specifications for highway construction have often been used for the specification of fills to support buildings or other structures, but may not be adequate. Trenter and Charles (1998) have produced a model specification for fills for building purposes.

58.8 Summary

Construction on fills is often more challenging than for similar structures on natural soils. However, additional risks can be minimised provided that:

- there is an understanding of the problems associated with the various type of fill and fill deposits;
- the construction is appropriate;
- thorough investigations are carried out;
- the design recognises that movement may not just be related to construction;
- adequate specifications for construction and monitoring are produced;
- construction is carried out to the specifications, with appropriate quality control and feedback.

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58.9.1 Useful websites

British Geological Survey (BGS); www.bgs.ac.uk

Department of the Environment (DOE) Industry Profiles; www.environment-agency.gov.uk/research/planning/33708.aspx

It is recommended this chapter is read in conjunction with

- Chapter 26 *Building response to ground movements*
- Chapter 34 *Non-engineered fills*
- Chapter 32 *Collapsible soils*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 59

Design principles for ground improvement

Robert Essler RD Geotech, Skipton, UK

The use of ground improvement on projects has increased worldwide as its design and application becomes more common. Many major projects now routinely adopt ground improvement and this experience is then disseminated through technical publications, which in turn increases knowledge. Hence, there is a better understanding of how and when such techniques can be incorporated into projects and what associated risks may be attached. In the UK there still remains a perception by some that ground-improvement techniques are difficult to design securely, that the design knowledge is restricted to a small number of engineers or that the techniques are only to be used when there is an unexpected problem on site. It is, therefore, important that young practising engineers have a basic grounding in the design of ground-improvement techniques so that they can recognise at an early stage when such processes could be used to advantage on their projects and have the confidence to propose such schemes. It is equally important that when such a situation arises that the engineer is able to progress the design knowledgably whether by discussion with contracting specialists or by referral to experienced engineers within their organisations or other companies.

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59.1 Introduction

It is a difficult task to summarise the design principles of so many varied processes with different methodologies and applications. It is, therefore, hoped that the techniques set out below and described in outline will allow the engineer to at least understand which processes might be suitable and where to look for further insights to better inform the decision-making process. **Table 59.1** gives some insight into the selection of a ground-improvement process in terms of applicability, performance and risks.

59.1.1 Background

The improvement of ground by injection of grout dates back to at least 1802 (Littlejohn, 2003). This would suggest then that in the 21st century it should be a well-practised, well-understood technique. It is, therefore, surprising that ground improvement, particularly by grouting, is still perceived as a process that is not very well understood and can be difficult to incorporate within a project to provide definable performance. There is no doubt that until the seventies or eighties there was very little technical knowledge in the public domain, but from this time onwards the technical literature has increased substantially. In the USA, the first specialist grouting conference was in 1982 (New Orleans, 1982) and attracted a total of 64 papers. By the third conference (New Orleans, 2003), this number had risen to 127 from 21 countries. Because the UK construction market is significantly smaller than that in Europe or the US, the development of these more experience-driven techniques is not as advanced and limited to specialists who, generally, belong to larger groups working worldwide. In 2008 the probable size of the European ground treatment (grouting) market was in excess of £500 million annually. This should be compared to the UK where the market is probably on average £10–£20 million annually and is distorted when large projects stretch the available resources. Therefore, it is critical to the development of these techniques that they become more understood and, thus, more available as tools for the engineer to employ in design.

59.1.2 Definition of ground improvement

Ground improvement can be defined as the introduction of materials or energy to soils to affect a change in performance of the ground such that it performs more reliably and can be incorporated within the design process.

59.1.3. Types of ground improvement

The major forms of ground improvement discussed within this chapter are as follows:

a. void filling;

b. grouting (permeation, compaction and jet grouting);

c. improvement by densification or replacement (vibrocompaction, dynamic compaction and stone columns);

d. soil mixing.

There are other forms of ground improvement but the processes set out above can be regarded as the major techniques

Technique	Depth capability					Suitable ground					Cement-based grouts			Chemical grouts	
	0–2	2–5	5–10	10–20	20+	Clay	Silt	Sand	Gravel	Rock	Cement PFA	Cement bentonite	Other cement grouts	Silicate	Resin others
Permeation grouting															
Hand lancing	✓	✓				Nil	Nil	✓	✓		✓	✓			
End of casing	✓	✓	✓	✓		Nil	Nil	✓	✓		✓	✓		✓	
Tube A manchette grouting	✓	✓	✓	✓	✓	Nil	<10%	✓	✓		✓	✓		✓	✓
Jet grouting															
Single system	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	Cement/ slag	Not usual	Not usual
Double system	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	Cement/ slag	Not usual	Not usual
Triple system	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	Cement/ slag	Not usual	Not usual
Compaction grouting															
Drilled compaction grouting	✓	✓	✓	✓	✓	✓	✓	✓	✓		✓				
Driven compaction grouting	✓	✓	✓			✓	✓	✓	✓		✓				
Vibrocompaction	✓	✓	✓	✓	✓			✓	✓						
Vibroreplacement	✓	✓	✓	✓	✓	✓ C_u >15 kPa	✓	✓	✓						
Soil mixing															
Dry-soil mixing		✓	✓	✓		✓ C_u <50kPa	✓	✓ (loose)							
Wet-soil mixing		✓	✓	✓	✓	✓	✓	✓	✓						

Table 59.1 Selection of ground-improvement techniques

used for a very high proportion of the ground improvement carried out. Of all these techniques, jet grouting, soil mixing, vibrocompaction and vibroreplacement probably represent most of the ground improvement carried out and should be considered carefully.

59.2 General design principles for ground improvement

The following general principles apply to ground improvement and need to be followed to ensure that the design is appropriate and the application successful.

59.2.1 Ground conditions

Because most ground-improvement techniques require some interaction with the ground, the correct identification of the soils present is of vital importance. It is, therefore, important that adequate site investigation is carried out prior to any ground improvement to ensure that the correct decisions are taken as to which technique is most suitable.

Problem soils to consider carefully would be very soft organic clays, peats and single-sized sands and silts.

As well as carrying out normal laboratory testing, chemical testing is also important as some chemicals can affect the strength development of some binders.

Table 59.2 sets out some soils that need careful investigation to ensure that appropriate ground improvement can be designed. It should be stressed that adequate and thorough ground investigation is always a prerequisite for successful ground improvement.

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Soil type	Issues	Investigation techniques	Ground-improvement techniques
Sands and gravels	Fines content and collapse potential	Obtain representative samples for laboratory testing	All forms of ground improvement
Soft clays	Long-term settlement	Obtain undisturbed samples, carry out CPT testing	Compaction grouting, vibrocompaction and dynamic compaction. Soil mixing and vibroreplacement (insensitive clays, Cu > 15 kPa)
Peat and organic clays	Acidic ground, secondary compression	Chemical testing and if possible large diameter undisturbed sampling	Jet grouting and soil mixing
Karstic rock	Presence of solution features	Geophysics and intrusive surveys	Compaction grouting
Fills	Presence of organic materials, obstructions and chemicals	Chemical testing, geophysics and intrusive surveys	Vibrocompaction or dynamic compaction, compaction grouting

Table 59.2 Ground-related issues for ground improvement

59.2.2 Site trials

It is inevitable that for some applications there will be some doubt as to what degree of improvement might be obtained. Site trials are invaluable for informing the design on a project-specific basis and it is always recommended that some form of trial, whether an initial period of more intensive monitoring, validation at the start of the works or a completely separate advance trial, is carried out. The exact form of a trial will depend on both familiarity with the process within the soils under consideration and equally the criticality of the degree of improvement to the design. The works programme should be developed assuming some period of initial trialling so that due allowance can be made for any resultant changes. Where the ground-improvement performance is critical then advance trials are preferred with sufficient programme time not only for the actual site works but also for the on-site and off-site testing and evaluation.

A good example of this was the jet grouting carried at the Kingston Bridge, Glasgow (Coutts *et al.*, 1992) where the proposed jet grouting was critical to the stabilisation of a major bridge and merited a separate advance trial to confirm the performance. A recent example was the construction of a sandwich wall in Amsterdam where jet grouting was critical to the support of a historic building and a series of trials were carried out well in advance of the actual works (De Wit *et al.*, 2007).

59.2.3 Performance

It is always important when designing ground-improvement works that the performance required is documented to such a degree that it can be validated on site. It is imperative that whatever design parameters are required from the ground-improvement scheme are correctly translated into field performance that can be validated so that the design can be assured. For example, a designer must decide whether the design parameters need to be the average of field results, the minimum, the maximum or even to have some other relationship.

59.2.4 Environmental issues

Environmental issues resulting from ground improvement can either be due to polluting the ground with the cements or chemicals used or equally as a result of changes to the local groundwater hydrogeology. When considering ground-improvement design it is, thus, important to review these potential effects.

59.3 Design principles for void filling

59.3.1 Methods and key issues

Void filling is most commonly employed to deal with abandoned mines or redundant underground structures and involves the drilling of holes on a regular grid and the injection of either a fluid grout or paste.

Generally the design is to firstly create a perimeter barrier around the area to be treated to economise the operation. This is most often formed by drilling larger diameter holes (100–250 mm in diameter) and introducing gravel-sized material washed in with a grout to bind the gravel together.

Figure 59.1 shows how perimeter holes are executed using a mix of gravel and grout, which will not travel and, thus, when set forms a barrier to the more fluid infill grout. Typical perimeter holes would be installed on a 2–3 m grid.

The design of the main works is then executed by drilling holes (50–75 mm in diameter) on a regular grid, typically ranging from 3 m to 6 m, and injecting a cement-based grout or paste. The design of the grid to adopt is based on how open the void is and, thus, how easy it will be for the grout to travel. Usually the works are carried out following the recommendations of CIRIA Special Publication 32 *Construction over Abandoned Mineworkings* (Healy and Head, 1984), which sets out the extent of treatment beyond the structures of interest.

Key design issues relate to the control of any groundwater trapped within the voids, which may need to be pumped out as the void is filled, and ensuring that access shafts are located and treated as these can be a source of future settlement.

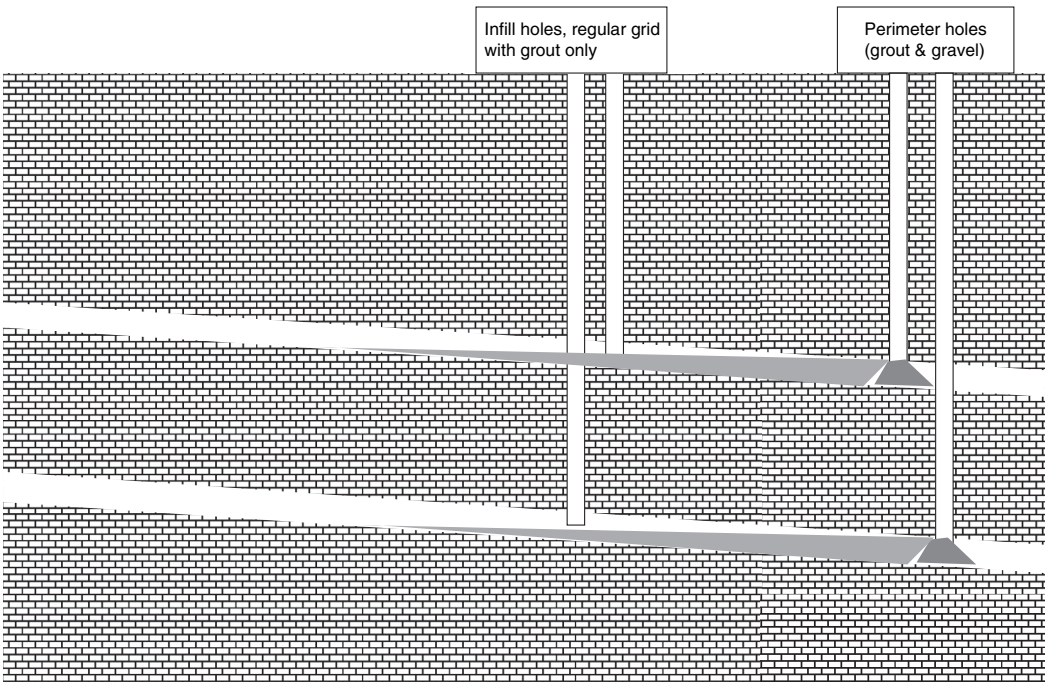


Figure 59.1 Schematic of perimeter and infill grouting for void filling

Groundwater pumped from these voids could be contaminated and should be disposed of appropriately.

Quite often there are underground records of the mines and these can be used to design the works and assess the amount of infilling required and expected drilling depths. Historically a cement and pulverised fuel ash (PFA) mix was used in the proportion of 1 part cement to 10–20 parts PFA. The Environment Agency is now reluctant to sanction the use of PFA as it contains heavy metals and, thus, can itself introduce contaminants into the ground. Replacement grouts can be foam concrete or simple cement and sand mixes.

59.3.2 Design principles

Designing for void filling is essentially observational with probe holes being drilled on a regular grid to locate any voids and subsequently grouting them. The design process should ensure that the depth of drilling is sufficient to investigate all known voids and the plan area treated should include the potential collapse zone of voids at the boundary of the site. Adequate site investigation is, thus, very important. As the resultant void is infilled, the grout is contained and a low strength of less than 5 MPa is usually sufficient.

It is essential good practice when developing a new site to check available coal mining records. These are available from the Mining Records Office in Mansfield.

59.3.3 Execution controls

Execution is controlled by documenting all holes drilled and using this information to develop the underground void profile. The grout or paste injected is controlled by testing its properties against a testing and inspection plan, which sets out limits on properties such as density, viscosity and bleed.

59.3.4 Validation

Validation is usually by the use of the ongoing probe holes within secondary or tertiary grid positions and proof grouted. Where complete filling is important then validation by secondary grouting can be carried out using a more fluid grout and higher pressures.

59.3.5 Typical performance obtained

Generally about 95% of the void can be expected to be filled as some *in situ* bleed can take place. Filler strengths have a wide range depending on individual project design. Historically, void filling of mine workings is regarded as a low-tech solution and there are very few instances of problems arising from this type of work. Care should be taken, however, when the void being infilled is of a different type such as an abandoned tunnel within an urban environment as performance here can be more critical and post pressure grouting may be required. Where the infilled void is likely to be loaded by subsequent construction, the grout properties need to be considered in more detail as higher strengths may be required.

59.4 Design principles for compaction grouting

59.4.1 Methods and key issues

Compaction grouting, probably originating in the United States, is most commonly employed to increase the relative

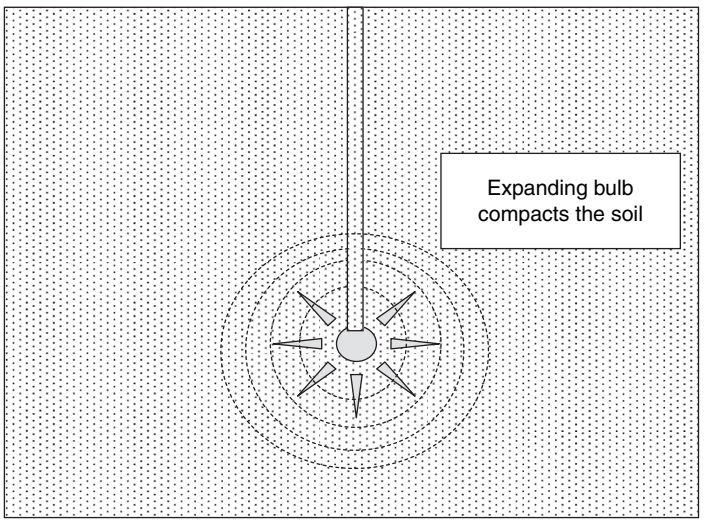


Figure 59.2 Compaction grouting

density of granular materials to improve stiffness or reduce future settlement liability. Compaction grouting is essentially the injection of a low-slump (typically 25–100 mm) grout such that an expanding frictional bulb forms. **Figure 59.2** shows the schematic effect of the technique. This expansion causes deformation and densification around it and ultimately improves the ground. This expansion will continue until the ground has densified sufficiently to resist the expanding pressure. This is observed during the process of grouting through the increasing injection pressure and also by ground heave. If the slump is relatively high then a hybrid form of grouting is used where both expansion and hydro-fracture occur.

The design is, thus, based on expanding cavities within the ground until a maximum relative density is achieved. The design process generally considers the improvement midway between injection points as this will be the least improved. In the UK compaction grouting is mainly restricted to ground improvement where neither waterproofing or underpinning is required. It is sometimes used for settlement control but as control is more difficult than the use of fracture grouting, it is not favoured. Compaction grouting, on the other hand, is commonly used in the United States for settlement correction, ground improvement and seismic risk mitigation, where it has a long history of success. Typical uses in the UK are restricted to grouting to improve made ground or solution features formed within limestone or chalk. Compaction grouting has the advantage that the compactive effort coupled with the associated high pressures mean that a wider hole spacing can be used than for permeation grouting and, thus, it can be more economical in improving granular soils.

A secondary benefit when using compaction grouting to stabilise solution features is that the improvement can be achieved without the introduction of significant volumes of fluid grout or water, which could further destabilise the feature. For historic or domestic properties, there is also an advantage that the original foundation support can be left in place without significant rebuilding as the soil itself is improved and no new load carrying capacity is required.

The technique is most appropriate for granular soils where the compactive effort is greatest. In silts the technique appears to work well and assists in forcing water from the silt and increasing strength. In clays the process is likely to cause significant pore water pressure response, which although causing improvement to the clay in the long term can result in settlement. It is, thus, unusual to apply compaction grouting in clays unless they exist as thin layers or bands within the main body of improvement.

Historically, compaction grout is a mix of cement, bentonite and sand with an approximate water solids ratio of 0.1 to 0.2 by weight, designed to be pumpable but having a low slump. Where available PFA can be added to increase pumpability but environmental issues may preclude its use in the future. There are now proprietary mixes available in Europe, such as Blitzdammer manufactured by Heidleberg Cement, which are supplied ready mixed in dry form and which allows a much smaller site set-up.

59.4.2 Design principles

Generally compaction grouting is designed on the basis of an expanding bulb compacting the ground around it. The degree of improvement depends on the soil type and existing overburden, as compactive effort is lost when the ground starts to heave. When designing compaction grouting, the hole spacing, which usually ranges from 1.5 m to 3.5 m, is related to the depth of the compaction grouting and the zone of improvement required. A closer spacing is used where confinement is least and also where the treatment is at shallow depths. Treatment up to ground level is not possible as the compactive effort is minimal and in this case the shallow zone can be excavated and replaced in layers while applying compactive effort or treatment by another form of ground improvement.

Table 59.3 gives an approximate indication of hole spacing and the likely degree of improvement. Care needs to be taken when dealing with fills as the variable composition can lead to unpredictable results.

Generally the density of the ground can be improved such that standard penetration test (SPT) N values are increased by a factor of up to three depending on the fines content. The closer the spacing, the higher the degree of improvement but overburden plays an important part in providing the necessary confinement for the expanding bulb.

Design grouting pressures range from 40 bar at depths greater than 15 m to no more than 10 bar at depths less than 5 m. Holes are drilled to depth and the casing withdrawn in 0.5 m to 1 m stages while injecting grout. Typically stage volumes are controlled by pressure or resultant movement: the grout is injected until either the target pressure is reached or movement is triggered.

Soil type	Depth of treatment (m)	Hole spacing (m)	Improvement range (%)
Loose sands and sandy gravels	<3	1	150–200
	5–10	1–2	150–300
	>10	2–3	<300
Medium dense sands	<3	1	125–150
	5–10	1–2	150–175
	>10	2–3	<200
Silts	<3	1	<150
	5–10	1–2	<200
	>10	2–3	<200

Table 59.3 Hole spacing for compaction grouting

Grout strength is not usually an issue as the cement in the mix is sufficient to produce cube strengths in excess of 10 MPa. Low-strength compaction grout can be produced by substitution of the cement with non-cementitious material.

A good example of the use of compaction grouting was the Castle Mall shopping centre at Norwich. The site was underlain by chalk at shallow depth containing numerous solution features of variable size and depth. The infill material to these features was also variable but generally very weak. The development consisted of an 18-m-deep excavation within a perimeter of piled retaining walls and the presence of solution features within the site complicated the foundation design. Because the site investigation had located some features (8 out of 68 boreholes) but other features were likely to be present, the foundation design had to take account of the random location and nature of the features. The foundation design considered three solutions:

1. piled foundations, permanently cased through the features;
2. raft, spanning over any feature;
3. pad foundations combined with compaction grouting to improve the infill material.

The chosen foundation solution was pad foundations with ground improvement, as this allowed the chalk bedrock to be utilised for the majority of the foundations and compaction grouting would then allow the remaining foundations to be safely supported.

The engineer developed a strategy for dealing with the presence of features according to location and size, which is shown in **Figure 59.3**. This strategy was followed for the site works and was either a simple probe and bulk fill with low-pressure grouting or high-pressure compaction grouting.

The performance of the compaction grouting was based on post-testing using the SPT. This was set at an average of 15 and a minimum of 10. About 7% of the site area was discovered to contain features; there were 60 in number with a maximum size of 12 m in diameter. The largest feature had a plan area of 200 m^2. Grout volumes per metre of hole ranged from 0.26 m^3 to 1.0 m^3.

59.4.3 Execution controls

As with most ground improvement, execution controls are twofold: quality control of the materials employed and the resultant effect on the ground. Compaction grouting is controlled by ensuring a consistent low slump for the grout whilst monitoring for excessive line pressure or resultant movement.

59.4.4 Validation

The most common validation for compaction grouting is to compare *in situ* testing before and after treatment, usually using a cone penetration test (CPT) or SPT. The designer normally sets a post-treatment minimum value to be obtained based on the required degree of improvement. On some projects where justified, zone testing can be carried out, although the additional information gained may not justify the significant cost of the testing. **Figures 59.4** and **59.5** show typical results for SPT and CPT testing before and after treatment.

59.5 Design principles for permeation grouting

59.5.1 Methods and key issues

Permeation grouting by definition requires that grout is injected into the interstitial pores of the ground to replace air or water and, hence, bind the individual particles together.

A number of techniques are available, which are described below in Section 59.5.2.

59.5.1.1 Grout selection

With permeation grouting the design of the grout is important, as it must penetrate the interstitial pores with minimum disturbance. If the grout viscosity is too high or the particulate size is too large then complete uniform permeation will not be possible and this will cause problems with both treatment quality and also resultant movement.

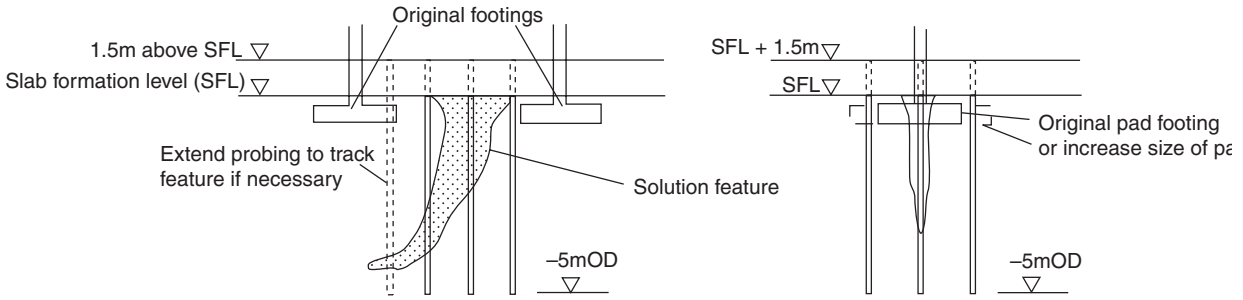
The design of the grout to be used depends both on the ground conditions and the technique being used. Ultimately, when using permeation grouting only limited types of grout can be used for lancing and end-of-casing grouting due to the relatively poor nature of the delivery system. This is discussed below for the above mentioned systems. Only Tube A manchette grouting can use a variety of grouts and the selection of grout for this system can greatly influence the quality of the end product.

Figure 59.6 shows the range of penetrability for the various grouts; most usually either cement bentonite or silicate-based grouts are used. As an example Thames gravels are commonly treated with either cement bentonite or silicate grouts or a combination of the two.

59.5.1.2 Cement bentonite

When used as an installation and injection grout, a typical mix would have a water/cement ratio of 2 with 10% bentonite to

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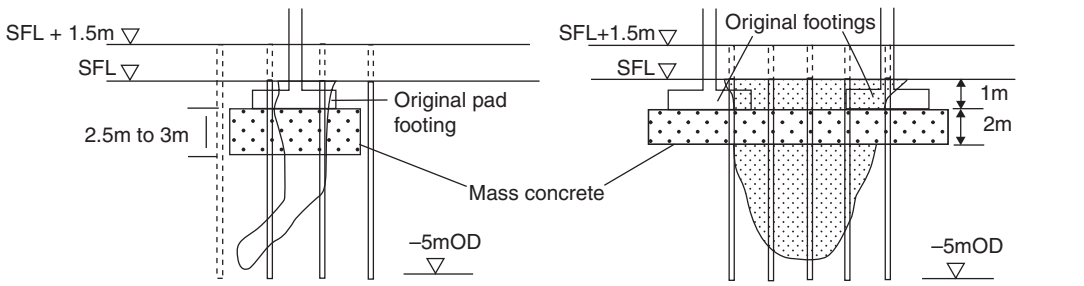


CASE 1 ▪ Features up to 4m across clear of foundations or up to 3m across at contiguous piled wall

Treatment ▪ Probe and bulkfill grout from 1.5m above slab formation level

CASE 2 ▪ Features up to 2m across beneath foundations

Treatment ▪ Probe and bulkfill grout from 1.5m above slab formation level

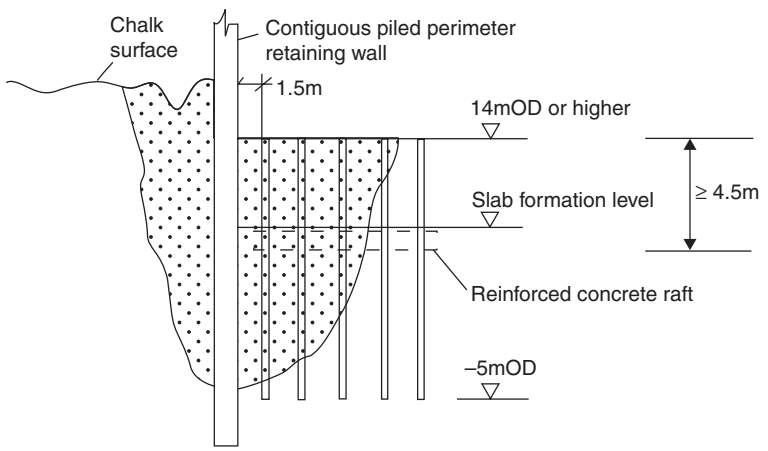


CASE 3 ▪ Features between 2m and 4m across beneath foundations

Treatment ▪ Probe and bulkfill grout from 1.5m above slab formation level
▪ Found original footings on mass concrete

CASE 4 ▪ Features larger than 4m across beneath foundations

Treatment ▪ Probe and compaction grout from 1.5m above slab formation level
▪ Found original footings on mass concrete



CASE 5 ▪ Features greater than 3m diameter at contigruous piled wall

Treatment ▪ Probe and compaction grout from high level
▪ Combine footings into a raft bearing on treated material

Figure 59.3 Cross-sections at solution features
Reproduced from Francescon and Twine (1992)

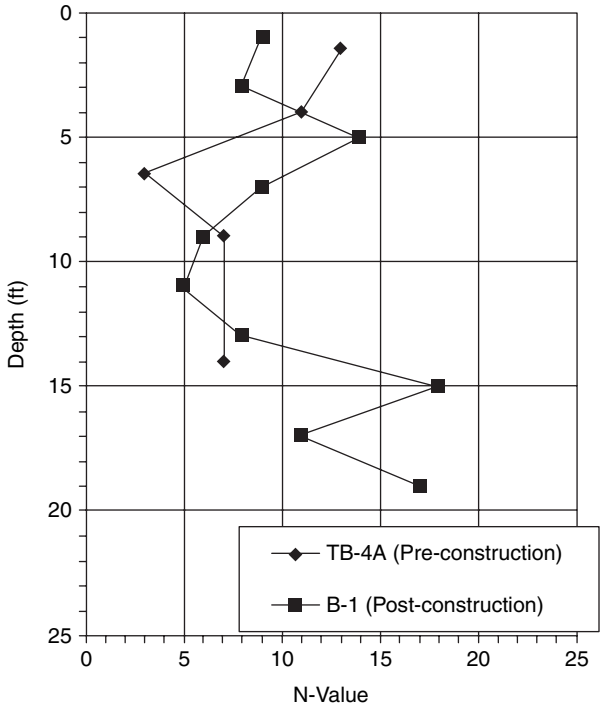


Figure 59.4 Example of pre- and post-testing for compaction grouting using SPT
Reproduced from Wilder, Smith and Gómez (2005)

cement. The unconfined compressive strength is in the range 2–4 N/mm^2 and the bleed <5%.

Sometimes the cement is replaced by a cement/PFA pre-blend where lower strength is required. An example might be where piling occurs after the grouting and there are concerns about penetrating grouted ground.

59.5.1.3 Silicate

In the UK, for grouting, the silicate is usually a proprietary brand, and the brands have different specific gravities. The silicate solution is mixed with a hardener based on organic esters, which react and cause the mix to gel. The gelling time depends on the type (specific gravity) of the silicate being used and the percentage of hardener added. The setting time is around 20–45 minutes and depends, broadly, on the hardener percentage, but is also temperature sensitive.

It is important to know the specific gravity when detailing the mix as differing dilutions will require differing amounts of hardener. A typical mix will contain 35–45% of silicate solution, 5–7% hardener and 50–60% water.

59.5.1.4 Microfine cement

Microfine cement is available in the UK in a number of grades.

The fineness of cement is given by the Blaine value, which is the surface area of particles per unit weight usually expressed as m^2/kg. The larger the value the finer the particles.

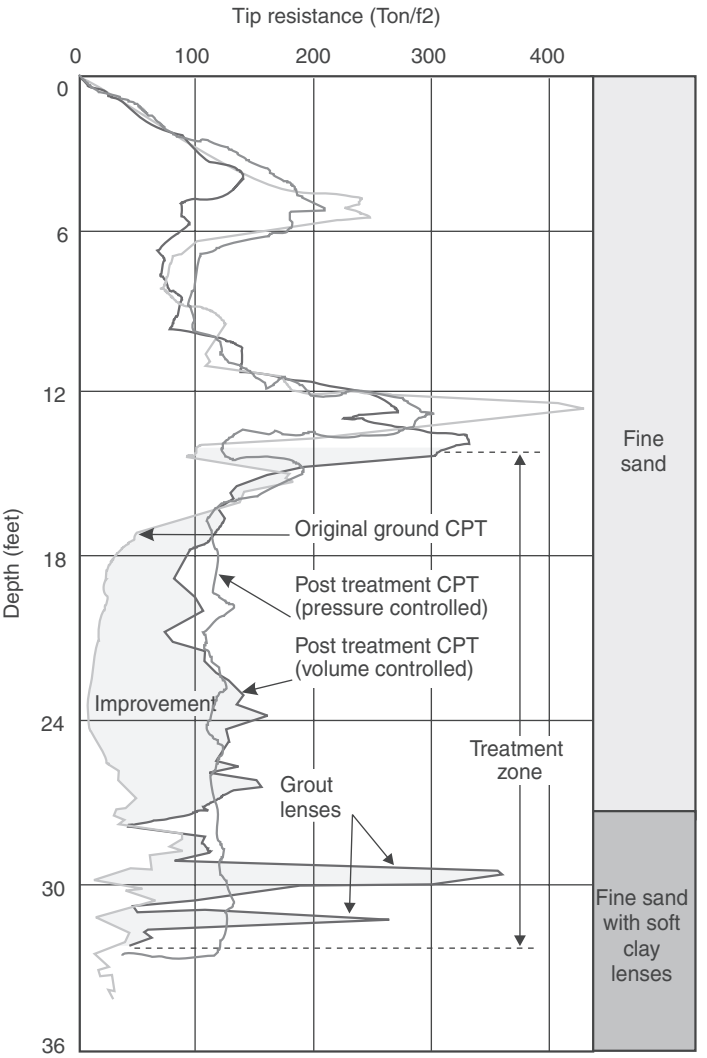


Figure 59.5 Example of pre- and post-testing for compaction grouting using CPT
Reproduced courtesy of Hayward Baker Inc © 2004

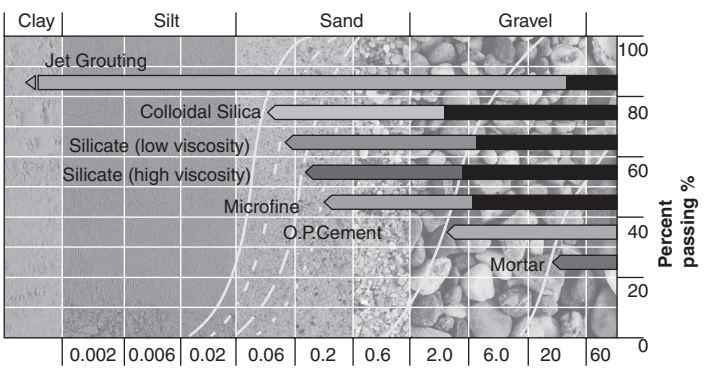


Figure 59.6 Range of penetrability for various grouts
Based on a figure from Keller brochure 67-03 E: Soilcrete® (jet-grouting process); Keller Group plc

As a rough guide the Blaine values for some cement-based materials are set out below

Material	Blaine value (m^2/kg)
O.P. cement	290–390
Rheocem 650	625
Rheocem 900	900

Microfine cement is normally utilised at relatively high water/cement ratios (1–3) and must be used with a plasticiser and bleed inhibitor to reduce the risk of blockage and to reduce the bleed. The finer the material the faster the setting time and with Rheocem 900, for example, penetration will be impaired after one hour from mixing.

Microfine cements are somewhat peculiar in that cubes taken for strength testing will bleed severely and may not set properly, but if mixed with sand will set quickly and produce a high strength. This can sometimes lead to problems on site if the engineer is not familiar with this behaviour.

59.5.1.5 Colloidal silica

This is a relatively recent grout and is a 'two-component' injection system, based on a nanometric colloidal silica suspension. The working time can be adjusted with an accelerator. The product may be used between +5°C and +40°C and contains neither solvents nor toxic components.

The grout will penetrate quite fine materials such as the Thanet, Harwich or Upnor Formation and has been used for the Channel Tunnel Rail Link for this purpose.

59.5.2 Design principles

59.5.2.1 Lancing

Lancing is one of the simplest of techniques but can be effective if carried out in the right circumstances. It is commonly used for temporary groundwater control for excavations below the water table but with very little head. **Figure 59.7** shows how an angled hole can be used to insert grout beneath a footing and improve bearing capacity. Lanced underpins are possible but risky as quality can be variable and, hence, can lead to settlement or even collapse. The most common use is to proof grout beneath slabs when settlement has taken place, loadings are increased or voids are present under slabs. Lancing is the process of driving a hollow tube or lance, at close spacings (usually less than 1 m), and injecting grout through the end as the lance is withdrawn. The lances are typically 25–38 mm in diameter. The tubes have a rivet blocking the drive end. A lance is driven in using a hand-held pneumatic drill and on reaching the desired depth it is withdrawn slightly so that the rivet can be knocked out by inserting a strand into the lance. Normally the depth is limited to 1–2 m but on occasions this has been increased to 3 or even 4 m in the right ground conditions. The grout injected is either a cement bentonite or silicate grout.

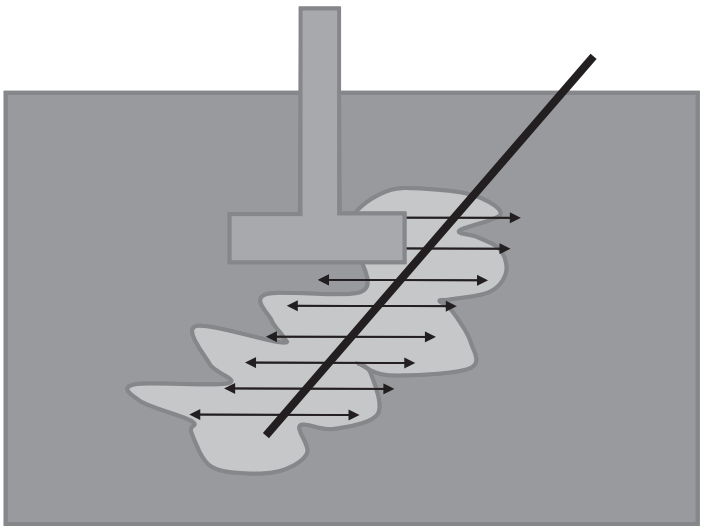


Figure 59.7 Example of hand lancing to improve the bearing capacity of a soil

Lancing is, therefore, restricted to the most open types of ground such as clean gravels and some forms of made ground. It can also be used to prove ground for voids where it is very cost effective.

Currently hand lancing is not favoured and should not be considered at the design stage, as prolonged use of the pneumatic tool leads to a condition called 'white finger' where feeling is lost in the fingers, which then leads to compensation claims. Health and safety issues mean that this work is very limited and the wearing of anti-vibration gloves is recommended. This makes the work more costly and its limited penetrability means that it is used less and less.

Because of the limited penetration, the hole spacing should be designed accordingly. Typically, the hole centres are 0.5 m to 0.75 m apart, except when carrying out under-slab proof grouting when the spacing can be increased to 1 m or 1.2 m. As a guide expect to inject only 10–15% of the ground by volume unless the ground is very open or voided when injected volumes could rise to 25%.

59.5.2.2 End-of-casing grouting

Typically, end-of-casing (EOC) grouting is used as a first-stage grouting in open or voided ground in advance of a second-stage tube A manchette (TAM) or compaction grouting scheme where use of the TAM or compaction grouting to fill voids and open ground would be slower and more costly. This type of grouting is used for probing and filling voids or open ground at depth. For example, it can be used for grouting around sheet piles when they have been jetted in and there is concern about toe restraint, canal lock and gate structures that are subjected to voided ground and as a precursor to more sophisticated grouting techniques.

End-of-casing grouting is a technique quite commonly used in open ground. As the name implies, the system comprises the

drilling in or driving of a 75–114 mm casing to depth, the withdrawal of the drill rods or removal of the drive point and the injection of grout. It can be used to quite considerable depth but is usually limited to 10 m. It is limited by breakout of grout around the casing. It is now commonly used at shallow depths due to the health and safety issues of hand lancing as described above.

It is limited to relatively open ground as the injection pressures are relatively low. Cement bentonite or silicate-based grouts are commonly used. It has the advantage over hand lancing in that the casing can be advanced through most ground conditions. All rig sizes can be used as the casing can be handled by the smallest of rigs. Generally EOC grouting can be used at wider spacings than hand lancing as the confining pressure is somewhat increased and some sealing of the casing can take place. Typical spacings would be 1.0 m to 1.5 m depending on soil type. Grout types are either cement-based (common) or silicate (rare). The grout mix can vary from cement/bentonite in open ground to cement/PFA in voided ground. The selection of mix is determined from the size of void and, therefore, the ease of injection.

59.5.2.3 Tube A manchette grouting

Tube A Manchette (TAM) grouting or sleeve port pipe grouting as it is known in the United States is one of the more sophisticated of permeation grouting techniques. It is probably the most commonly used technique, with jet grouting, for most ground treatment applications. In its conventional form it is used for permeation grouting but can also be used for compensation grouting where it is used effectively as a displacement grouting technique. Because the pipes are sealed into the ground the problems associated with EOC grouting are overcome and both the quality and range of application are increased. It can be used for groundwater control, underpinning, excavation retention, slope stability and bearing capacity enhancement.

It is widely used throughout the world and is a technique common to most countries and, hence, can be expected to be available from local specialists, although the technique is variable so control is important.

Figure 59.8 shows the principle of operation of a tube A manchette. The system comprises a 50–60 mm diameter steel or plastic pipe with openings at predetermined spacings, usually either 2 or 3 per metre length through which grout can be injected. The openings are covered with flexible sleeves, which open when pressurised by the grout and seal again when the pressure is removed. The injection or TAM pipe is installed in 90–150 mm diameter boreholes and sealed in with a cement/bentonite grout with typical Unconfined Compressive Strength (UCS) in the range 3–5 N/mm^2.

Each injection position can be isolated from the rest of the sleeves in a pipe by the use of a double packer. These are devices that are lowered to straddle the position of the sleeve and two flexible sleeves are inflated to isolate the injection. Grout is introduced down the centre of the packer and out through the sleeve. Sometimes, for small diameters, mechanical packers with leather washers can be used. In this way the injection

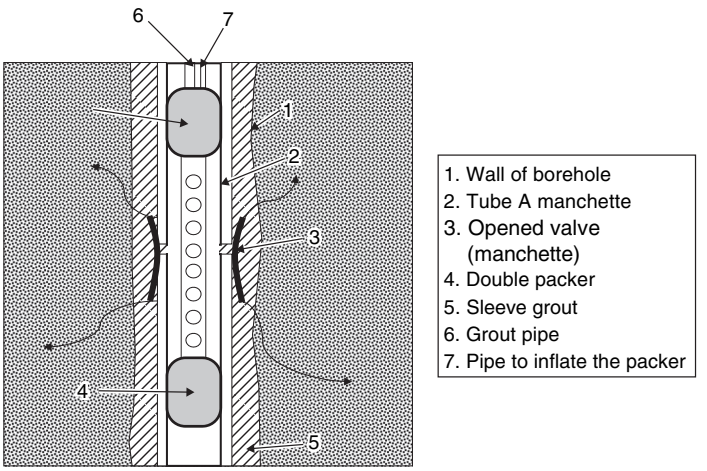


Figure 59.8 Operation of tube A manchette

volume, pressure and location of injection can be controlled to a high degree. For larger projects some of the major specialists use computer-controlled injection modules where a high degree of control and verification can be obtained. At the end of the day, the experience of the site supervisor and engineer will have a large influence on the quality of work carried out and the reliance on such modules should be limited to an ability to translate the designer's scheme into practice.

TAM grouting has been used for a number of years and **Figure 59.9** shows the layout from a project in the 1960s. **Figures 59.10** and **59.11** show typical layouts for tunnelling and dam construction.

With the sealing in of the injection pipes there is effectively no path for the grout except into the ground. In certain cases this will inevitably lead to either injection of grout into ground that it cannot permeate or reinjection into ground where there is no available pore space. Either of these situations will eventually lead to ground or structural heave and this should be taken into account in both design and specification. An assessment of services or structures at risk should be carried out and an appropriate monitoring regime implemented. Heave of 10–20 mm can occur when injecting silicate grouts into gravels without careful control and can be higher with cement-based grouts. It is possible to eliminate most heave by careful sequencing and volume control. This heave can be of use in compensation grouting, which is discussed below.

During design, careful attention should be paid to the location of any service or underground structure, as the grout can end up in services, especially old sewers where the lining is not competent. It is essential to identify such services prior to grouting and consider the effects of the grouting on the service. In particular for sensitive services (i.e. gas or chemical pipelines) additional monitoring and control will be required. For grouting adjacent to tunnels, due consideration to the effects of the injection pressure need to be considered especially for segmental construction.

The sequence of injection needs to be carefully designed to avoid the risk of trapped groundwater. If the sequence is

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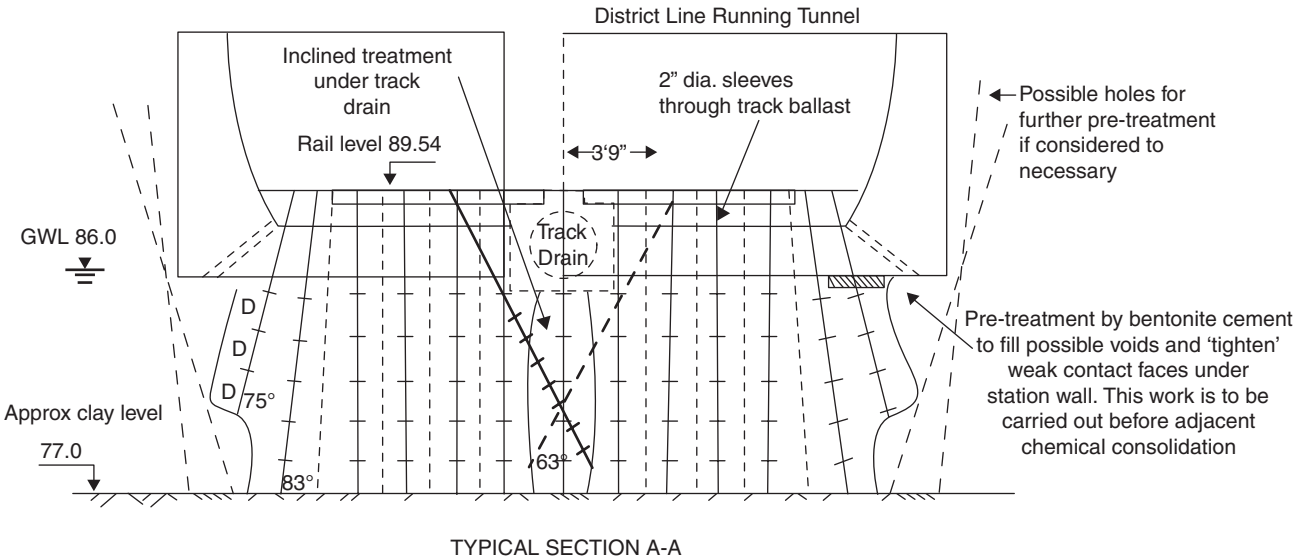


Figure 59.9 Example of historical layout of injection pipes
Courtesy of London Underground Ltd

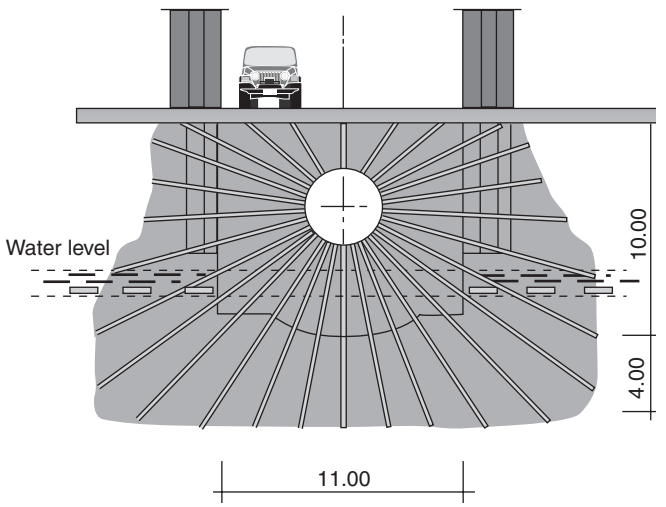


Figure 59.10 Example of injection layout for construction of tunnel using pilot tunnel

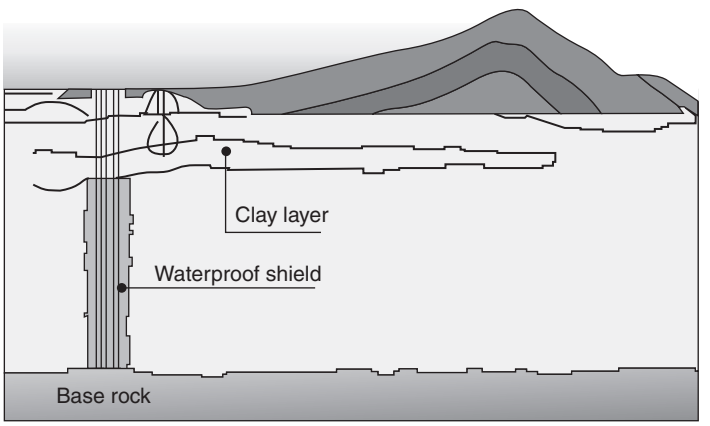


Figure 59.11 Example of use of permeation grouting to provide cut-off for a dam

such that as the grouting programme proceeds there is no flow path for the displaced groundwater, then it can emerge into structures or above ground or if unable to be displaced it can prevent ground treatment taking place as the grout cannot penetrate the ground. An escape path needs to be allowed when considering the injection sequence and the scheme monitored for excess water pressure.

The hole spacing is generally in the range 1.2 to 1.7 m, usually with a triangular grid. Wider spacing is rarely used as the ability to permeate such distances becomes more problematic. The effect of hole deviation is discussed below for jet grouting where it is considered in detail but is also applicable to permeation grouting.

The design must consider hole spacings, grout type and target grout injection volumes.

For most applications the holes are installed on a triangular grid and the target injection is derived from the following equation

$$Q = \frac{D^2 \sin 60° n}{N} 1000 \qquad (59.1)$$

where Q is the target injection volume per sleeve (litres), D is the hole grid spacing (m), n is the ground porosity and N is the number of sleeves per metre.

Typically for a 1.5 m grid in ground similar to Thames Gravel, the target injection volume will be about 150 litres.

Figure 59.12 shows the required injection volumes per metre for varying ground porosity and hole spacing.

The rate of injection is also important and injection rates of 8–12 litres per minute are typically used, leading to injection times of 10 to 20 minutes for each sleeve. Silicate grout should be injected at the lower end of these rates as penetration can be more difficult in finer ground.

Injection pressures are usually based on pressures measured at the top of the holes or at the injection module. Typical injection pressures are 0.25 bar to 0.5 bar per metre of overburden. Some of the more sophisticated modules can carry out line calibrations to determine the pressure loss through the system to the packer and correct for this on the pressure recording system. This is useful as most historical work was based on manual injection through a distributor arrangement (banjo) at or close to the borehole mouth. Hence, the line loss between the pump and borehole was effectively eliminated and the 0.25–0.5 bar injection pressures work on this basis. Overcorrecting could lead to excessive pressures in the ground, which could encourage hydro-fracture rather than permeation.

The use of silicate grouts will lead to lower ground strengths and lower deformation moduli than when cement-based grouts are utilised. Typically, the strength of treated ground will have a UCS of around 1–3 MPa.

An example of the use of permeation grouting was on the Channel Tunnel Rail Link (CTRL) where the new railway lines passed under the existing London–Southend railway lines. **Figure 59.13** shows the section at the crossing. The resulting cofferdam was predicted to cause too much movement to the adjacent viaduct and it was decided to install a grouted plug to reduce movements. Because of the presence of the alluvium, the plug did not have to provide waterproofing and, hence, it was decided that permeation grouting would be acceptable.

Figure 59.14 shows the TAM layout and also the distribution of grout injected. Both silicate- and cement-based grout were utilised and **Figure 59.15** shows the improvement in SPT test results following grouting. The cofferdam was excavated with movements of less than 10 mm.

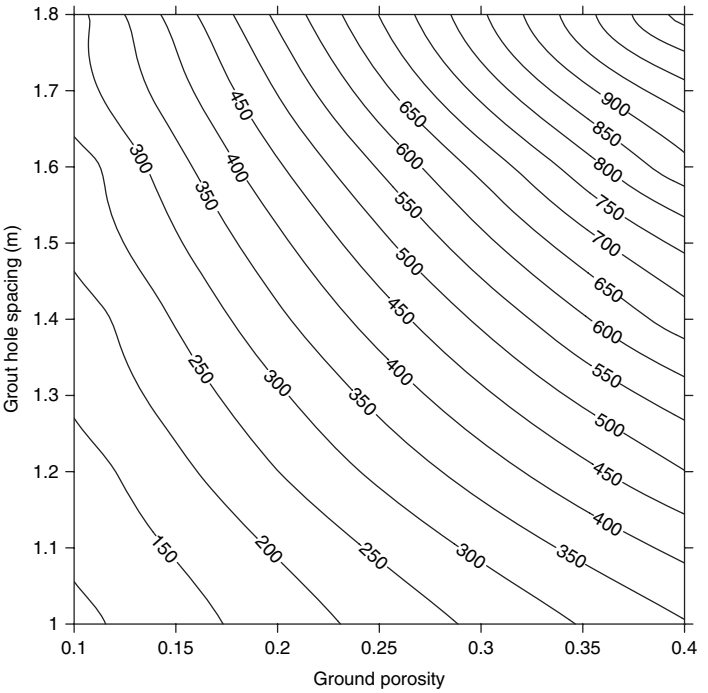


Figure 59.12 Required grout volumes per metre for differing ground porosities and hole spacing

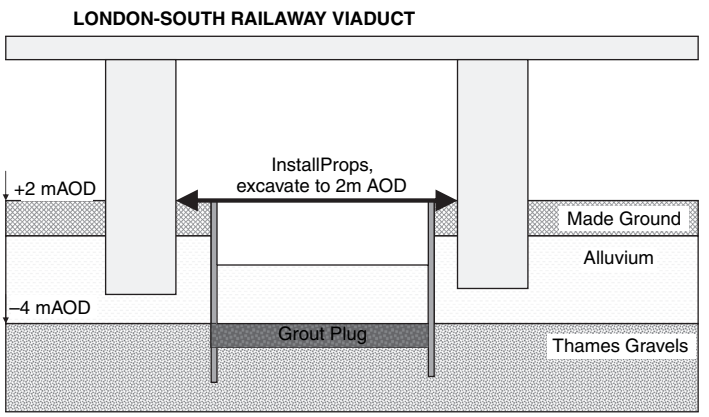


Figure 59.13 Section at CTRL railway crossing

59.5.3 Execution controls

Most execution control relates to initially ensuring that the injection holes are installed in the right locations and then monitoring the injection of grout. Secondary controls can be evaluating movements caused by the grouting.

For the simpler systems such as lancing or end of casing, execution control is limited and generally will be paper-based simple records of hole numbers and volumes of grout injected. Additional refinements can be to record volumes injected for each stage of injection.

TAM grouting tends to push the limits of capability of ground treatment and, hence, needs careful control to avoid mistakes on site. In particular, the large number of injections requires a high degree of site control to ensure each sleeve is injected into the correct hole. With the advent of site computers this is now more easily and efficiently accomplished and their use should now be regarded as a specified requirement for most projects.

59.5.4 Validation

Validation for hand lancing is not generally carried out except by the drilling and grouting of holes at intermediate positions. Where lancing is used for proof grouting under a slab or void filling, then additional intermediate holes are useful to assess the degree of coverage.

Validation for EOC grouting is generally restricted to either falling or rising head tests within the casing installed for testing

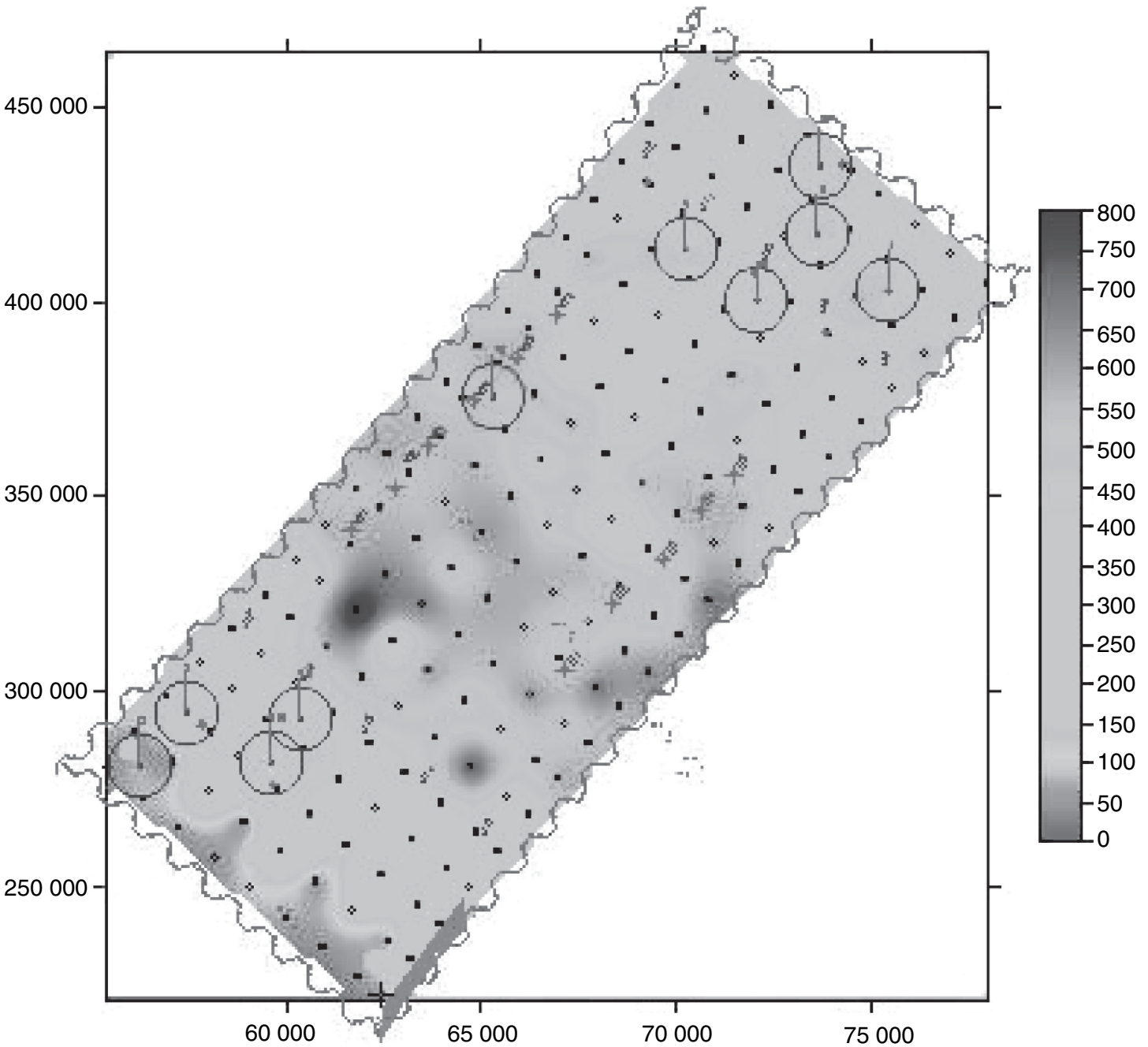


Figure 59.14 Example of validation of permeation grouting by reviewing injection data

purposes, or through piezometers installed following ground treatment. A further test could be the drilling and grouting of intermediate holes, the results being judged on the grout take. If strength or deformation is an issue then either coring or pressure meter testing could be carried out but test cell formation can be a problem and core recovery poor due to the fact that the material consists of relatively strong aggregate grouted with a relatively low-strength grout.

Validation for TAM grouting is essentially as described for EOC grouting above. Additionally it is possible to re-inject through sleeves to assess grout take and refusal pressure. When assessing the success of a TAM grouting project it is common practice to review the primary and secondary grout injection volumes. Normally one would expect to see a reduction in grout take from primary to secondary grouting coupled with some increase in refusal pressures. **Figure 59.14** shows

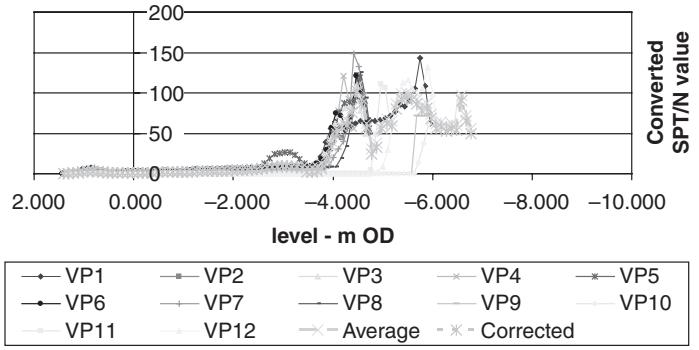


Figure 59.15 Comparison of pre- and post-testing

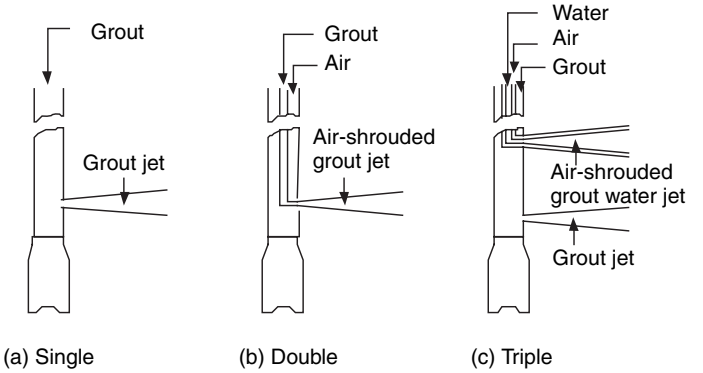


Figure 59.16 Jet-grouting systems
Reproduced from Essler and Yoshida (2005); Taylor & Francis Group

such a validation following completion of grouting in terms of grout volumes and **Figure 59.15** shows a validation in terms of *in situ* testing.

59.6 Design principles for jet grouting

59.6.1 Methods and key issues

Jet grouting differs from permeation grouting in that the intent with the latter is not to disturb the fabric of the ground whereas with the former this has to be the case for successful application. Because the product of jet grouting tends to be more structural in nature, it has a great range of application. When installed beneath buildings it can extend foundations through poor ground or support them while excavations, often unsupported, are carried out next to them. It can provide support underground where openings are necessary such as for tunnel boring machine (TBM) break-out or break-in, cross passages or equally to support tunnel faces. Because it does not rely on passage through the ground its product is usually more predictable and significantly less dependent on ground strength or composition. It can provide groundwater control at the base of excavations and can prop retaining walls at the same time. It is a multi-application tool for support and groundwater control, and simultaneously increases the efficiency of site usage.

There are three basic systems available:

(1) single system;

(2) double system;

(3) triple system.

The three systems are illustrated in **Figure 59.16**.

The single system is the injection of only grout at high pressure. This was the first system to be used. The column diameter is limited and the borehole has a tendency to become blocked, often resulting in ground heave. The design column diameters are small, usually up to 1.2 m in diameter. Some designers take the view that the single system provides the highest-strength columns and that air bubbles trapped in the columns from the other systems reduce strength.

The double system is effectively the single system with the addition of an air shroud to the nozzle. The addition of the air shroud increases jetting efficiency dramatically, typically giving a 30% increase in design diameter for the same jetting energy. These are very versatile and powerful systems with a design column diameter up to 4–5 m in most soils although such large diameters are only practical on large projects with no site constraints. Typical column diameters in normal use would range from 1 m to 3 m.

The triple system differs from the single and double systems in that erosion of the ground is carried out by a high-pressure water jet shrouded with air with an additional low-pressure grout line. The design column diameters achieved with this system are greater than those of the single system but as the energy is relatively low compared to the double systems, columns are usually only up to 1.7 m to 2 m (in exceptional ground) in diameter. In the US an advanced super triple system can deliver much higher energies and as a result design diameters of 3–4 m are possible.

The range of design applications for the systems are similar and selection is a site-specific requirement. The single system is most commonly used for horizontal applications, where air may cause problems. For most other applications the choice normally rests between the use of the double or triple system.

Generally the triple system will produce more spoil but is usually less viscous and, hence, offers less risk for blockage and probable structural or ground movement.

Jet grouting, historically, has been considered a risky ground treatment process due to the high pressures involved and the spoil generation. With good experience, site operatives, site control and design, jet grouting is not riskier than other processes.

The main risks associated with the process are the use of high pressures and the potential generation of movement. When jet grouting beneath buildings, care must be taken to ensure that heave of the walls or floors is maintained within agreed limits. It is absolutely critical that the construction details of the floors and walls are understood so that the effect of jet grouting can be considered. Where floor construction is poor and unsealed then it is advisable to maintain visual inspection and continual monitoring during jet grouting. Wall movement during jet

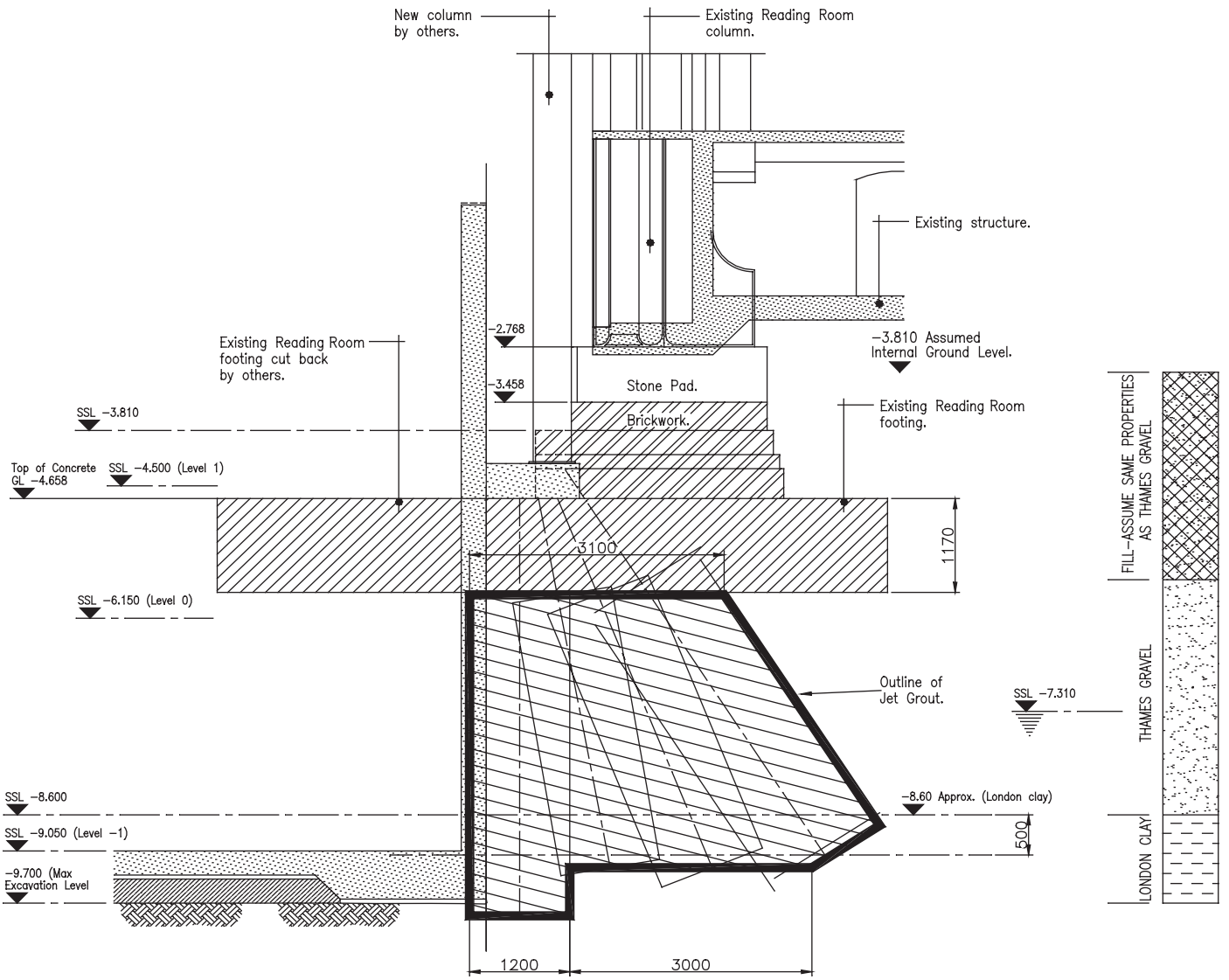


Figure 59.17 Cross-section of British Museum jet-grouted underpin
Reproduced with permission from CIRIA SP199, Scott and Essler (2003), www.ciria.org

grouting can be controlled in the long term by precise levelling and in the short term by the use of rotating laser targets fixed to the wall.

Design of the grouting sequence is extremely important for underpinning operations as there is the possibility of loss of support if too large an area is jetted. Where the structural foundation condition is poor, i.e. with poorly cemented masonry or brick, then a pre-grouting exercise using lancing or end of casing can be considered with the intent to reinforce the available bonding and reduce the risk of partial wall collapse into a freshly completed column. For underpinning work it is usual to restrict the jetting of adjacent columns to intervals of at least 24 or 48 hours. For isolated foundations, the jetting sequence must be designed to prevent loss of support and in extreme cases, temporary load transfer should be provided during the jet grouting operation. There is usually excellent bonding between the foundation and the jet-grout columns as subsequent adjacent columns always treat this zone and ensure perfect contact.

Grouting close to an underground structure needs careful consideration and any grouting carried out within 3–5 m of any underground structure will require a specific risk assessment. Wherever possible, consultation with the original designer is advisable to agree the specifics of the ground treatment.

The most common application for jet grouting is either underpinning or to restrict movement within excavations. **Figure 59.17** shows the use of jet grouting to underpin the Reading Room of the British Museum (Scott and Essler, 2001, 2002). This is a Grade 1 listed building and the jet grouting provides permanent support. **Figure 59.18** shows the effect of

the installation of a deep jet-grout strut within an excavation in significantly reducing movements (Driesse *et al*., 2006). Such jet-grouted struts are widely employed worldwide but have had little use to date in the UK.

59.6.2 Design principles

Because of the ability of jet grouting to bond ground and create relatively strong bodies (UCS range 2–10 MPa), the design process is similar to the design of brick or masonry.

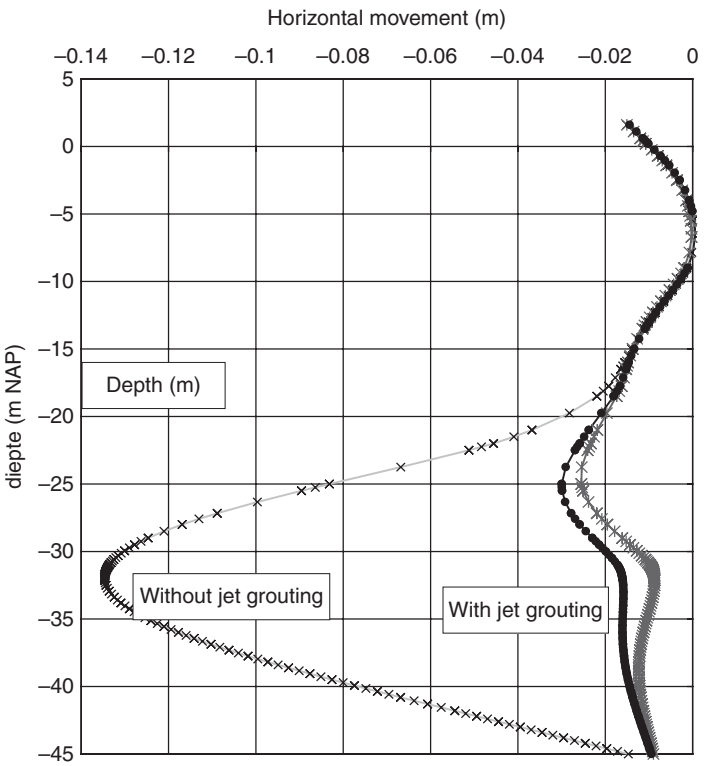


Figure 59.18 Effect of installation of jet-grouted strut at 33 m

The strength of treated ground is usually assessed on the basis of unconfined compressive strength tests on samples obtained by coring. The histograms shown in **Figure 59.19** demonstrate experimental unconfined compressive strengths in sandy and cohesive soils. The Japan Jet Grouting Association has adopted these distribution charts, defining that the unconfined compressive strength used in design as a minimum safe value should be adopted as within the 1% to 3% range of these charts.

This definition gives the standard unconfined compressive strengths set out in **Table 59.4** (where the water/cement ratio of the grout is typically 1).

It has to be stated that the adoption of these design values in certain conditions could lead to conservative designs in granular soils. Typical values of UCS adopted for design in the UK would be in the range 5–10 MN/m^2 for granular materials. Strength is usually only an issue for (deep excavations or tunnelling projects) base slabs and shaft break-out or break-in. In these cases, strength can influence the design as the jet grouting is required to span across openings and, hence, has to be designed with a minimum structural integrity.

Jet-grout column layout is designed based upon the hole deviation (i.e. the vertical tolerance) and the column diameter.

For most operations, when drilling holes, the hole tolerance is unlikely to be better than 1 in 100 and is typically specified at 1 in 75. For shallow holes this does not create an issue but as depth increases then it becomes a more important influence on the design of the jet-grout layout.

When creating a jet-grout base slab, the design is normally based on a triangular grid.

Figure 59.20 shows the effect when a single column deviates from the design position. The solution is to decrease the column spacing. This can have a significant influence on cost as the column diameter and spacing effect the number of columns.

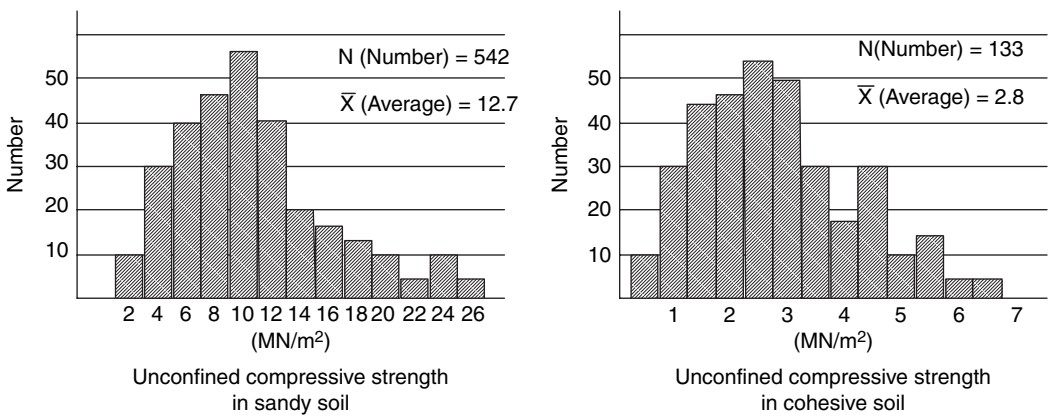


Figure 59.19 Typical strength distribution for jet-grouted soils for sandy and cohesive soils
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Consider the design for a base slab 20 m by 20 m, at a depth of 10 m below ground level (Case A) and 20 m below ground level (Case B).

59.6.2.1 Case A

If we assume that the current jet-grout system will produce column diameters of 1.5 m and that the expected hole tolerance is 1 in 75 then:

Jet-grout design spacing assuming no deviation	1.2 m	
Expected hole deviation	0.133 m	
Hole spacing to take account of deviation	1.0667	
Column spacing in ***x*** direction	1.0667 m	
Row spacing in ***y*** direction (60° grid)	1.0667 sin 60°	= 0.924 m
No. of columns in row	20/1.0667	= 19
No. of rows	20/0.924	= 22
No. of columns for 10-m-deep base slab	22 × 19	= 418

Soil type	Q_u: Unconfined compressive strength (MN/m²)	*C*: Cohesive strength (MN/m²)	*f*: Bond strength (MN/m²)	σ_t: Bending tensile strength (MN/m²)
Cohesive	1	0.3	0.1	0.2
Granular	3	0.5	0.17	0.33

Table 59.4 Typical design values for jet-grouted materials

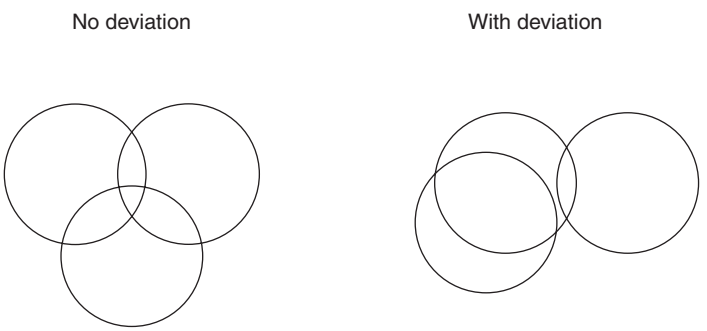


Figure 59.20 Effect of drill-hole deviation on column overlap

59.6.2.2 Case B

By the same calculation but with a depth of 20 m and, thus, allowing for more deviation, the number of columns increases to 550, an increase in the number of columns of 32%.

Therefore, it is important to understand that depth plays a significant role in determining cost and potentially the jet-grout system. It is good practice to measure the hole deviation when depth becomes an issue (usually at depths greater than 15 m) and some specialists can offer hole deviation measurement using the jet-grouting equipment, thus, reducing delays to a minimum

If on the other hand we consider the same scenario except this time we use a more powerful system capable of 3-m-diameter columns, the number of columns changes as shown in **Table 59.5**.

Clearly we see a difference in that with 1.5-m-diameter columns, the number of columns increases by 132 from case A to B, but for 3-m-diameter columns this increase is only 18 columns.

The above highlights the current drive to increase column diameter as this not only reduces cost but provides more security at depth as the column interlock is not as critical.

The other factor that influences cost and, hence, design is the actual column volume cut versus the volume required.

If we consider the above cases, the volume of column cut per metre is respectively 1.767 m³ and 7.069 m³ for 1.5 m and 3-m-diameter columns. **Table 59.6** illustrates the comparison.

The volume jetted will affect both cost and spoil production as they vary linearly in proportion to the meterage jetted.

Good examples of the correct design of jet grouting are the base struts incorporated into the design of the station boxes (Driesse *et al*., 2008 a,b) and the sandwich wall at central station (De Wit *et al*., 2006, 2007) on the North–South project in Amsterdam (Driesse *et al*., 2008a, b). **Figure 59.18** shows that

Case	Depth (m)	Column diameter (m)	Nominal design spacing (m)	Number of columns	Percentage increase with depth
A	10	1.5	1.2	418	
B	20	1.5	1.2	550	32
A	10	3.0	2.7	72	
B	20	3.0	2.7	90	25

Table 59.5 Effect of depth and diameter on column numbers required

Case	Depth	Column diameter (m)	Nominal spacing (m)	Number of columns	Column volume jetted per metre (m³/m)	Column volume as % of slab volume (400 m³/m)
A	10	1.5	1.2	418	739	185
B	20	1.5	1.2	550	972	243
A	10	3.0	2.7	72	509	127
B	20	3.0	2.7	90	636	159

Table 59.6 Effect of column diameter and depth on column volume required

without these struts, the wall deflection would have been excessive with resultant damage to the adjacent historical buildings. The initial design of the jet grouting is shown in **Figure 59.21** and essentially mimicked conventional strutting; however, as the strut was some 33 m below ground level, it ignored the difficulties of achieving positional accuracy without which the strut would not transmit the required loads. The resultant redesign assumed a uniform distribution of the columns within the station box with a nominal overlap. Extensive finite element analysis of over 1000 layouts of columns randomly positioned based on assumed diameter and positional accuracy demonstrated that the strut would behave as designed. **Figure 59.22** shows the revised layout. Designing jet grouting based on a statistical investigation of the column diameter and position is important and is recommended for any design where the depth and soil conditions could have significant effects. It is especially relevant for designs where the jet grouting is required for waterproofing, as column variations and, hence, gaps between columns actually control the water flow.

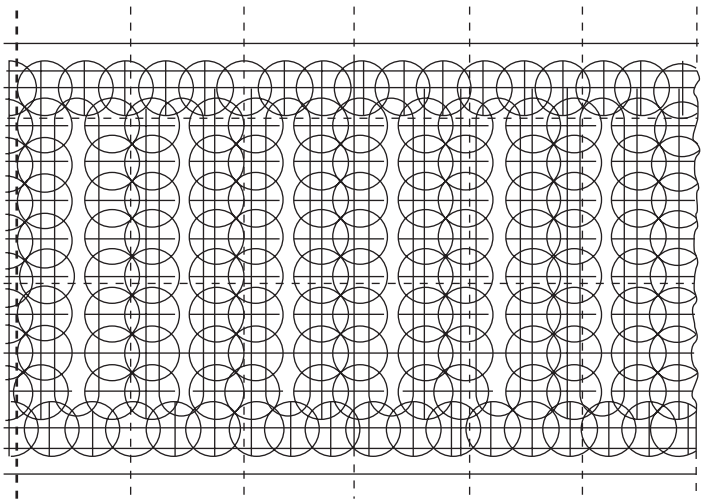


Figure 59.21 Initial column design layout for the North–South Line, Amsterdam

59.6.3 Execution controls

Execution controls comprise the quality assurance of the grout materials and the grouting process. Generally, most jet-grouting rigs are fitted with sophisticated instrumentation that records all the required process parameters. This gives the designer confidence that the columns have been constructed reliably. There are some systems that include a built-in borehole deviation system that can be used to actually monitor the as-built locations of individual columns. As the rigs are instrumented, the drilling energy can be easily plotted against depth and this can give the designer valuable insights into ground variability and it can drive process adjustment.

59.6.4 Validation

Validation is generally as for TAM grouting. The validation process consists of quality control of the grout materials and the column jetting parameters followed by post jet-grouting testing. Because jet grouting generally is structurally sounder, coring can be more successful if care is taken. In addition the process lends itself to trial exposure whereby a number of initial columns are jetted on site and then exposed after 24–48 hours to determine size and uniformity. With jet grouting, the column diameter tends to be the most important parameter for validation as this is fundamental to the construction of gravity or underpinning blocks. In granular ground, strength is not usually an issue.

Laboratory testing of cores can be carried out and the designer needs to be careful in relating potentially solid core results to the mass properties. **Figure 59.23** illustrates results of destructive

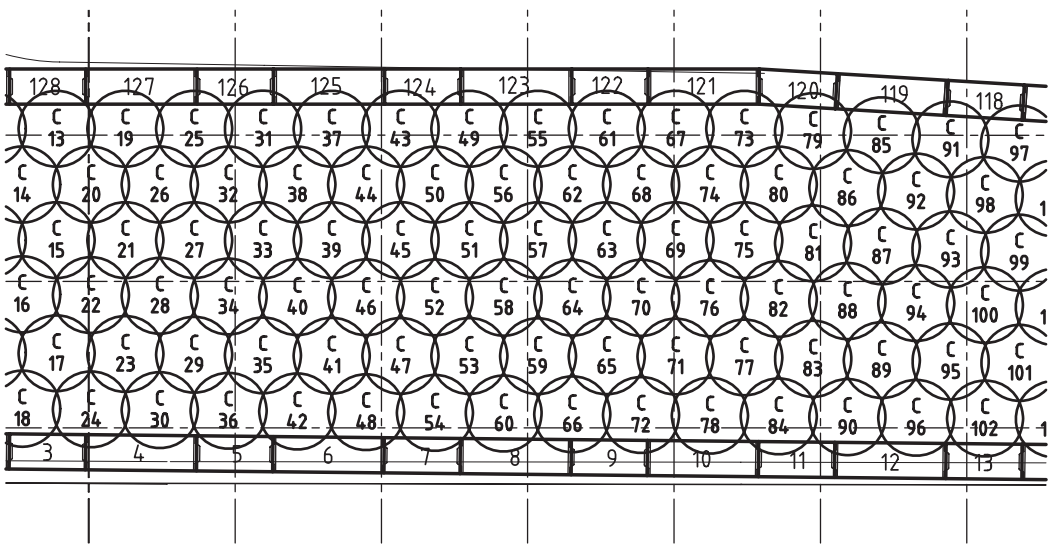


Figure 59.22 Revised column layout for the North–South Line, Amsterdam

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drilling, which is relatively quick and economical. It is a powerful validation tool during design and it can be calibrated against coring and subsequent laboratory testing. Destructive drilling must be instrumented and the drilling energy then plotted against depth. There is a correlation with the core strength, which can be used to identify zones of low strength. This methodology should be considered on all major projects where repetitive testing of large numbers of columns is required.

59.7 Design principles for vibrocompaction and vibroreplacement

59.7.1 Methods and key issues

Kirsch (1993) describes the techniques in detail but typically vibrocompaction is carried out by inserting a vibrating closed-end steel tube of diameter 300 mm to 500 mm, commonly known as a poker, into the ground, thus, causing the surrounding ground to be densified by the ground vibrations. The design of individual pokers is commercially sensitive but in principle they operate by hydraulically rotating an eccentric weight horizontally within the poker. In this way the vibrations have a significant horizontal amplitude and cause ground densification.

The vibrations cause the soil grains to be rearranged into a more dense packing and, thus, improve the soil density. Water jetting can be used in association with the poker to assist penetration of the poker and flush out soft finer material. It also has the effect of locally reducing the effective stresses and aiding ground restructuring.

Generally the poker is lowered to depth and then raised in stages as the ground is compacted. The applied energy can be monitored during compaction as higher energy is needed when the surrounding ground becomes densified. The poker can be systematically raised and lowered in stages with the lowering phase used to judge the compaction of the underlying ground.

Care needs to be taken with this process to ensure that the ground can be densified solely by compaction. Problems occur if the content of silt and clay is too high. If bands or layers of silt or clay are present these are unlikely to be improved leading to unexpected settlement. The layout of penetration points is usually triangular with the spacing generally following from the soil type and available poker power. Generally, the more powerful the poker, the wider the spacing. Very powerful pokers have been developed over the years to deal with large areas

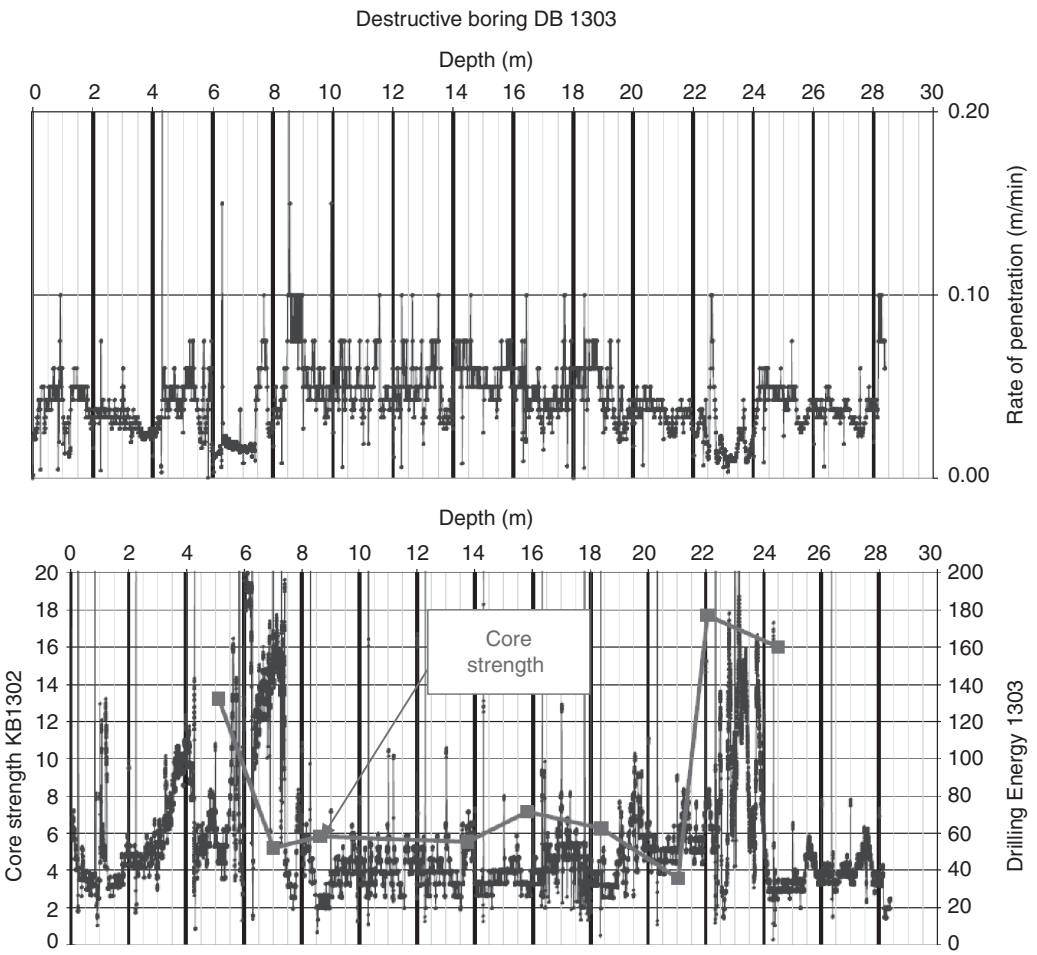


Figure 59.23 Use of destructive drilling to check jet-grout columns

of marine sands and reclaimed materials where wide spacings can be economically employed.

Vibroreplacement or stone columns is a similar process to vibrocompaction but it relies on creating columns of compacted stone, which will transmit load or resist shear. On reaching depth, stone is introduced either at the top of the hole or at the bottom via a special tube built into the poker. These methods are known as top feed and bottom feed, respectively. The stone is compacted as the poker is withdrawn and thereby improves the ground. Bottom feed is used where there are weak soils with a high water table and, thus, the column may be unstable.

59.7.2 Vibrocompaction and vibroreplacement design principles

Vibrocompaction is commonly utilised to provide reduced total or differential foundation settlements or to provide seismic improvement to weak soils. **Figure 59.24** indicates the range of soils suitable for treatment.

Design methods are generally based on the principle of overall ground density improvement (vibrocompaction) or an overall improvement based on the percentage area of improved ground.

Typical expected settlements are as set out in **Table 59.7**.

Priebe in 1995 produced an excellent paper on vibroreplacement and **Figure 59.25** gives an indication of the improvements possible.

The area ratio is defined as the area of a column (A_c) as a percentage of the area supported by the column (A). The improvement factor is effectively the increase in stiffness or shear capacity over the original soil. Thus, if around 20% of the area were replaced with stone columns, this would provide around 200% to 250% improvement in settlement reduction.

The use of area ratio gives a guide to the column spacing, assuming that column diameters range from 0.5 m to 0.75 m.

An initial simple design concept is to consider vibro-stone columns as relatively strong springs (although of lesser rigidity than piles) surrounded by the weaker springs of the soils requiring treatment. The applied load is then shared between a composite structure of stone column and soil on an areal basis. This can be expressed as:

$$Aq = Q_c + (A - A_c)\sigma_s \tag{59.2}$$

where A is the area supported per stone column, q is the applied pressure, Q_c is the safe capacity of the stone column, A_c is the cross-sectional area of the stone column and σ_s is the safe capacity of the surrounding soil.

In this formula the safe capacity of the soil is sometimes taken as zero for better post-treatment settlement performance.

Soil type	Bearing pressure (kPa)	Settlement (mm)
Made ground: mixed cohesive and granular	100–165	5–25
Made ground: granular fill, ash, brick rubble, etc.	100–215	5–20
Natural sands or sands and gravels	165–500	5–25
Soft alluvial clays	50–100	15–75

Table 59.7 Bearing pressures and settlements for vibro-improved ground

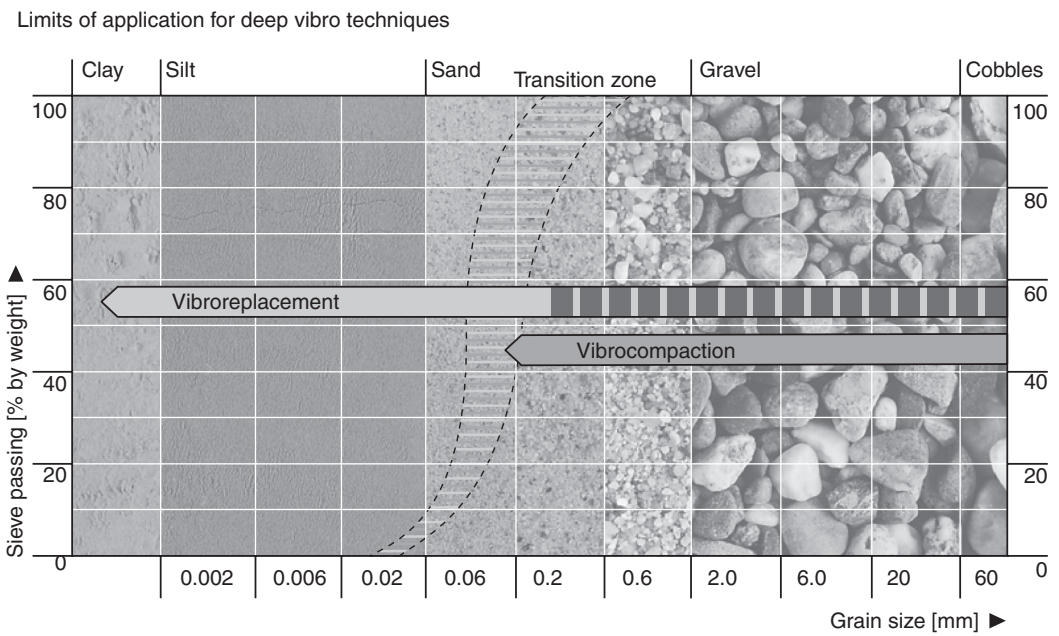


Figure 59.24 Range of application of vibroreplacement and vibrocompaction
Reproduced from Keller brochure 10-02 E: Deep Vibro Techniques; Keller Group plc

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There are three main design procedures used in the UK:

(1) Hughes and Withers (1974);

(2) Baumann and Bauer (1974);

(3) Priebe (1995).

The most commonly used method for an initial calculation of the safe capacity of a stone column is that of Hughes and Withers:

$$\sigma_v' = (1+\sin\varphi_c)(\gamma_b h_c + 4c_u + P)(1-\sin\varphi_c) \qquad (59.3)$$

where σ_v' is the ultimate vertical effective stress in the soil (kN/m^2), ϕ_c is the friction angle for the stone column material (typically 45°), γ_b is the bulk density of the soil (kN/m^3), h_c is the critical depth (m), c_u is the undrained shear strength of the soil (kN/m^2) and P is the surcharge (kN/m^2).

The critical depth h_c is often taken as the depth from ground level to the base of the foundation plus one stone column diameter.

The safe capacity of the stone column is then given by:

$$Q_c = \sigma_v' A_c F \qquad (59.4)$$

where Q_c is the safe capacity of the stone column (kN), A_c is the area of the stone column (m^2) and F is the factor of safety.

Figure 59.26 shows how the capacity of a stone columns varies with depth and soil type.

When loads are applied rapidly, e.g. when building road embankments on soft ground, silos, coal stockpiles, etc., an allowance should be made for the development of excess pore water pressures in the soils surrounding the stone columns. This is often taken as $-u$ within the bracket of the first formula or as an ru percentage. Thus in the first equation, ρ is replaced by $\rho - u$ or $\gamma_b h_c$ replaced by $ru\gamma_b h_c$.

The majority of stone-column bearing capacity and reduced settlement designs now use the Priebe (1995) method where the area ratio of **Figure 59.25** can be used to assess the initial settlement reduction factor that is applied to the untreated settlement within the treatment depth. There will then be further potential settlements due to the imposed loads beneath the treatment depths, as well as ongoing self-weight settlements within fills and made ground, secondary consolidation, etc.

However, the Priebe method then considers the beneficial effects of column rigidity, depth of overburden and group effects to further refine the post-treatment settlement predictions. As a result many geotechnical consultants and specialists provide computer-generated calculations using this method.

Clearly, the treatment depth is a fundamental factor in the design, which will also dictate the vibro-equipment that is to be used to construct the stone columns (see Chapter 74 *Design of soil nails*).

One of the largest vibroreplacement contracts was carried out in Barrow-in-Furness in 1991 for a proposed on shore gas terminal (Raison, 1999). Before development, the majority of

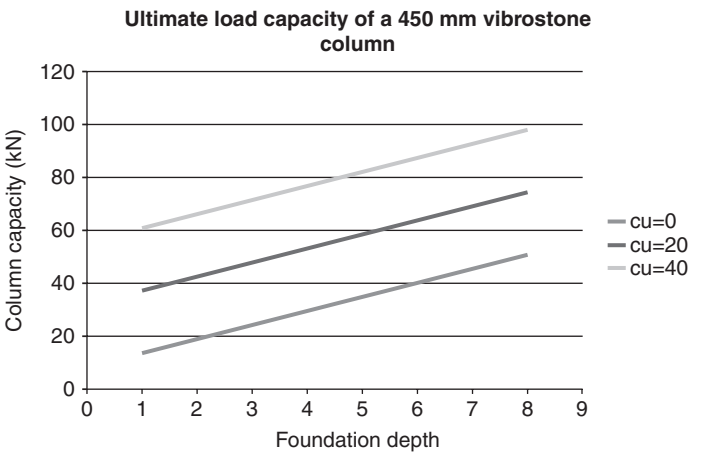


Figure 59.26 Stone column capacity for various depths and soil types

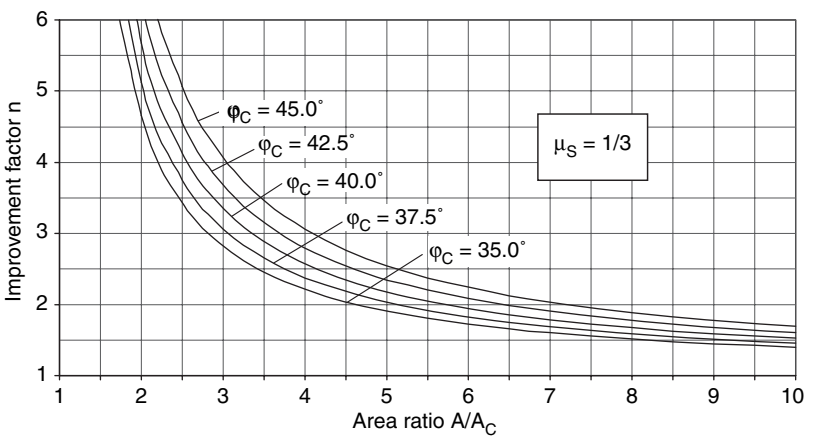


Figure 59.25 Design of stone columns
Reproduced from Priebe (1995)

the site comprised three former settlement lagoons containing saturated pulverised fuel ash (PFA), a waste product from the adjacent coal-fired Roosecote power station. Site investigations revealed ground conditions beneath the PFA as comprising very loose silty gravelly alluvial sand over more dense glacial sands, with glacial till and sandstone at depth. Ground conditions were locally variable and simplifications were necessary for design purposes. A typical geological section and variation of SPT with depth is shown in **Figure 59.27**. Particle size grading of the PFA indicated a sandy silt, which was often clayey. It was concluded during the project design stage that protection from seismic episodes, although small in the UK, was required and a combination of driven piles and vibroreplacement was selected by the contractor, Keller. While the vibroreplacement was not deemed suitable for improvement of the very soft clayey PFA, which was deemed too fine being a sandy silt, it could improve the underlying soils and, thus, provide an acceptable foundation for the driven piles, which would be considerably shorter as a result. **Figure 59.28** shows the selected combination of driven piles and vibroreplacement columns. Driven piles were generally installed at 2 m centres and the vibro stone columns were installed at centres ranging from 2.75 m to 3 m.

59.7.3 Execution controls

Execution control is limited to setting out and depth, measurement of stone consumption or the compaction energy (usually measured as hydraulic or electric power consumed). The above parameters can usually be measured by the rig instrumentation and provides an execution record for each column.

59.7.4 Validation

Validation is typically in the form of SPT or CPT testing prior to and post-testing in association with plate load tests of individual columns or zone testing on a group of columns (Greenwood, 1991). **Figure 59.29** shows the validation for the Barrow project in terms of CPT testing.

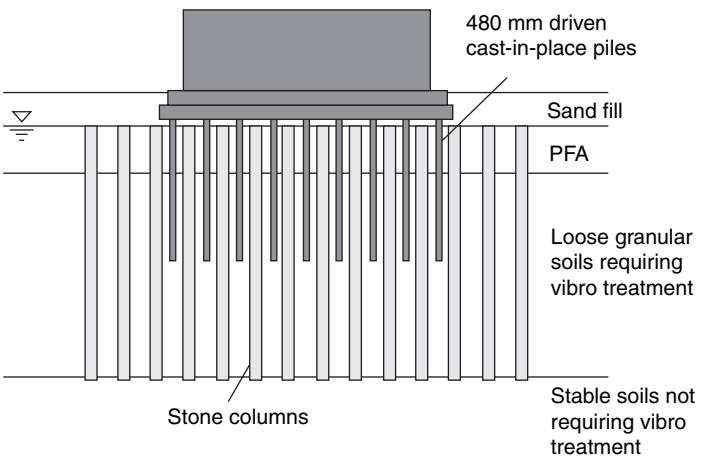


Figure 59.28 Proposed foundation improvement for Barrow
Reproduced from Raison *et al.* (1995)

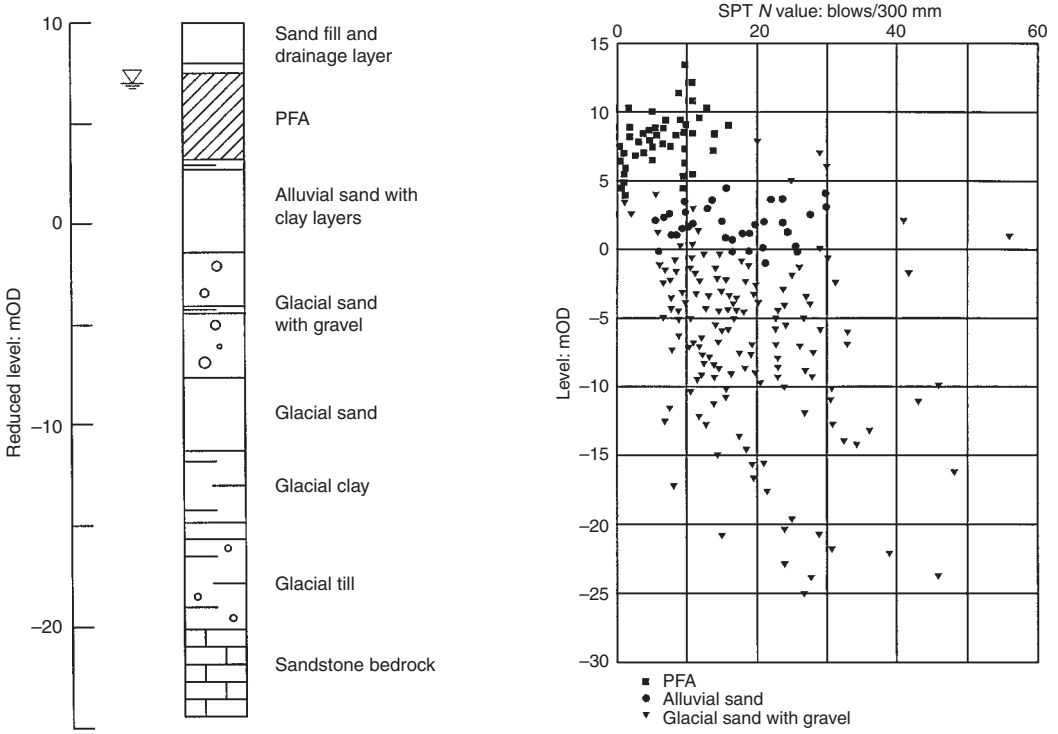


Figure 59.27 Ground conditions at Barrow
Reproduced from Raison (1999)

59.8 Design principles for dynamic compaction

Dynamic compaction is similar to vibrocompaction in that the intent is to compact the ground and, thus, increase stiffness. It is more effective in granular materials although it has been historically successful in finer materials.

Dynamic compaction involves the repetitive dropping of heavy weights at fixed grid points to compress the ground. Historically, Menard owned a patent for this work and carried out pioneering works in Europe and the Far East. The weight of their dynamic tampers were as heavy as 40 tonnes and supported on very high specially constructed tripods.

The design of dynamic compaction is based on the input of a minimum dynamic energy per volume of ground and is carried out in a series of phases or passes.

The maximum depth of treatment depends both on the weight of tamper and height of drop and is typically given by the following relationship

$$\text{Depth of treatment} = 0.5 \times (\text{height of drop} \times \text{weight})^{0.5}.$$

Thus, considering the use of a 20-tonne weight dropping 20 m, the depth of treatment would be limited to around 10 m. Fines content has a significant effect on compactive effort and care also needs to be taken if there are bands or layers of clay or silt, which will not be improved.

Typically, depending on the depth of treatment the compaction is carried out in three phases to ensure that the full depth of the soil is treated:

Pass 1

This will be based on widely spaced points and is designed to compact the deepest soils first. Spacing might be 7–10 m for a 10–12 m depth of treatment.

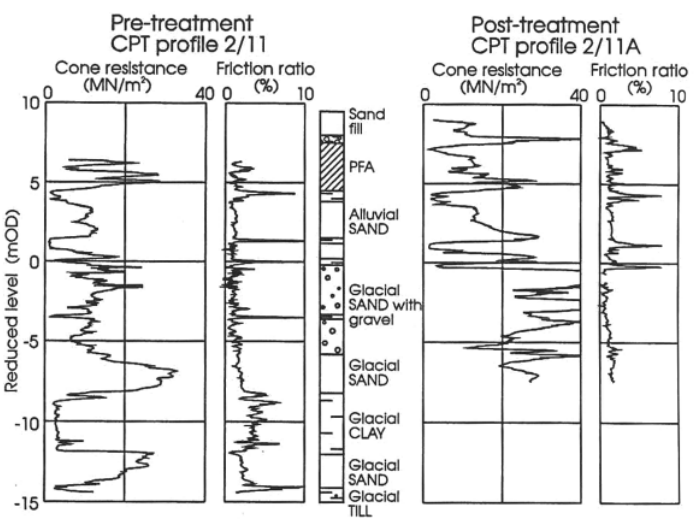


Figure 59.29 Results of pre- and post-CPT testing at Barrow
Reproduced from Raison *et al.* (1995)

Pass 2

This might be carried out on a 4–5 m spacing and is designed to compact the intermediate depths of ground below around 2 m.

Pass 3

This is a final pass on a close grid, which is almost the plan area of the tamper, and is designed to compact from 1 m to 3 m depth. Sometimes the tamper configuration will be changed to provide a larger area.

Often a final surface compaction is necessary to complete the densification.

As an approximate guide the application of around 25–30 tonne·m energy per cubic metre would provide a significant compaction of up to 60–70% relative density for 'clean' cohesionless soils (say less than 15% clay or silt content). **Table 59.8** is a guideline for the energies that might be required.

59.8.1 Methods and key issues

Key issues associated with dynamic compaction are the high vibration levels associated with the process, which can be detrimental to nearby structures. This means that dynamic compaction must occur at least 20–30 m from buildings (or perhaps 50 m from relatively sensitive buildings), although this would need to be established on a project-specific basis. Results of ground-borne vibrations are shown in **Figure 59.30**.

59.8.2 Execution control

Execution control is limited to ensuring that the correct energy is applied. This requires control of the height and the number of drops for each grid point. The craters formed by the process are usually dozed level and a further guide to the improvement made can be seen by surveying the reduction in ground level.

59.8.3 Validation

Validation is similar to vibrocompaction and replacement and typically consists of cone penetration or other *in situ* testing. Zone testing can also be carried out, although the limited depth of influence of the zone tests needs to be recognised compared with the proposed foundations.

Type of deposit	D_{50} (mm)	PI	Permeability range (m/s)	Total energy (tonne·m/m³)
Pervious coarse-grained, e.g. sand	> 0.1	0	$> 1 \times 10^{-4}$	< 20
Semi-pervious, e.g. silt	0.01 to 0.1	< 8	1×10^{-4} to 10^{-8}	< 30
Impervious above the water table, e.g. silt or clay	< 0.01	> 8	$< 1 \times 10^{-8}$	< 30
Landfill				< 50

Table 59.8 Applied dynamic compaction total energy guidelines

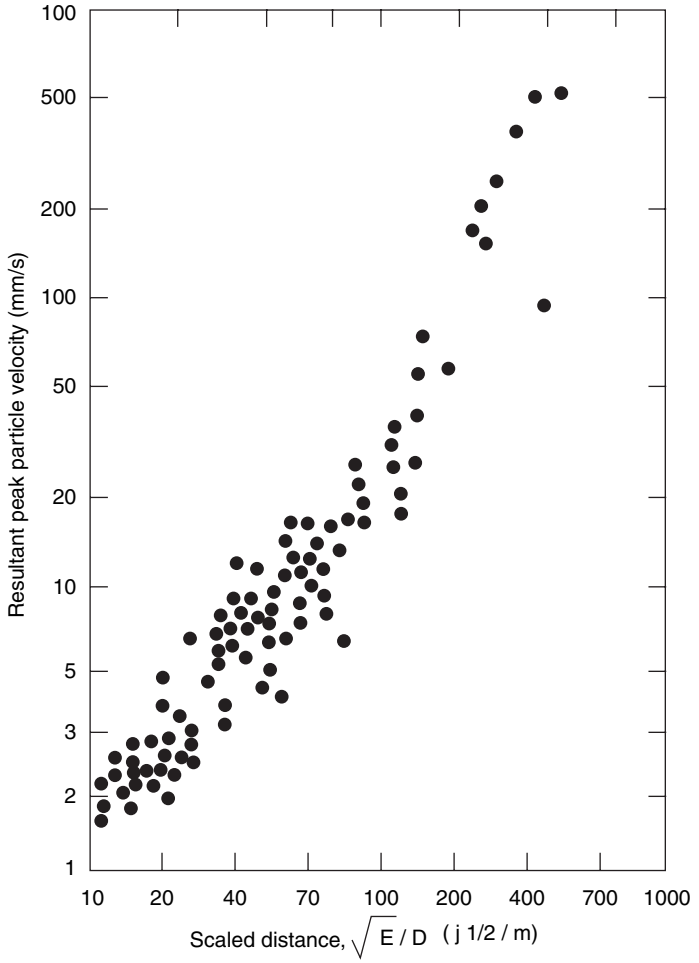


Figure 59.30 Ground-borne vibration levels caused by dynamic compaction
Reproduced from CIRIA C573 Mitchell and Jardine (2002) www.ciria.org

59.8.4 Sources of further information

Ground control and improvement

Demonstration Project 116, US Dept of Transport, Publication FHWA-SA-98–086R (2001)

Ciria Report C573 – A Guide to Ground Treatment

59.9 Design principle for deep soil mixing

Deep soil mixing (DSM) was first established by Intrusion-Prepakt in the mid-1950s and then further developed in Japan in the 1960s. The level of research and development activity in Japan in relation to DSM remains the highest in the world and annual DSM construction in Japan alone exceeds 1 million cubic metres of ground treated, with an accumulated total in excess of 25 million cubic metres.

The two generic forms of deep soil mixing are dry-soil mixing and wet-soil mixing. Dry-soil mixing consists of injecting a powder binder while mechanically disaggregating the soils. Wet-soil mixing consists of injecting a fluid binder while mechanically disaggregating the soils.

Dry-soil mixing is principally used in very soft to soft clays, silts and peats because the equipment was historically used for these soils, whereas wet-soil mixing can be used in more granular and stiffer soils.

Dry-soil mixing tends to remould the material while injecting the binder, while wet-soil mixing is designed more to try and mix in the binder to create a homogeneous material.

59.9.1 Design methods and key issues

The design of soil mixing is based on the required composite shear strength of the improved soils to achieve an appropriate factor of safety using limit equilibrium analysis.

For vertical support, columns can be utilised in interlocking patterns or as isolated columns. Where applied to resist shear, isolated columns are not recommended as their bending resistance is low and, thus, are typically utilised as panels of soil-mixed columns (typically 600 mm diameter at 500 mm centres perpendicular to the shear direction). Some configurations are shown in **Figure 59.31**.

The composite shear strength is based on the following equation:

$$C_{u\ (\text{composite})} = a\ .\ C_{u\ (\text{column})} + (1–a)\ C_{u\ (\text{soil})} \qquad (59.5)$$

where a is the area replacement ratio of the treated ground. Hence, if the strength of the soil-mixed columns $C_{u\ (\text{column})} = 150$ kPa, the strength of the untreated material $C_{u\ (\text{soil})} = 20$ kPa and a composite shear strength of around 50–60 kPa was needed then an area replacement ratio of about 25–30% would be required. This would correspond to panels with 600mm-diameter columns (at 500 mm centres) spaced about 1.6 m to 2 m apart.

The detailed design should establish the area replacement ratio and soil-mixed column strengths required.

For drained, analysis typical parameters are:

$$c'_{(\text{column})} = bC_{u(\text{column})} \text{ and } \phi'_{(\text{column})} = 30°\ . \qquad (59.6)$$

Typically b is around 0.3; this value can be used for the initial design but should be established for individual site conditions.

As with all ground-improvement techniques, both laboratory and field trials are extremely important to confirm the design requirements.

Peats and organic clays are especially difficult to improve due to the high sulphate and acidic water content, which can depress cement hydration and lower the strengths of the mixed materials.

Most soil mixing is designed on the basis of consideration of the final *in situ* water/cement ratio as this ultimately affects the strength development as defined classically by Abrams in the 1920s.

Figure 59.32 shows data obtained by Swedish researchers for various soils and water/cement ratios.

To estimate the required binder content, the initial moisture content of the soil is required together with any water added

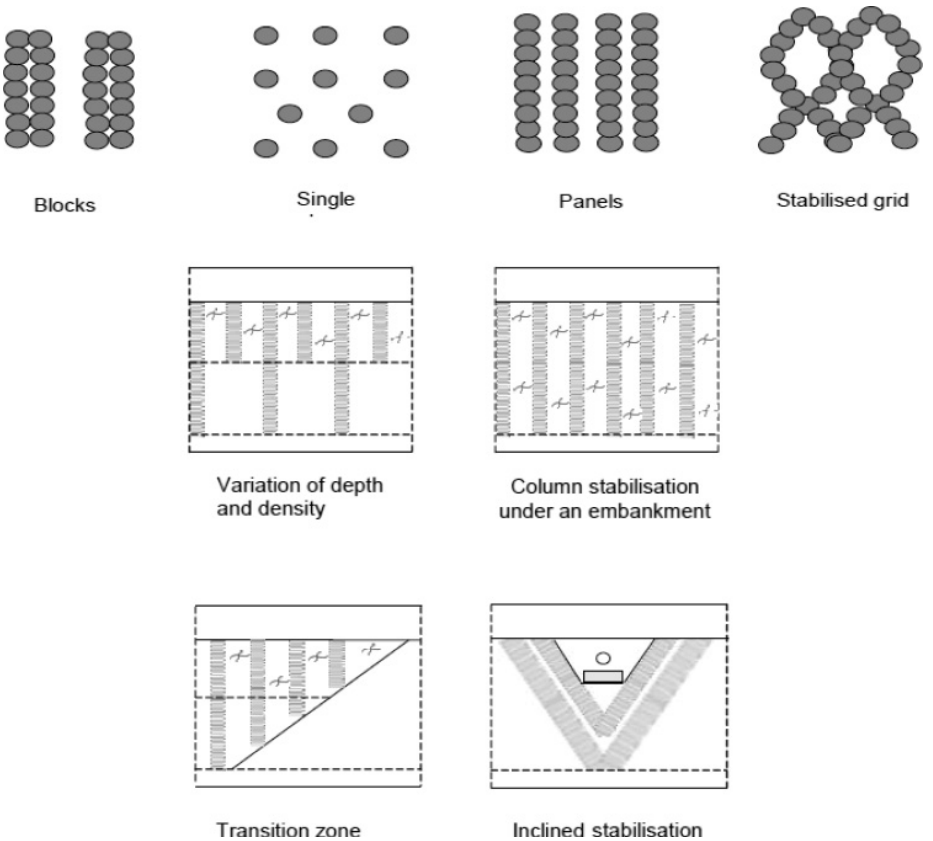


Figure 59.31 Examples of soil-mix column layout
Reproduced from EuroSoil Stab (2002); IHSBRE Press

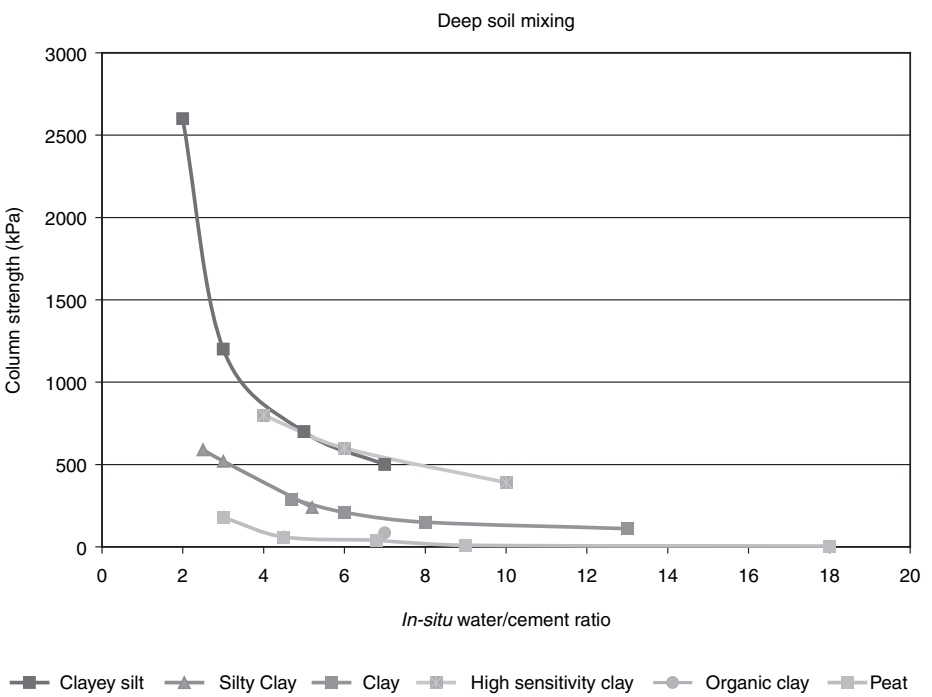


Figure 59.32 Expected soil-mix column strengths based on *in situ* water/cement ratio
Data taken from Åhnberg *et al.* (1995)

from the binder (if fluid) and the overall water/binder ratio is calculated based on a design addition of binder.

For peats and organic clays, a binder containing ground granulated blast furnace slag (GGBFS) is preferred as this binder tolerates sulphates and acid, and it provides a higher strength.

One of the first successful large-scale uses of dry-soil mixing in the UK was carried out within the Port of Tilbury. Overloading of an aggregate stockpile caused a failure within the underlying soft clays and resultant failure of the quay wall. **Figure 59.33** shows the cross-section of the mode of failure. A number of solutions were considered based on piling, vibroreplacement and soil mixing and it was decided that dry-soil mixing offered the most cost-effective and technically acceptable solution. Dry-soil mixing in panels was carried out to improve the soft clays and provide support to the aggregate stockpile. To create panels, 800-mm-diameter columns were installed to the top of the underlying gravels with columns spaced at 700 mm. Individual panels were spaced at 2.3 m to 2.8 m centres. The replacement volume for the soil-mixed columns varied from 25% to 33% assuming a column shear strength of 150 kN/m^2. Over 3000 columns were installed and the design was verified using an initial trial where columns were tested *in situ* and also exhumed. Extensive production testing was carried out using both CPT and pull-out testing and this is described by Lawson *et al.* (2005). **Figure 59.34** shows a comparison between CPT, pull-out testing (PORT) and laboratory testing. In general, the field tests gave similar results whereas the laboratory testing gave much lower results, which was attributed to sample disturbance.

59.9.2 Execution controls

Execution controls consist of controlling the binder content during mixing and most specialists provide instrumentation on the rigs that record all important parameters.

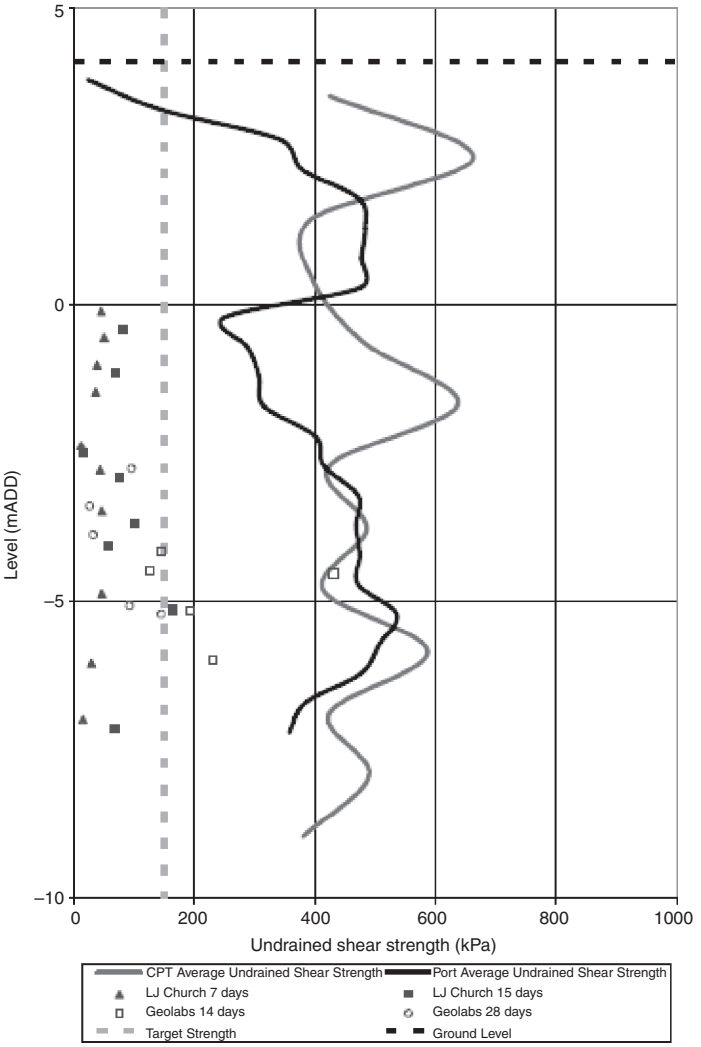


Figure 59.34 Results of testing for dry-soil mixing, Tilbury
Reproduced from Lawson *et al.* (2005)

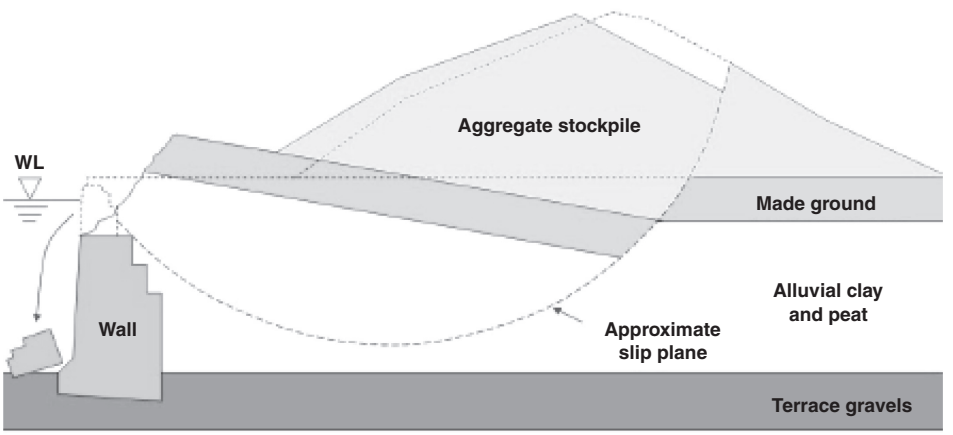


Figure 59.33 Failure of quay wall at Tilbury
Reproduced from Lawson *et al.* (2005)

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For wet-soil mixing additional controls are required for grout batching as for other forms of grout mixing.

59.9.3 Validation

Validation is generally by *in situ* testing. This can be cone penetration testing, plate testing or zone testing. Additionally for dry-soil mixing a special vane penetration and pull-out test has been developed. The vane, which is around 0.45 m wide, is inserted below the mixing head with a cable running up the Kelly bar. The mixing head is inserted as usual and the column is mixed, leaving the vane in place centrally and immediately below the column. After a set period of time, normally between three and seven days, the vane is withdrawn at a fixed rate of 2 cm/s and the extraction load monitored. This can then be related to the *in situ* strength using empirical relationships. Care needs to be taken to ensure that cable friction is taken into account and it is common for the vane to be withdrawn around 0.5 m within 24 hours of column construction to break or reduce this friction.

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59.10.1 Further reading and useful websites

Xanthakos, P. P., Abramson, L. W. and Bruce, D. A. (1994). *Ground Control and Improvement*. USA: John Wiley & Sons, Inc. An excellent book; covers most forms of ground improvement in considerable detail and although it may not cover the most up to date innovations remains a source of significant knowledge.

59.10.1.1 Design principles for void filling

Building Research Establishment. (2006) *Stabilising Mine Workings with PFA Grouts. Environmental Code of Practice,* BRE Report 488. UK: BRE, Bracknell.

The Mining Records Office; www.coal.gov.uk/services/history/index.cfm

59.10.1.2 Design principles for compaction grouting

There are many specialist conference proceedings published in the Unites States and this is the primary source of information.

American Society of Civil Engineers, The (ASCE) has published a number of special publications:

Johnsen, L. F., Bruce, D. A. and Byle, M. J. (eds) (2003). Grouting and ground treatment. In *Proceedings of the Third International Conference*, New Orleans. Geotechnical Special Publications (GSP) 120. USA: ASCE.

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59.10.1.4 Design principles for vibrocompaction and vibroreplacement and dynamic compaction

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59.10.1.5 Design principles for deep soil mixing

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EuroSoilStab *Design Guide: Soft Soil Stabilisation,* CT97–0351 Project No.: BE 96–3177 (2002). UK: BRE Press.

Swedish Geotechnical Institute. *Design of Lime-Cement Columns*, Report 4/95.

It is recommended this chapter is read in conjunction with

- Chapter 25 *The role of ground improvement*
- Chapter 84 *Ground improvement*
- Chapter 100 *Observational method*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 60

Foundations subjected to cyclic and dynamic loads

Milutin Srbulov Mott MacDonald, Croydon, UK
Anthony S. O'Brien Mott MacDonald, Croydon, UK

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The basic concepts and references concerning shallow and deep foundation response to earthquake, wave and machinery effects are introduced. The information provided in this chapter will be useful to many readers because available recommendations in the codes and standards concerning the response of foundations subjected to cyclic and dynamic loads is limited with regard to the requirements encountered in practice. Soils tend to respond nonlinearly even at rather small strain. Nonlinear dynamic analysis and testing can produce chaotic results and the best approach to a problem solution is to understand the fundamental issues associated with the problem, have an idea about the possible range of results and know the limitations of analysis methods used for its solution.

60.1 Introduction

Foundations can be subjected to large cyclic and/or dynamic loads in the following situations:

(i) seismic loads triggered by earthquakes;

(ii) wave forces imposed on offshore (marine) structures;

(iii) foundations which support vibrating machines.

Cyclic load effects may also need to be considered for tall slender structures subject to wind loading, although the cyclic loads will tend to be relatively minor compared with the situations for (i) to (iii) above. The design of foundations which are subjected to cyclic/dynamic loads is a complex matter and should only be carried out by appropriately qualified specialists. Hence, the intent of this chapter is to give a brief introduction to the topic and provide a list of technical references which give more detailed guidance. The adverse effects of cyclic/dynamic loading include:

(i) degradation of ground strength, which can lead to failure at foundation loads below those expected based on 'static' strength;

(ii) degradation of ground stiffness and/or 'ratcheting' type effects, leading to an accumulation of permanent foundation displacements;

(iii) dynamic loading can induce site/structure-specific amplification of loads/movements due to resonance type phenomenon;

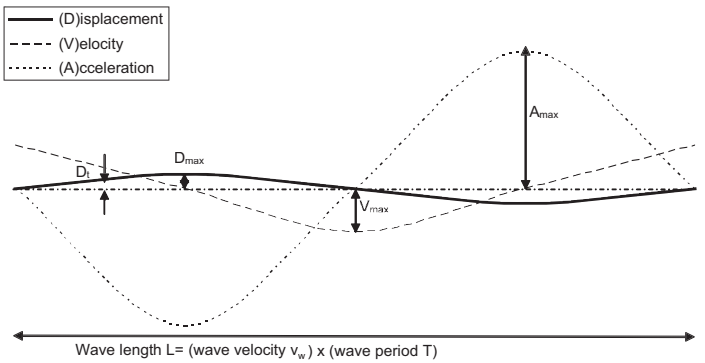
(iv) earthquake loads can induce large-scale ground instability which will impose large additional forces on foundations or cause catastrophic collapse.

It is noted in Chapters 52 *Foundation types and conceptual design principles* to 55 *Pile-group design*, inclusive, that the main body of experience is for foundations which are loaded vertically. A feature of cyclic/dynamic loading is that it commonly involves large horizontal loads, and therefore it is far more difficult to predict foundation behaviour. Hence, a more cautious approach to foundation design is required.

60.2 Cyclic loading

Vibration of foundations occurs when loads on them (from machinery) or their base displacements (from earthquakes) vary in time or a structure is forced out of its equilibrium position (by blast) and left to oscillate freely. In the case of dynamic loads, the inertial forces are significant in comparison with the acting static loads. More details about dynamic and seismic loading of soil are given in Chapter 24 *Dynamic and seismic loading of soils*. The main parameters of cyclic behaviour are:

- **Amplitude of vibration** indicates how much a cyclic state differs from the steady state condition of a body (**Figure 60.1**). Acceleration amplitude is directly proportional to the inertia force acting on a body. Square of velocity amplitude is directly proportional to the kinetic energy of a vibrating body. Displacement amplitude of a vibration is proportional to the amount of deformation (strain) in cyclic condition (strain is the ratio between induced displacement and length over which such displacement has been achieved). The ratio between particle and wave propagation velocity equals the strain induced by unidirectional vibration within a body. Product of unit density, particle velocity and wave propagation velocity equals the stress acting within a body during its unidirectional vibration, e.g. Kramer (1996), Timoshenko and Goodier (1970) (stress is the ratio between applied force and the area over which it has been applied).
- **Vibration period** is the time difference between two equal states of body vibrations. Product of vibration period and wave propagation velocity equals to wave length. Vibration period, i.e. its reciprocal value called vibration frequency, is important. If the vibration period of free oscillation of a body corresponds to the vibration period of vibration excitation then resonance could occur, which



Shear strain γ_t at time t = (displacement D_t at time t)/(v_w x t) = (particle velocity v_{pt} at time t)/ (wave velocity v_w)
Shear force T_t at time t = (mass m) x (acceleration A_t at time t)
Shear stress τ_t at time t = T_t /(cross sectional area)= (unit density ρ) x v_{pt} x v_w
Kinetic energy E_t at time t = (mass m) x (particle velocity V_{pt}^2 at time t)/2

Figure 60.1 A sketch of sinusoidal transversal wave with amplitude related values

can lead to the increase of the amplitudes of the source of vibration and damage or destruction of the oscillating body. In the case of harmonic (sinusoidal) vibration caused by machinery, the maximum amplitudes of displacement D, velocity V and acceleration A are related to each other as $V = \omega D$, $A = \omega V = \omega^2 D$, where ω is so-called circular frequency $= 2\pi f$, f is vibration frequency. Earthquakes are chaotic and the predominant periods (frequencies) of acceleration, velocity and displacement are different.

- **Number of cycles** (i.e. duration of vibration) is important because longer vibration duration can cause greater damage to materials (due to so-called fatigue effect) than a few vibration cycles if the amplitudes are the same. Prolonged vibration causes (micro) cracks even in metals and in soil it can cause a number of problems such as state of liquefaction, flow type failure of slopes, loss of bearing capacity as well as excessive settlements and horizontal displacements of foundations. Seed *et al*. (1975) developed the concept of an equivalent number of significant stress cycles (caused by the horizontal ground acceleration) to represent an irregular time history of shear stresses by a uniform series of harmonic (sinusoidal) stress cycles. The equivalent number of uniform stress cycles N_{eqv} was selected to cause pore pressure build-up equivalent to that of an actual shear stress history at harmonic stress amplitude of 65% of the maximum actual shear stress (caused by the peak horizontal ground acceleration). Seed *et al*. (1975) data can be approximated by a simple formula $N_{eqv} = 0.0008\, M_L^{4.88}$, where M_L is local (Richter) earthquake magnitude. Other researchers related N_{eqv} to different magnitude types (e.g. Hancock and Bommer, 2004) and showed that N_{eqv} depends also on the site-to-source distance and depth below ground level (e.g. Green and Terri, 2005), i.e. soil type. Sarma and Srbulov (1998) used a large number of actual acceleration time histories caused by earthquakes to show that 95% of energy contained within a strong motion recorded corresponds to the level of acceleration equal to about 0.65 of the peak acceleration.

An example of the effect of soil liquefaction on loss of bearing capacity of shallow foundations and tilting of the whole buildings is shown in **Figure 24.12** of Chapter 24 *Dynamic and seismic loading of soils*. In other cases, buildings sunk vertically or pipes and shafts floated due to soil liquefaction. Piles can be damaged or destroyed in liquefied soil when a significant lateral movement of ground occurs due to flow type failures. A devastating effect on the piled foundation caused not by soil liquefaction but by 'double resonance' condition (amplification of bedrock motion by the soil deposit and amplification of the soil motion by the structure) is shown in **Figure 60.2**. The peak horizontal bedrock acceleration was only 0.03 g to 0.04 g but has been amplified several times (e.g. Kramer, 1996).

60.3 Earthquake effects

60.3.1 Overall design considerations

The design process would normally involve the following main steps:

(i) Assess the earthquake magnitude and source location. Often several 'design' earthquakes will be considered, varying from a relatively small event (for this the foundation would be expected to remain serviceable with minimal repair requirements) to a large event (for which there would be a 'no collapse' requirement). The earthquake properties will be defined for bedrock level.

(ii) Assess the overall topography in the vicinity of the proposed structure, in particular location of slopes/embankments and cuttings/ditches/river banks, etc.

(iii) Assess local amplification/attenuation of bedrock earthquake motion through overlying soil deposits.

(iv) Assess whether soil liquefaction may occur.

(v) Assess consequences of soil liquefaction, which may vary from local settlement to global failure.

(vi) Assess dynamic soil–structure interaction effects.

(vii) Depending upon (v) or (vi), assess the type and extent of deep ground improvement to mitigate the potential adverse effects.

60.3.2 Soil behaviour

A detailed description of soil behaviour in cyclic condition is given in Chapter 24 *Dynamic and seismic loading of soils*. In summary, both loose to medium dense sand and silt and soft to firm clay can exhibit a decrease of shear strength and stiffness with increase in the number of cycles and cyclic amplitudes. The loss of shear strength of sands during earthquakes is known as liquefaction. Liquefaction has been the cause of catastrophic and extensive structural damage and loss of life. Liquefaction is induced by the generation of significant excess pore pressures during an earthquake, which results in a loss of shear strength and stiffness. Following earthquake shaking, the excess pore water pressures will dissipate as seepage takes place between zones of high excess pore water pressure to zones of low excess pore water pressure. The resulting upward groundwater flow

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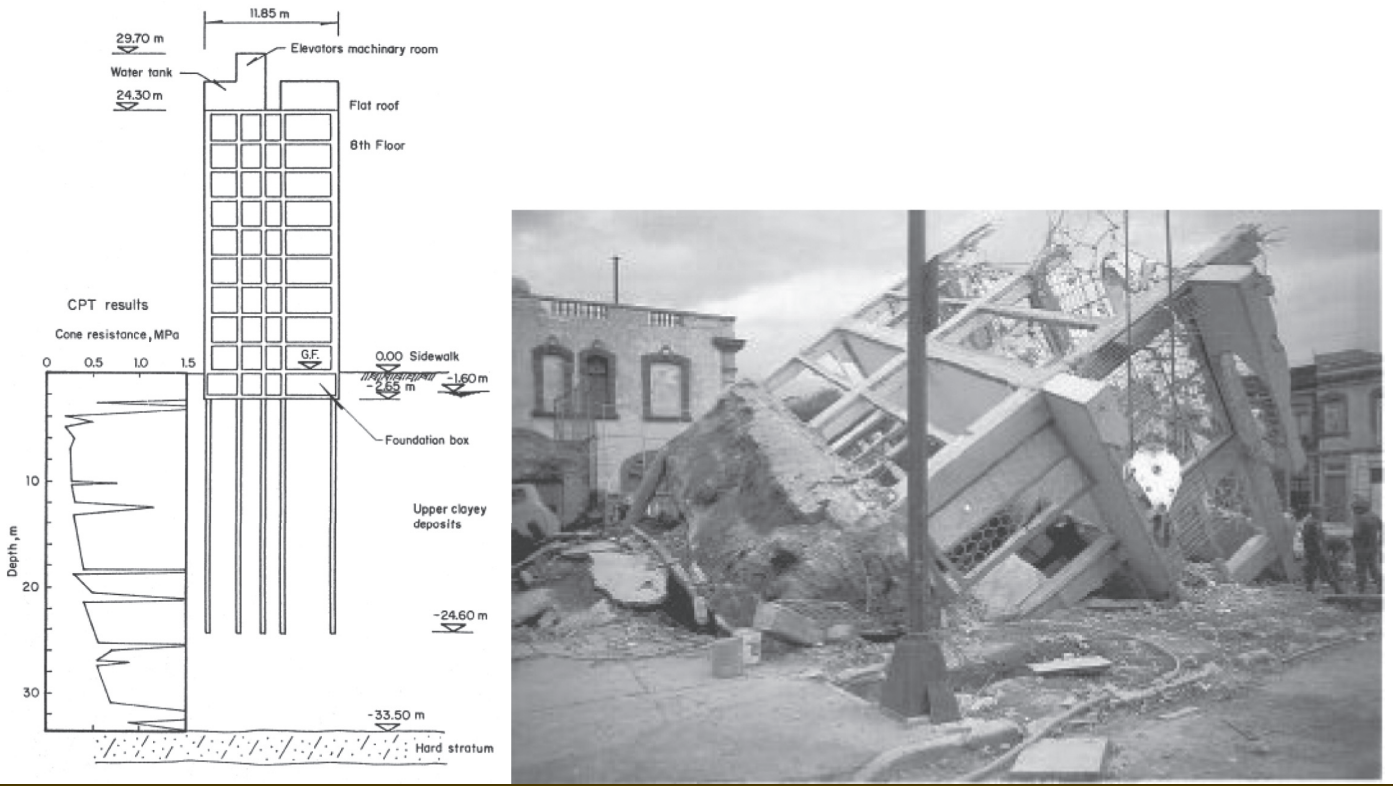


Figure 60.2 Elevation of the ten-storey building in Mexico City and after toppling
Reproduced from Meymand (1998)

greatly complicates the assessment of the effects of an earthquake, and cannot be reliably quantified by laboratory testing or computer-based analyses. Considerable engineering judgement is required. An important factor is the 'layering' of the soil deposits. The presence of a clay layer in loose sand could cause global instability, as upward seeping water becomes trapped beneath the clay seam leading to reduced soil shear resistance. The factors which influence the risk of liquefaction are reasonably well understood and include:

(1) **Soil type and ground water table level**: saturated coarse-grained soils (particularly fine/medium sands and coarse silts) in areas with a high water table level are at most risk. In general, the younger and looser the deposit the more vulnerable it will be to liquefaction. In general, deposits which are of Pleistocene age or older are usually considered to be at low risk. Dense coarse-grained soils, with a relative density greater than 80% are not likely to liquefy.

(2) **Depth**: with increasing depth, the confining pressure increases and liquefaction resistance increases. Commonly observed liquefaction depths are less than about 15 m. There are no recognised 'maximum' depths for liquefaction, although in general the risk of liquefaction below 20 m or 25 m depth is believed to be low.

(3) **Drainage conditions and soil fabric**: poor drainage allows pore pressure build-up and increases the risk of liquefaction; and as noted above, if loose sands are sandwiched between clay layers the loss of strength may be more serious due to seepage effects.

(4) **Earthquake-induced shear stress**: as the intensity of ground shaking increases the shear stress increases and liquefaction is more likely.

(5) **Earthquake duration**: as duration increases (generally a function of earthquake magnitude) the number of loading cycles increases, leading to an increase in excess pore water pressure.

Simplified empirical analysis procedures have been developed, and the factor of safety against liquefaction = CRR / CSR (e.g. refer to Youd *et al.*, 2001; BS EN1998-5:2004); CRR = cyclic resistance ratio of the soil at a particular depth and CSR is the cyclic shear stress ratio imposed by the earthquake at a particular depth. The CSR is usually calculated by using Seed and Idriss' simplified equation. For more challenging situations ground response analyses (GRAs) can be carried out, but GRAs should only be carried out by seismic experts. Seed and Idriss' simplified equation is:

$$\mathrm{CSR} = 0.65 \frac{a_{\max}}{g} \frac{\sigma_v}{\sigma'_v} \mathrm{r_d} \qquad (60.1)$$

In the above equation, $a_{\max}$ is the peak ground surface acceleration for the design earthquake, σ_v and σ'_v are total and effective

vertical stress respectively, r_d is a stress reduction factor which is depth-dependent. For simple projects, the value of a_{max} at ground surface can be estimated from the local seismic codes (http://www.iaee.or.jp/), which provide the peak horizontal accelerations at bedrock but also amplification factors for different soil types. The liquefaction resistance can be estimated empirically from *in situ* tests (commonly standard penetration tests (SPT) and cone penetration tests (CPT)), e.g. Figure B.1 of BS EN1998-5:2004, or from laboratory tests. The use of laboratory tests to assess CRR is challenging and requires very high quality samples and specialist laboratory testing (e.g. cyclic simple shear tests). It can be useful for sandy soils with high fines content (e.g. sandy silts or very silty sands), or sands/silts which are suspected to be overconsolidated, and where empirical SPT/CPT based methods may be overly conservative. Comprehensive guidance on assessing liquefaction risks is given in Youd *et al*. (2001) and Kramer (1996).

Clays and cohesive silts can suffer a loss of strength/stiffness during earthquake shaking, usually termed 'cyclic mobility'. Fundamentally the overconsolidation ratio and strength sensitivity (ratio of peak to remoulded strength) are important factors. Bray *et al*. (2004) have suggested the following classification based on moisture content (w), liquid limit (w_L) and plasticity index (I_p):

(a) $w/w_L \geq 0.85$ and $I_P \leq 12$: susceptible to liquefaction or cyclic mobility*;

(b) $w/w_L \geq 0.8$ and $12 < I_P < 20$: moderately susceptible to liquefaction or cyclic mobility*;

(c) $w/w_L < 0.8$ and $I_P \geq 20$: no liquefaction or cyclic mobility, but may undergo significant deformation if cyclic shear stress > Static undrained shear strength (S_u).

* This classification may be revised on a site-specific basis using data from laboratory cyclic shear testing of good quality field samples (e.g. samples obtained using thin-walled tube samples with sharpened (i.e. >5°) cutting edge and no inside clearance).

Bray *et al*. (2004) suggest that the residual strength (S_r) for silt and clay zones be determined as per guidelines given below:

(a) $w/w_L \geq 0.85$ and $I_P \leq 12$:S_r = remoulded shear strength ($S_{remoulded}$), unless appropriate testing of undisturbed samples can show greater strength;

(b) $w/w_L \geq 0.8$ and $12 < I_P < 20$: $S_r = 0.85s_u$ where s_u = static undrained shear strength;

(c) $w/w_L < 0.8$ and $I_P \geq 20$: $S_r = s_u$.

Coarse-grained soils like gravel can also liquefy in certain circumstances based on case studies (e.g. Kokusho *et al*., 1995). Dense to very dense coarse granular soil and overconsolidated fine-grained soil tend to dilate initially during shearing (i.e. after initial cycles) and exhibit greater shear resistance and stiffness than normally consolidated soil.

Dynamic soil–structure interaction effects are dependent on the variation of soil shear modulus with depth and with strain amplitude. Empirical correlations are available for G_{max} (or G_o) (refer to Chapter 52 *Foundation types and conceptual design principles*); however, whenever feasible site-specific *in situ* measurements of G_{max} should be carried out (using, for example, seismic-cone or down-hole geophysics tests). The variation of shear modulus with strain amplitude is usually based on cyclic triaxial tests (ASTM D3999-91) or resonant column tests (ASTM D4015-92). When the results of tests of soil stiffness are not available, BS EN1998-5:2004 provides a table of average soil damping ratios and average reduction factors (± one standard deviation) for shear wave velocity v_s and shear modulus G (for upper 20 m depth range) as a function of peak horizontal ground acceleration, which are plotted in **Figure 60.3**.

When the results of tests of shear strength of sandy soil in cyclic condition are not available, the frictional angles of sandy soil φ can be inferred from the critical cyclic stress ratios τ/σ' (= tanφ) that were proposed initially by Seed and Idriss (1971) as the boundaries between liquefied and non-liquefied

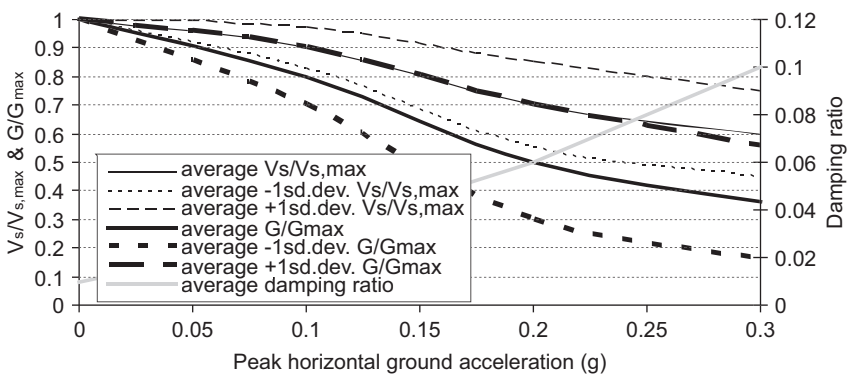


Figure 60.3 Average soil damping ratios and average reduction factors (± one standard deviation) for shear wave velocity v_s and shear modulus G within 20 m depth

Reproduced with permission from BS EN 1998-5 © British Standards Institution (2004)

sandy soil and later included in BS EN1998-5:2004 (**Figure 60.4**). For dense sand, the friction angle in cyclic condition is assumed to be equal to the angle in static condition, which can be inferred from e.g. Peck *et al*. (1974).

60.3.3 Earthquake loading

In most cases, earthquakes are caused by sudden slip of a tectonic fault or tectonic plate in the subduction zone. Earthquake magnitude represents a measure of the energy released at the source unlike earthquake intensity, which is a measure of earthquake effects at a site. Besides active tectonic faults, earthquakes can be caused by large landslides, cave collapse, volcanoes and explosions. There are different types of earthquake magnitudes.

- Richter local magnitude M_L (Richter, 1935) is the logarithm of the maximum trace amplitude (in micrometers) recorded on a Wood-Anderson seismometer located 100 km from the epicentre of an earthquake.
- Surface wave magnitude M_s (Gutenberg and Richter, 1936) is a world-wide magnitude scale based on the amplitude of Rayleigh waves with a period of about 20 s.
- Body wave magnitude m_b (Gutenberg, 1945) is a world-wide magnitude scale based on the amplitude of the first few cycles of longitudinal waves (usually with the period of about 1 s).
- Moment magnitude M_w (Kanamori, 1977; Hanks and Kanamori, 1979) is proportional to the logarithm of seismic moment M_o, which is a product of a tectonic fault rupture area, the average amount of fault slip and the rock stiffness.
- Other types of magnitudes exist but are less frequently used. Correlations between various common magnitude scales were plotted by Idriss (1985).

Ambraseys (1990) derived the following relationships between various common earthquake magnitude scales:

$$\begin{aligned} 0.77 \cdot m_b - 0.64 \cdot M_L &= 0.73 \\ 0.86 \cdot m_b - 0.49 \cdot M_S &= 1.94 \\ 0.80 \cdot M_L - 0.60 \cdot M_S &= 1.04 \end{aligned} \qquad (60.2)$$

Maximum amplitudes of ground motion caused by earthquake at a site can be estimated from various attenuation relationships (e.g. Douglas, 2011) or from national seismic codes (www.iaee.or.jp/). The published attenuation relationships do not consider one or more of the following important factors affecting earthquakes:

- tectonic fault type (normal, strike-slip, reverse, oblique);
- seismic wave path (site-to-source distance, rupture directivity and fling step in the near field);
- sediment basin depth and near edge effect;
- local soil layer thickness and stiffness;
- topography (slopes, ridges, canyons);
- wave bounce from Moho surface (i.e. the Earth's crust and mantle boundary).

When the above factors have not been considered in the attenuation relationships, their effects are accounted for by adding one standard deviation to the average value of ground motion amplitude, which is predicted by the attenuation relationship. More information on the effects of these factors is provided by Stewart *et al.* (2001).

Seismic wave amplitudes and wave periods (lengths) affect both foundations and structures above them. Foundations tend to average ground wave amplitudes along foundation lengths (Yamahara, 1970; Newmark *et al*., 1977) because of different stiffness of foundations and underlying soil and in this process undergo additional stressing. Averaged amplitude values are smaller than the peak values but wave amplitude averaging causes also additional rotation (Newmark, 1969). Seismic body waves are considered to propagate nearly vertically under ground surface because of a number of their refractions at the layer boundaries along their path from the source to the surface. Vertically propagating body waves exhibit phase shift when soil layers through which they propagate have different thicknesses so the waves arrive at the surface with different amplitudes spatially at time instants. Piles cause averaging of amplitudes of seismic body and near-surface propagating waves. Shallow foundations cause averaging of amplitudes of near-surface propagating waves such as Reyleigh and Lowe waves as well as of the amplitudes of body waves when their amplitudes at the surface vary spatially at time instants. For very long structures,

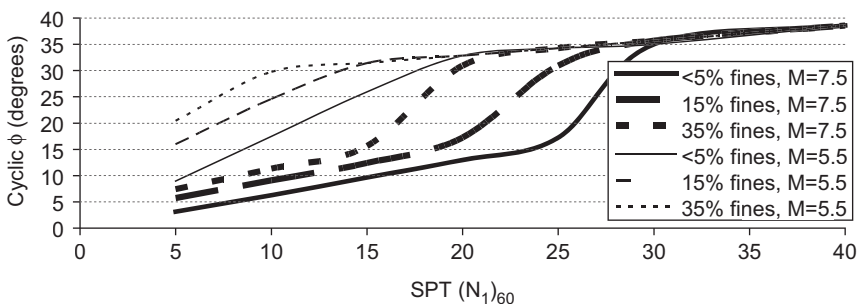


Figure 60.4 Dependence of friction angle π in cyclic condition on normalised SPT blow count $(N1)_{60}$ and earthquake magnitude M
Data taken from BS EN 1998-5:2004 and Peck, Hanson and Thornburn (1974)

phase shift in the vibrations of the foundations at the ends is considered. Foundations are also affected by inertia forces caused by ground acceleration and foundation mass but such forces may be relatively small in comparison with structural inertial forces if the structural mass and acceleration are significantly greater than foundation mass and acceleration.

Structural vibration can be amplified or attenuated with respect to the foundation vibration depending on the fundamental vibration period (frequency) of a structure. A damped single degree of freedom oscillator (assumed to be 'elastic') on a rigid base is used to define the so-called response spectra, which describe the maximum response of the oscillator to a particular input ground motion as a function of the fundamental frequency (period) and damping ratio of the oscillator. The smoothed envelope of many response spectra are given in seismic design codes for the design of structures. The effect of local soil layers on design spectral values is usually considered by application of multiplying coefficients to bedrock spectral values for different soil classes. BS EN1998-5:2004 (BSI, 2004) specifies in which cases site-specific dynamic soil–structure interaction analyses must be considered and what factors such analyses need to involve but not how the analyses are to be performed. Soil–structure interaction generally increases the period of vibration and damping ratio of structures (e.g. ATC, 1978; BSSC, 1995). The increase in the structural period may appear to be beneficial according to smoothed design spectra, whose amplitude decreases with the period increase after a threshold period usually in the range of 0.5 s to 1.0 s. However, site-specific acceleration spectra like the ones in Mexico City (e.g. Kramer, 1996) and in Kobe (e.g. Gazetas and Mylonakis, 1998) indicate that an increase in structural vibration period can be responsible for structural damage due to resonance effects between the vibration of a structure and soil layers, when the period of vibration of the soil deposit is relatively large (e.g. 2 s in Mexico City and 1 s or more in Kobe). Stewart *et al*. (1999) evaluated soil–structure interaction effects for 77 strong motion data sets at 57 building sites and found a pronounced influence of structure-to-soil stiffness ratio on inertial interaction, as well as secondary influences from structure aspect ratio and foundation embedment, shape and flexibility. They found that kinematic interaction effects on the 'input' motion at the bases of structures are found to be relatively modest in many cases, whereas inertial interaction effects on the structural response to these motions can be significant. Wolf (1994) provides simplified expressions for the calculation of an equivalent period T_e of soil–foundation–structure systems. Some codes also provide simplified expressions. These can be useful in order to assess resonance type effects.

Seismic codes specify only amplification of bedrock acceleration by overlying soil layers. Srbulov (2003) plotted the ratios between the horizontal peak accelerations at the surface and at depth (equivalent to 'bedrock') shown in **Figure 60.5** from 66 recorded case histories. This shows that peak ground surface acceleration is attenuated for peak horizontal accelerations at depth (i.e. bedrock) greater than 0.3 g to 0.4 g. Attenuation of large bedrock accelerations is most likely because of soil yield at greater accelerations. Idriss (1990) also indicated attenuation of bedrock acceleration on soil sites when the acceleration on rock sites exceeded about 0.3 g to 0.4 g (but Idriss' relationship is based on calculations rather than recorded data).

60.3.4 Global failure/deformation mechanisms

During earthquakes, the propagation of seismic waves causes cracking and elevation differences even on level ground. However, the greatest ground deformations are caused by slope failures, rupture of ground surface by tectonic faults and soil liquefaction followed by flow type failures.

Keefer (1984) studied the effects of 40 historical earthquakes on slope failures. The types of failures, their relative

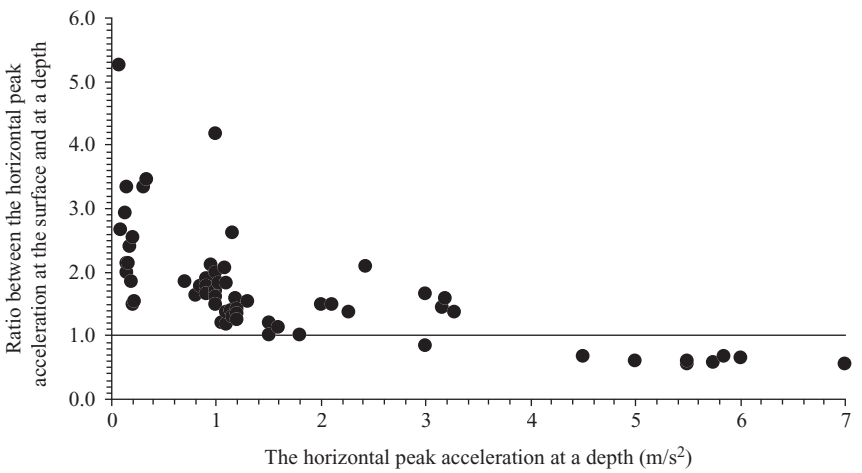


Figure 60.5 Ratios between the recorded horizontal peak accelerations at the surface and at depths (soil or rock) versus the horizontal peak accelerations at depths

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frequency and minimum triggering earthquake magnitudes are given in **Table 60.1**.

Keefer (1984) and Rodriguez *et al*. (1999) plotted the graphs of the maximum recorded distances of occurrence of landslides depending on their type and earthquake magnitudes. Their graphs indicate that no slope failures are expected beyond 120 km epicentral distance even from earthquakes with the magnitudes up to 8 and that disrupted landslides are more prone to earthquake triggering than coherent landslides, spreads and flows. More information on slope stability issues is provided in Chapter 23 *Slope stability* and on slope stabilisation methods in Chapter 72 *Slope stabilisation methods*.

If a tectonic fault causes a rupture of ground surface, it results in destruction of structures crossing the fault and therefore placement of structures over active faults must be avoided wherever such faults exist.

If liquefaction occurs then the consequences can vary enormously from modest ground settlement to catastrophic large-scale ground instability. A range of factors will influence the nature of any consequences including: the earthquake characteristics; site stratigraphy and topography; nature of structures (and their foundations) and earthworks on the site. Three important consequences are:

(i) reduction of soil shear strength, leading to global instability of the ground (e.g. embankment collapse leading to failure of adjacent pile foundations) and increase in lateral pressure on retaining structures;

(ii) settlement caused by reconsolidation of liquefied soil;

(iii) lateral deformation of gently sloping ground.

Figure 60.6 indicates how site-specific features and the spatial distribution of liquefied zones can affect the consequences of liquefaction.

Type of slope failure	Frequency of occurrence during earthquakes	Minimum triggering earthquake magnitude M_L
Rock falls, disrupted soil slides, rock slides	Very frequent	4.0
		4.0
Soil slumps, soil block slides	Frequent	4.5
Soil lateral spreads		5.0
Soil avalanches		6.5
Soil falls	Moderately frequent	4.0
Rapid soil flows, rock slumps		5.0
Sub-aqueous landslides	Uncommon	5.0
Slow earth flows, rock block slides		5.0
Rock avalanches		6.0

Table 60.1 Types of slope failures, relative frequency and minimum triggering earthquake magnitudes

(a) **Figure 60.6(a)** shows a continuous layer of liquefied soil which daylights in a river bank, leading to post-earthquake instability and collapse of the building.

(b) **Figure 60.6(b)** a liquefied layer at depth, with no potential for lateral instability, leads to post-earthquake settlement, which does not cause serious damage to the building, but may cause damage to connections into the building (e.g. gas mains, water pipes) if they have insufficient flexibility. Ishihara and Yosmine (1992) and Cetin *et al*. (2004) provide guidance on methods to assess reconsolidation settlement.

(c) **Figure 60.6(c)** shallow pockets of sand cause differential settlement and associated damage to the building (but not collapse).

(d) **Figure 60.6(d)** a zone of liquefied soils leads to post-earthquake instability of the embankment (termed 'flow' failure) (because the overall soil shear resistance is less than the 'static' shear stress imposed by the embankment, as conventionally assessed by slope stability analyses. The key difference is that the 'residual' shear strength of the liquefied layer needs to be assessed.) The lateral deformation of the ground will lead to additional forces on the piles, which could lead to structural failure of the piles if they are inadequately designed.

During earthquake shaking, ground deformations can be assessed from Newmark's (1965) sliding block method. This method is useful for obtaining approximate estimates of deformation, especially for embankments/slopes comprising non-liquefiable soils. Lateral spreading deformations of mildly sloping ground can be assessed by several methods, including the empirical approach of Youd *et al*. (2002). It is commonly assumed that liquefied soil has zero shear strength. However, in most circumstances the liquefied soil will maintain 'residual' shear strength because of fast dissipation of excess pore water pressure. Olson and Stark (2002) have provided relationships for the residual shear strength of liquefied soil based on SPT and CPT results. Recently, Srbulov (2011) back analysed 14 case histories of flow type slope failures in liquefied soil. In layered deposits, not all soil layers liquefy. Often the uppermost soil layer (or 'crust') will not liquefy when the water table level is at depth and the soil is not fully saturated. Excess pore water pressure causes ejection of sand and water at the ground surface in forms of sand volcanoes (**Figure 60.7**).

When the top non-liquefied soil layer is thick enough to prevent formation of sand volcanoes then sills (intrusions) are formed within non-liquefied layers. It also means that such layers are subjected to uplift pressure, which decreases their bearing capacity.

Soil liquefaction can cause not only flow type failure of slopes but also loss of bearing capacity of foundations and their excessive settlement and horizontal movement. Shallow foundation can tilt or sink (e.g. Liu and Dobry, 1997). Piled

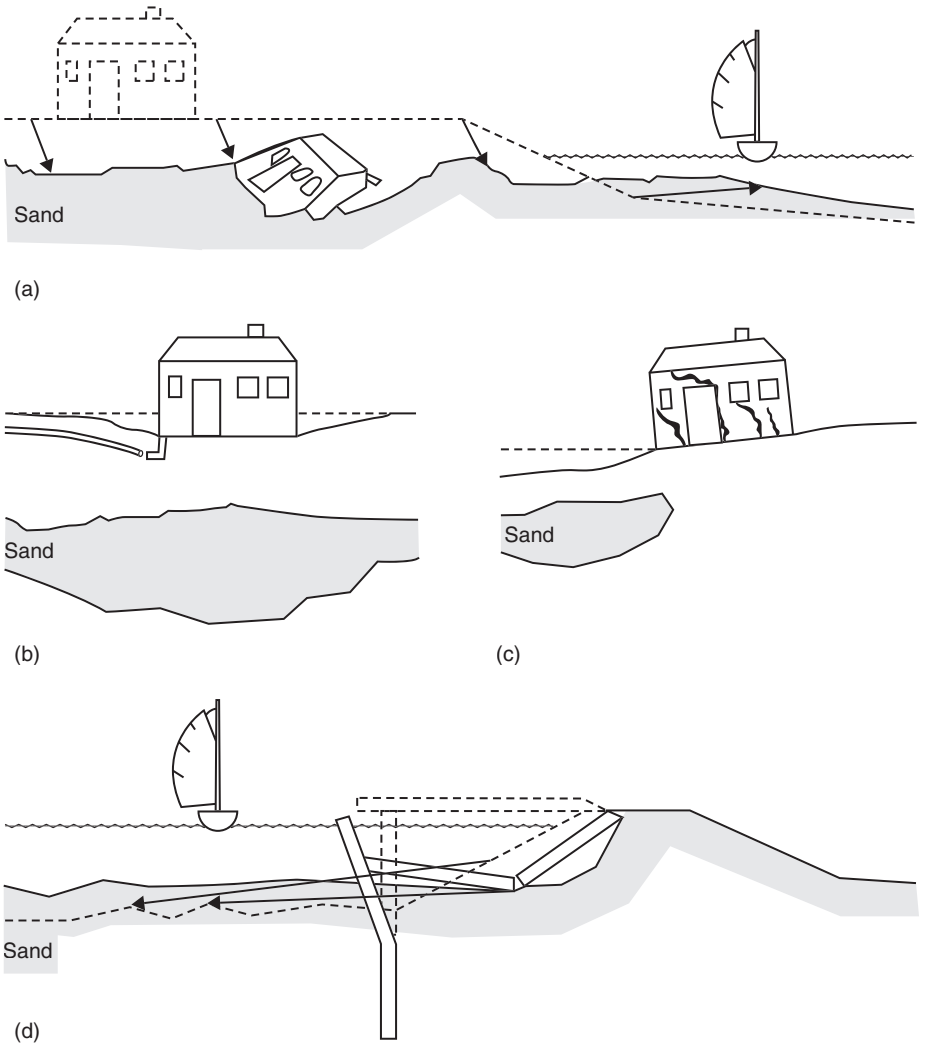


Figure 60.6 Effects of soil liquefaction: (a) flow type slope failure leading to a building collapse, (b) deep soil layer and building excessive settlement with break of a pipe, (c) differential settlement and severe damage of a building, (d) flow type slope failure of an embankment and damage to the deck and supporting piles

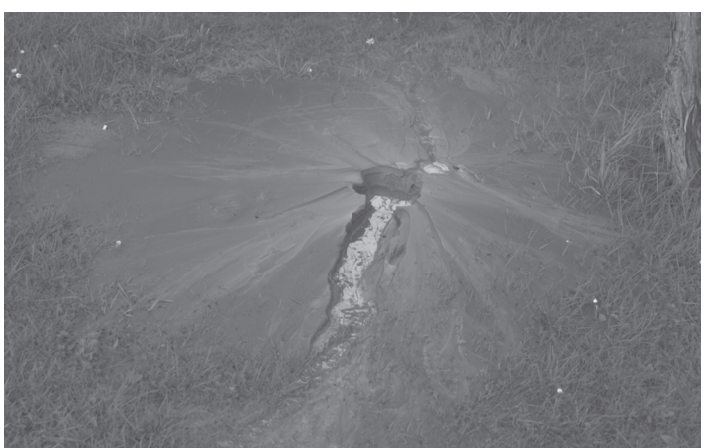


Figure 60.7 An aerial view of sand volcano caused by sand ejection at the surface due to its liquefaction
© Jon Sullivan

foundation can be sheared in case of flow type slope failure (e.g. Hamada, 1992) because the top non-liquefied crust exerts significant lateral force, which equals the passive lateral resistance of the layer.

60.3.5 Shallow foundations

The failure mechanisms for shallow foundation in cyclic conditions are the same as those for static conditions, which are considered in Chapter 53 *Shallow foundations*. Soil bearing capacity, foundation sliding and overturning resistances are usually checked by using static methods with addition of an inertia force from the foundation and structural acceleration. Soil properties in cyclic conditions need to be used with the static methods. Also, the effect of inertial force on soil under the foundation needs to be considered, for example using an equivalent inclined foundation or inclined ground surface. The angle of inclination may correspond to the angle necessary to

rotate the resultant of gravity and inertia forces acting on the soil under the foundation into the vertical. BS EN1998-5:2004 (BSI, 2004) provides an informative formula for assessment of seismic bearing capacity of shallow foundations. The formulae for static bearing capacity that use the coefficient $N\gamma = 2(N_q + 1)\tan\phi$ are not considered safe to use.

Settlement of shallow foundation during earthquakes is caused by soil densification, i.e. decrease in soil porosity due to ground shaking (soil porosity is the ratio between the volume of voids and the total volume of soil). Recently, Srbulov (2011) calculated earthquake-induced settlement of a shallow foundation. Sliding of shallow foundations can be estimated using Newmark's (1965) sliding block method and charts derived from this method for level ground (e.g. Ambraseys and Srbulov, 1995). Permanent rotation of foundation can be assessed using a rotating block method (e.g. Srbulov, 2011).

Seismic codes contain limited recommendations for analyses of shallow foundation vibration. For simplified soil–structure interaction analyses, shallow foundations are represented by equivalent vertical, horizontal and rotational springs K and dampers C as sketched in **Figure 60.8**. Ground motion can be applied at the ends of the springs and dampers or on a structure. Vertical springs and dampers are needed even if only the horizontal ground motion component is considered because of coupled horizontal and rocking mode of vibration, with the vertical component at the edge of the foundations.

The formulae for calculation of values of the dynamic stiffness and damping coefficients are tabulated in relevant textbooks (e.g. Gazetas, 1991; Wolf, 1994). If foundation vibration is considered excessive for the intended use of the structure then foundation vibration isolation can be used. Recently, Srbulov (2010) performed simplified numerical analysis of the response of a rubber bearing isolated building in Japan during a strong earthquake.

60.3.6 Deep foundations

For deep foundation types, piles are most frequently used although vertical shafts and caissons are used occasionally. Deep foundations are subjected to kinematic and inertial interactions.

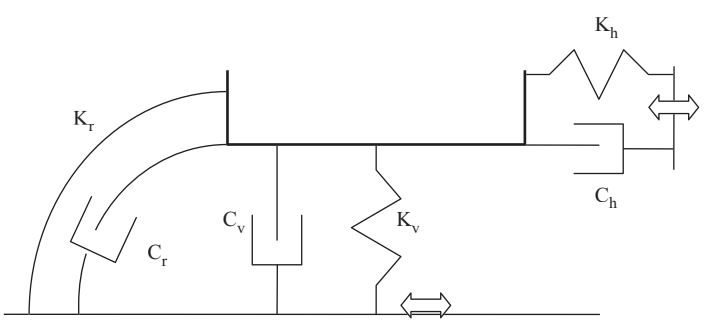


Figure 60.8 Soil and foundation stiffness and damping represented by equivalent springs and dampers

Kinematic interaction is caused by different stiffness of foundation and surrounding soil. A stiffer foundation tends to average ground movement and in this process becomes stressed. The largest effects occur at the interfaces between layers with different stiffness including liquefied layers. Large differential horizontal displacements of different layers may cause significant bending moments in deep foundations and require the use of continuous reinforcement along the whole foundation length. In simplified analyses, the effects of kinematic interactions are determined using horizontal equivalent soil P–y springs (e.g. API RP 2A-WSD, 2007), which ends are subjected to the horizontal displacements that are calculated considering one-dimensional wave propagation in the vertical direction in the free field (**Figure 60.9**). As y values represent differential horizontal displacement between a pile and adjacent soil and not absolute horizontal soil displacement, several iterations are necessary to calculate the values of y from applied previous P forces on a pile based on previous estimates of y values. Kinematic pile interaction has positive effect on decreasing the peak horizontal acceleration that is transferred to the superstructure above the piles and increasing the vibration period of structural vibration, which in turn results in decreased input acceleration, in general.

Inertial interaction occurs because of structural mass, which together with structural stiffness causes a different vibration frequency (period) for a structure with respect to the adjacent ground. Different structural periods cause different periods of pile vibration, which tend to oscillate asynchronously with the ground. A superposition of incoming ground waves to a pile and outgoing ground waves from a pile vibration can cause superposition of wave amplitudes and new wave pattern as sketched in **Figure 60.10**.

Seismic codes contain limited recommendations for deep foundations during earthquakes. For simplified soil–structure interaction analyses, the formulae for calculation of values of the dynamic stiffness and damping coefficients of individual

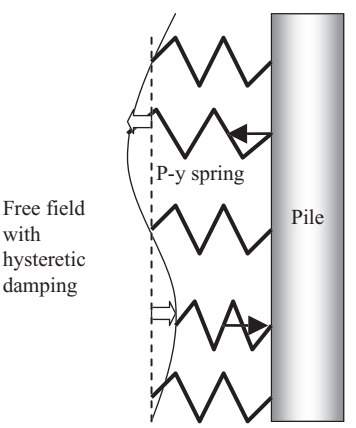


Figure 60.9 Sketch of the horizontal P-y springs and their connections with the free field boundary and a pile

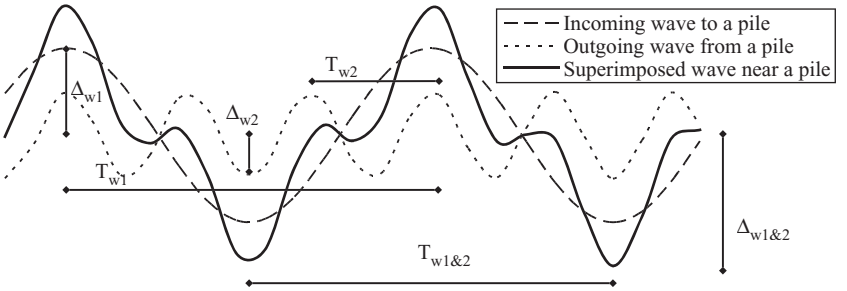


Figure 60.10 Sketch of superposition of wave amplitudes near a pile due to different vibration properties of the adjacent ground and piles

piles are tabulated in relevant textbooks (e.g. Gazetas, 1991) for different variations of soil stiffness modulus with depth.

Recently, Srbulov (2011) used simple engineering beam theory for analyses of inertial and kinematic interaction for 11 case histories of piled foundations subjected to earthquakes.

If large lateral ground displacement occurs in the post-seismic stage then the effect of lateral forces imposed on piles can be analysed using the methods described in Chapter 57 *Global ground movements and effects on piles*.

60.3.7 Risk reduction measures

Risk reduction measures are dependent on the cause of risk. When foundation response is a source of hazard, in competent ground but during strong earthquakes, then the increase in number, depth or diameter of piles can be helpful. The use of raking piles to support horizontal load is still debatable because some inclined piles behaved very well and the others failed during earthquakes. If brittle failure of concrete piles represents risk then the use of reinforcement along full pile length and addition of a steel sleeve provides the desirable ductility of the foundation. On the other hand, concrete infill prevents buckling of steel pipe piles during strong ground motion. When behaviour of weak ground is a source of hazard then the following measures can be considered:

- Slope stabilisation methods are described in Chapter 72 *Slope stabilisation methods.*
- When the amount of fines does not exceed about 20%, stone columns are efficiently used for increase of ground density, permeability and shear strength, which all decrease the potential of soil liquefaction.
- Ground drains and trenches can be used for speeding up the dissipation of excess pore water pressure. Ground drains may become clogged in time by siltation, when groundwater level varies, and by bacteria and algae growth.
- Soil mixing with binders can be used for the increase in stiffness of soil and decrease of amplitudes of its vibration when vibration and lateral deformations caused by stone column installation are not acceptable and when the spacing is limited, such as among existing piles and anchors (e.g. Kramer, 1996; Port and Harbour Research Institute, 1997). More details on ground improvement are provided in Chapters 59 *Design principles for ground improvement* and 84 *Ground improvement*.

60.4 Offshore foundation design

60.4.1 Typical condition and wave loading

Wave heights are determined from meteorological observations at sites of interest. The periods of sea waves range from about 2 to 27 s (e.g. Poulos, 1988b) and therefore the inertia forces arising from wave load are not significant but the effects of cyclic loads are significant and the equivalent static analyses, which are usually performed, need to consider soil properties in cyclic condition. An exception is pile driving, which can induce peak pile accelerations greater than 10 g although during a period of a millisecond. Detailed description of pile drivability analyses is provided by Dean (2009). The number of hammer blow counts from drivability analyses is used to calculate fatigue damage to pile steel and remaining useful lives in years based on so-called S-N curves (e.g. Offshore Technology Report, 2001; API RP 2A-WSD, 2007). The effect of cyclic wave loading on fatigue damage to pile steel is also considered using S–N curves. Other dynamic loads offshore involve debris flow, fast spreads, underwater avalanches and seaquakes which can be analysed using the methods applied to onshore events (e.g. Srbulov, 2008).

For most offshore foundations, the allowable deformations are relatively large, and usually the critical design criterion is an acceptable factor of safety against collapse under a severe storm event.

60.4.2 Soil behaviour

Soil behaviour in cyclic condition is described in Chapter 24 *Dynamic and seismic loading of soils*. Offshore marine clays can have very soft consistency or be underconsolidated and prone to cyclic degradation. Specific ground types encountered offshore are calcareous sand and corals. Calcareous sand has fragile grains that are crushed under impact load (e.g. Kolk, 2000) and can exhibit very small shear resistance, like very soft clay. Coral structure may be collapsible because of the presence of cemented skeletons and numerous voids between the parts of skeletons (e.g. Touma and Sadiq, 1999). Global and local scour could reach significant depths offshore in loose top sand. Local scour causes gapping, which can significantly decrease the lateral pile resistance and affect vibration period

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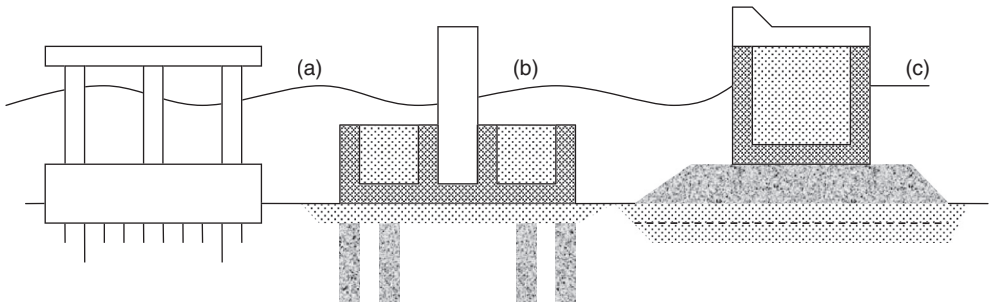


Figure 60.11 Sketches of (a) foundation with skirts and dowels under a gravity platform, (b) foundation of a pylon of a wind turbine with stone columns for ground improvement, (c) caisson breakwater on rubble mound over reinforced sand layer

(frequency). For this reason scour protection is often used. Global scour decreases vertical effective stress around piles. Even submerged, soil offshore may be partially saturated due to presence of natural gas.

60.4.3 Shallow foundations

Typical foundation types are shown schematically in **Figure 60.11**.

A number of recommendations exist for design of offshore shallow foundations such as: API RP 2A-WSD (2007), DNV-OS-J101 (2007), DNV classification notes 30.4 (1992), De Groot *et al*. (1996), BS 6349-7:2010 and others. The issues concerning shallow foundations onshore, which are described in Chapter 53 *Shallow foundations*, are also relevant for offshore foundations.

As noted earlier, the soil shear strength and stiffness used for design must allow for degradation due to cyclic loading (**Figure 60.12**). Another common feature of offshore design are the high horizontal loads, which need to be resisted (due to large waves during storm events). Hence, the correction for inclined loads and load eccentricity become significant (refer to Chapters 21 *Bearing capacity theory* and 53 *Shallow foundations*). Modern developments in bearing capacity theory, using V-H-M failure envelopes have been stimulated by the inadequacies of conventional theory (with its plethora of correction factors) for offshore design requirements. Offshore foundations are often very large and issues such as soil layering, stress level (for coarse-grained soil) and strength variations with depth (for fine-grained soil) can be important.

Besides ground-bearing capacity at the ultimate limit state, foundation settlement in the serviceability limit state needs to be considered. A large number of cycles generated during storms induce excess pore water pressures in the ground under shallow foundation. Dissipation of such pressure causes additional settlement in loose to medium dense sandy soils and soft clay/silts. The additional settlement may affect connecting tubes between an offshore structure and fluid exporting pipelines if such settlement has not been considered in design. The design considerations frequently lead to recommendations for ground improvement, which are sketched in **Figure 60.11(b)** and **(c)**.

60.4.4 Deep foundations

Typical deep foundations offshore are shown in **Figure 60.13**.

Offshore foundations typically need to resist larger loads (especially horizontal), and relatively large cyclic loads, than onshore foundations and the methods of construction (e.g. installation of caisson using suction) will usually be quite different. However, in other respects the general design principles for offshore foundations are similar to deep foundations onshore. Modern mono-piles that are used for offshore wind farms are several metres in diameter and have mass of several hundred tons. As mentioned before for the shallow foundations offshore, it is important to consider the effect of cyclic loads on soil properties but not inertia forces for the periods of vibrations of a few seconds. Dean (2009) describes in detail the foundations, methods for their design and installation, applicable codes and available literature. A number of recommendations exist for design of offshore deep foundations such as: API RP 2A-WSD (2007), DNV-OS-J101 (2007), SNAME TR-5A (2002) and others.

Cyclic wave loading during storms causes degradation of ground shear strength and stiffness, which in turn cause decrease of pile shaft friction and increase of lateral deformation of piles. The degradation of shaft resistance under axial loading is a function of the cyclic displacement and the number of load cycles. A cyclic stability diagram is an important concept (Poulos, 1988a), and three main regions need to be identified: a cyclically stable region; a cyclically metastable region (in which cyclic axial loading causes a reduction of shaft capacity, but the pile does not fail within the specified number of cycles); and a cyclically unstable region, in which cyclic loading leads to failure within the specified number of axial load cycles. Randolph (1983) has provided guidance on the boundary between stable and metastable regions. Reduced soil shear strength and the use of P–y curves in cyclic conditions are often used in order to account for lateral load effect, although several important phenomena are still the subject of ongoing research.

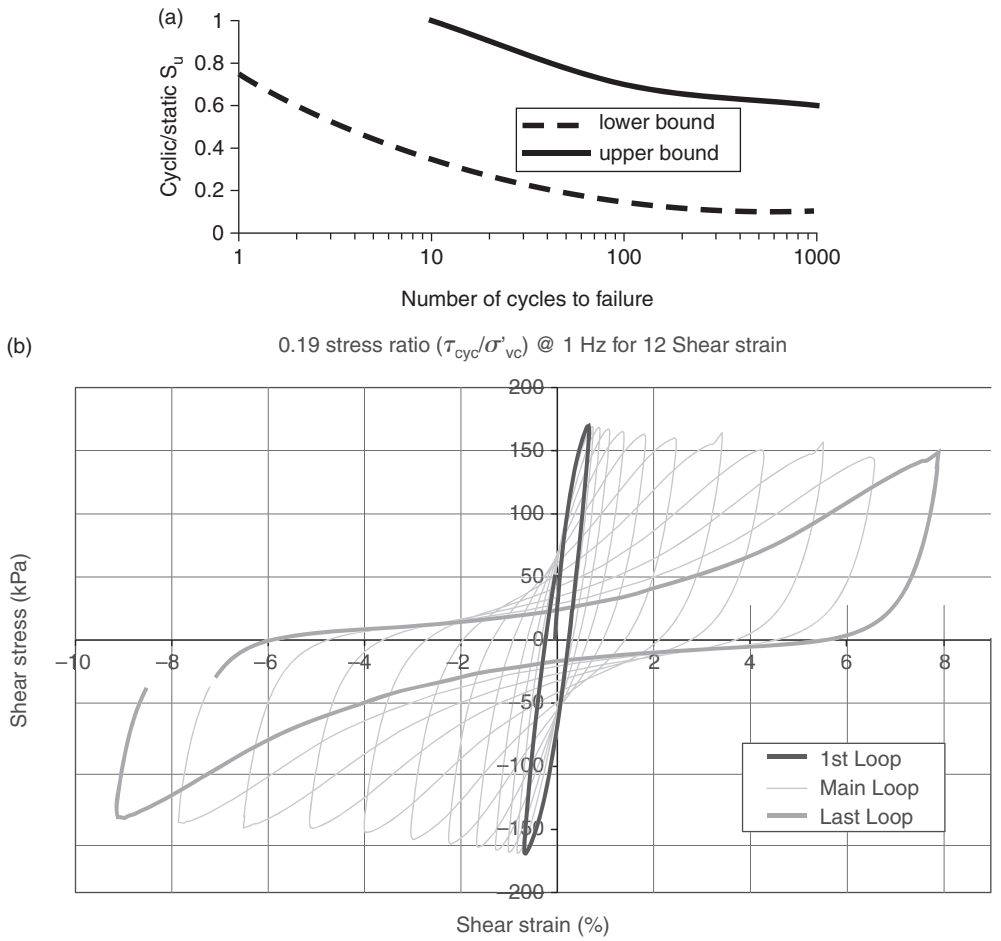


Figure 60.12 Effects of number of cycles on (a) ratio of cyclic to static undrained shear strength Su of clays (from data by Lee and Focht, 1976), (b) degradation of stiffness (ratio between shear stress and strain) of a non-plastic silt specimen in a cyclic simple shear test

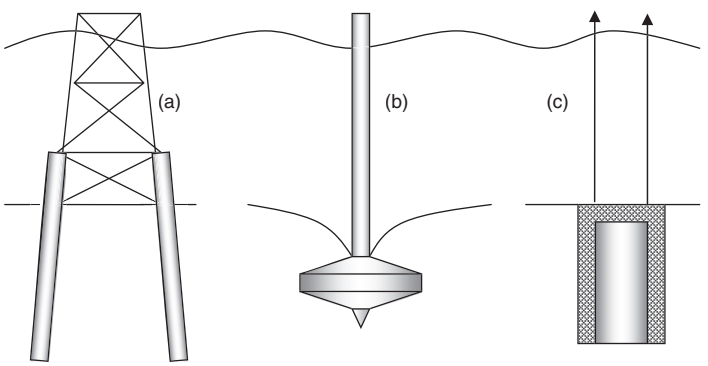


Figure 60.13 Sketches of (a) piled foundation of a jacket,(b) spud-can foundation of a jack-up (mobile platform), (c) caisson foundation of a tension leg or a deep water anchor

60.5 Machine foundations

Machine foundations are typically subjected to a large number of loading cycles with relatively small amplitudes. The operational conditions given below are an example of the requirements specified by the manufacturer of a particular machine:

- large compressor (MAN) foundation velocity < 2.8 mm/s in operational condition and < 6 mm/s in accidental case between frequencies from 25 to 190Hz;
- gas turbine (EGT) foundation velocity < 2 mm/s and where a peak to peak amplitude of any part of the foundation is less than 50 μm at the operating frequency of 250 Hz.

The specified frequency (i.e. its range) must be avoided in order to avoid resonance effects between the machinery and its foundation, which will damage the machine unless it has been switched off. The vibration resonance is indicated as the increase in vibration amplitude ratio at resonant frequency in **Figure 60.14** for harmonic vibration (e.g. Clough and Penzien, 1993), where $\beta_t = f_d/f_o$ is the tuning ratio, f_d is the frequency of machine vibration, f_o is the frequency of foundation free vibration, ξ is the damping ratio, a_o is the acceleration amplitude of foundation, and a_i is the acceleration amplitude of machine vibration.

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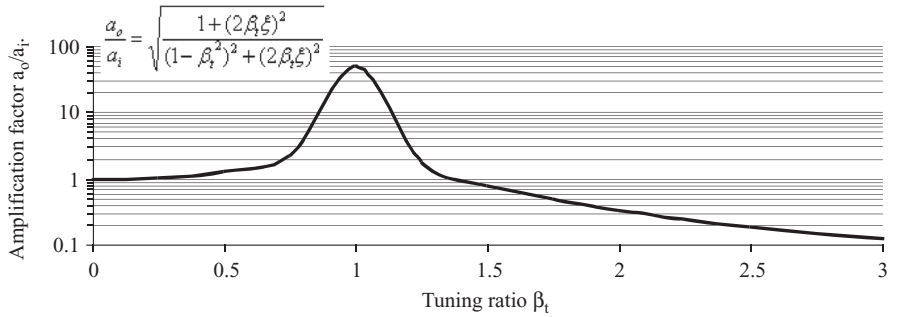


Figure 60.14 Amplification factor for a harmonic vibration with 1% damping ratio

Vibration type	Slender and potentially sensitive masonry walls	Propped or tied walls or mass gravity walls
Intermittent vibration	10@the toe 40@the crest	50% to 100% greater than for slender and potentially sensitive masonry walls
Continuous vibration	1.5 to 2.5 times smaller than the intermittent vibration limits	

Table 60.2 Threshold peak particle velocities in mm/s for minor or cosmetic damage according to BS 5228-2:2009

Foundations can be subjected to vibration with the origin located at a distance. For example, BS 5228-2:2009 (BSI, 2009) specifies the limits of peak particle velocities to which retaining walls are allowed to be subjected for minor or cosmetic damage (**Table 60.2**). The same limits are applicable to foundations.

Machine-induced vibrations usually only cause very small strains in the ground and therefore elastic soil properties are used for dynamic analyses. Also, soil damping ratio is small, in the order of 1%. The energy dissipation is mainly achieved by radiation damping.

Vertical vibration of machine foundations is usually considered separately from the horizontal and rocking vibrations, which are coupled. The basic equation of foundation motion arises from Newton's second law.

$$m \cdot a + c \cdot v + k \cdot d = F \cdot \sin(2 \cdot \pi \cdot f \cdot t). \quad (60.3)$$

In the above equation, m is foundation and machine mass, a is acceleration of foundation, c is damping coefficient (which is frequency dependent value), $c = \xi k/(\pi f)$, ξ is damping ratio (about 0.01), f is vibration frequency, v is velocity of foundation, k is stiffness coefficient, d is displacement of foundation, F is maximum amplitude of dynamic unbalanced force, and t is time. The formulae for calculation of values of the dynamic stiffness k and damping coefficients c are tabulated in relevant textbooks (e.g. Gazetas, 1991; Wolf, 1994). The formulae are provided for homogeneous soil and a soil layer over bedrock and should not be used for layered soil. If foundation vibration is considered excessive according to the requirements set by the manufacturer then foundation vibration isolation can be used. Recently, Srbulov (2010) performed simplified numerical analysis of the response of a rubber bearings isolated foundation block under a compressor in Holland.

The horizontal motion is coupled with the rotational motion, which equation is:

$$I_a \cdot \theta_a + c_\theta \cdot \theta_v + k_\theta \cdot \theta - e \cdot (c \cdot v + k \cdot d) = M \cdot \sin(2 \cdot \pi \cdot f \cdot t). \quad (60.4)$$

In the above equation, I is foundation and machine mass moment of inertia with respect to the centre of the masses, θ_a is rotational acceleration, c_θ is rotation damping coefficient, θ_v is rotational velocity, k_θ is rotational stiffness coefficient, θ is rotation of foundation, e is the eccentricity of the base of foundation with respect to the centre of mass of foundation and machine, and M is the maximum amplitude of rotational moment from unbalanced force with respect to the centre of mass of foundation and machine. The formulae for calculation of values of the rotational stiffness k_θ and damping coefficients c_θ are tabulated in relevant textbooks (e.g. Gazetas, 1991; Wolf, 1994).

Simple guidance exists for determination of minimum foundation mass for vibration control (e.g. Anyaegbunam, 2011). Older publications contain examples of foundation vibration analyses by hand calculations (e.g. Irish and Walker, 1969), which may be used for rough check of results of modern numerical analyses by computers. CP 2012-1 (1974) exists for foundations for reciprocating machinery, which generates biharmonic loads (e.g. steam engines, internal combustion engines, piston-type compressors and pumps), with normal rotational frequency range from 5 to 25 Hz and is relevant for machines placed on a rigid block. DIN 4024-1 (1988) & 2 (1991) contain useful recommendations. A frequently cited book is Richart *et al*. (1970). Not only theoretical but also experimental methods have been used for analyses of foundation vibration.

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It is recommended this chapter is read in conjunction with

- Chapter 24 *Dynamic and seismic loading of soils*
- Chapter 45 *Geophysical exploration and remote sensing*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Section 6: Design of retaining structures

Section editor: **Asim Gaba**

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Chapter 61

Introduction to Section 6

Asim Gaba Arup Geotechnics, London, UK

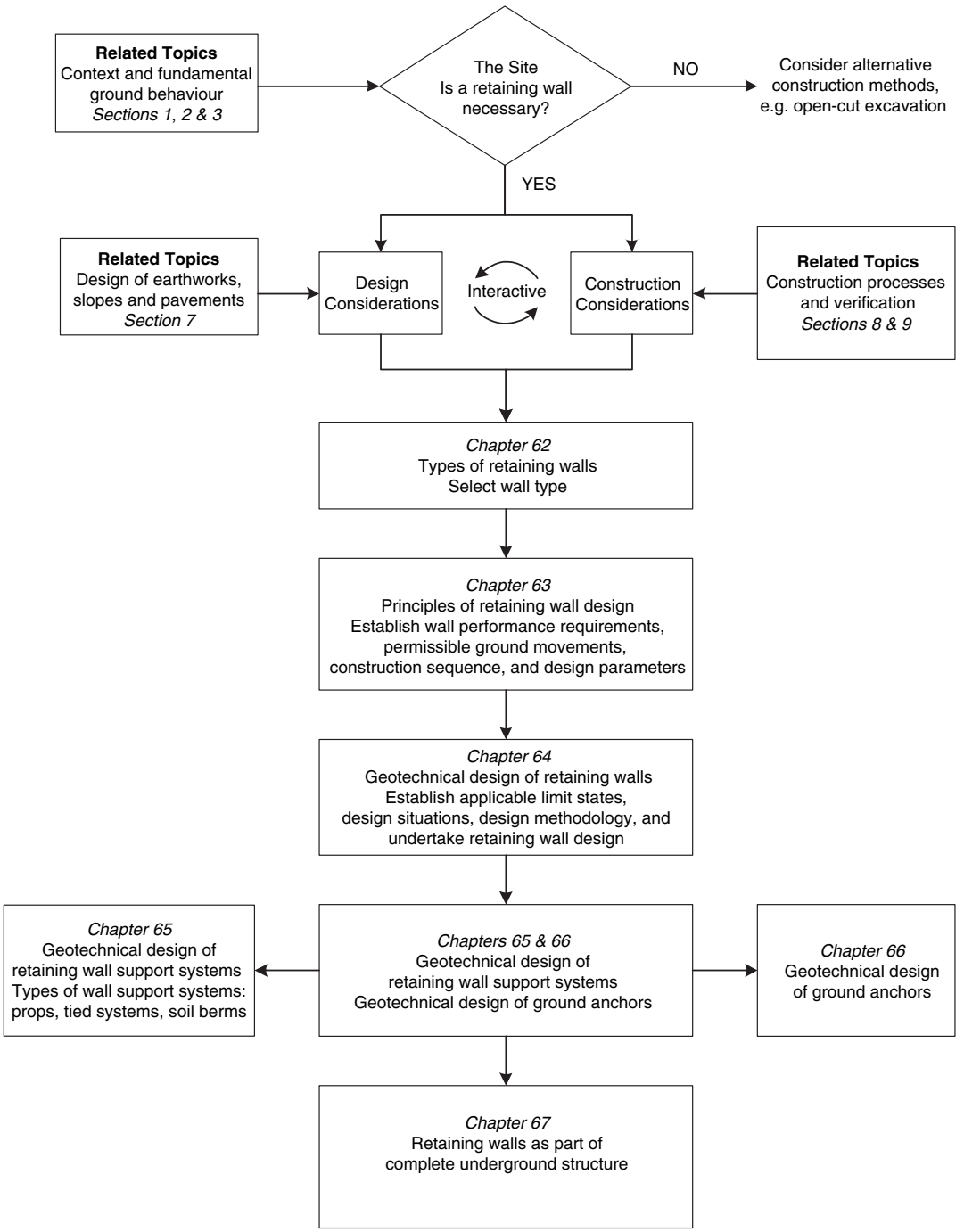


Figure 61.1 Relationships between the chapters in Section 6

Retaining walls are typically used to retain the ground at a steeper angle than that which can be sustained by the ground acting unsupported on its own. Common applications vary from modest terracing of sloping ground to perimeter walls for multi-storey basements and underground structures.

Retaining walls are usually divided into two broad categories: 'gravity' retaining walls and 'embedded' retaining walls and this distinction is also made in this Section. However, there is a third category – 'hybrid' walls – these have their own particular characteristics and their design may demand an approach that requires consideration and assessment of both gravity and embedded wall behaviour. An example of a hybrid wall would be a gravity wall supported partially by piles embedded into the ground beneath the gravity structure.

In common with other sections of this manual, this section is intended to provide guidance to practising engineers on the design of retaining walls. Given the considerable breadth of the subject and the vast range of ground conditions which a retaining wall designer may encounter, this section cannot, within the available space, be completely comprehensive. The intent is therefore to provide outline guidance to designers on:

- considerations of wall performance criteria, wall selection, construction methods and associated ground movements;
- fundamental principles of good design practice in relation to the selection, design and construction of retaining walls and their support systems;
- key mechanisms of ground and ground–structure interaction behaviour, which need to be considered;
- commonly used design methods for gravity and embedded retaining walls and their support systems;
- additional references for detailed study.

Figure 61.1 shows the relationships between the chapters in Section 6.

The chapters are arranged in a sequence to guide the reader in following a systematic approach in considering and dealing with the key issues that relate to the design of retaining walls, recognising that retaining walls may form part of an overall structure. It is important not to lose sight of the fact that the client's priority is the entire structure and not just the retaining walls.

Chapter 62 *Types of retaining walls* identifies different types of retaining wall along with the particular characteristics of each. The advantages and limitations of each wall type are compared and guidance is provided on the selection of the most appropriate wall type to satisfy the particular site and project requirements.

Chapter 63 *Principles of retaining wall design* provides guidance on the determination of key wall performance and design criteria, setting permissible limits for the associated ground movements and identifying the appropriate construction sequence for particular applications. The chapter also provides guidance on the assessment of drained and undrained ground behaviour and the selection of parameters appropriate for temporary and permanent works design.

Chapter 64 *Geotechnical design of retaining walls* offers good practice guidance on the geotechnical design of retaining walls. Using the approach outlined in this chapter, the reader will be able to size retaining walls to ensure adequate stability and calculate load effects to be used in the structural design of walls.

Chapters 65 *Geotechnical design of retaining wall support systems* and 66 *Geotechnical design of ground anchors* give the reader an overview of a wide range of different methods of providing lateral support to both gravity and embedded retaining walls for temporary and permanent situations. Chapter 65 provides guidance on the appropriate selection and design of propping systems and berms for the lateral support of a wall. Chapter 66 is a complementary and comprehensive chapter that deals with the design of ground anchors for the support of retaining walls and should be read in conjunction with Chapters 74 *Design of soil nails*, 88 *Soil nailing construction* and 89 *Ground anchors construction* of this manual.

Chapter 67 *Retaining walls as part of complete underground structure* deals with the situation where the retaining walls are part of an overall structure. It provides holistic considerations of the ground–structure interaction, retaining wall water tightness requirements and the importance of communications between the retaining wall designer and other design and construction professionals.

Following this theme, it is important to recognise that a retaining wall designer cannot function in isolation of others. To overcome the potential fragmentation of design and construction that can accompany the process of retaining wall design, it is important that the designer considers all parts of the construction sequence and, in doing so, communicates effectively with other complementary design and construction professionals. The client, designer and constructor and, where appropriate, the architect and the quantity surveyor, should be involved as early as possible to:

- optimise the temporary and permanent use of a retaining structure (such as adopting one wall instead of two to serve both the temporary and permanent requirements), which is also compatible with long-term maintenance requirements;
- establish appropriate design and performance criteria for the retaining structure, such as acceptable limits for wall deflection and associated ground movement;
- consider an appropriate wall type;
- consider an appropriate method and sequence of construction to ensure buildability.

Initial ideas should be reviewed and alternatives explored before agreeing the preferred solution. It is important to involve individuals with appropriate expertise and experience at all stages of the project to maintain adequate continuity and communication between the personnel involved in data collection, design and construction.

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Chapter 62

Types of retaining walls

Sara Anderson Arup, London, UK

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The term retaining wall applies to structures that retain the ground at a steeper angle than the ground can sustain acting on its own. Common applications vary in scope from modest terracing of sloping landscaping to perimeter walls for multi-storey basements and underground spaces. Several different forms of retaining wall exist with different performance characteristics in terms of water retention, strength and stiffness, as well as constructability characteristics such as land-take and requirements for specialist plant and equipment. The different characteristics of retaining walls must be fully appreciated as part of the process of selecting a wall for a particular application.

This introductory chapter to Section 6 identifies the different forms of retaining wall along with their particular characteristics.

62.1 Introduction

Retaining walls exist in many shapes and forms but can be considered in terms of three different groups:

- gravity walls;
- embedded walls;
- hybrid walls.

These are discussed in detail below.

62.2 Gravity walls

A gravity wall gains resistance to sliding and overturning through the weight of the wall and friction on its underside. Gravity walls are generally constructed on a flat surface prior to backfilling behind the wall to create the height difference that the wall retains. As such they are commonly used where the ground profile is being raised locally, e.g. to retain a highway or railway embankment.

Gravity walls can also be used to support excavations below ground level. For these scenarios the excavation is typically an open-cut and backfill is provided behind the wall. This requires additional excavation and backfilling as well as temporary land-take during construction.

Drainage provision in the form of interceptor drains behind the wall is a common feature of gravity walls in order to reduce the water-pressure load acting on the wall.

Key references for gravity retaining walls include:

- CIRIA C516 (Chapman *et al.*, 2000) – Section 2 and Section 4 mostly;
- BS8002 (BSI, 1994) – Section 4;
- Eurocode EC7 (BSI, 2004).

62.2.1 Modular and non-modular walls

Modular walls may be constructed using a wide variety of techniques and materials. They share the common feature that their construction involves the erection of a number of mainly similar modular elements.

Examples of modular walls include:

- pre-cast reinforced-concrete stem walls;
- masonry walls;
- dry stack masonry walls (e.g. porcupine walls);
- crib walls;
- gabion walls.

For modular walls, the modules are constructed off-site and can be assembled on site with a minimum of specialist site operations. In contrast, non-modular walls are predominantly constructed on site, e.g. *in situ* reinforced-concrete walls. As such they require a greater level of site activity and more significant specialist activities such as shuttering, steel fixing and *in situ* concreting.

62.2.2 Reinforced-concrete stem walls

Reinforced-concrete stem walls are constructed from *in situ* or pre-cast reinforced concrete. The walls are L-shaped or inverted T-shaped walls where the concrete stem cantilevers from the wall base to support the retained material behind the wall. A shear key may be used to increase the shear resistance; see **Figure 62.1**. The walls are not reliable as water-retaining structures and, therefore, are usually designed and constructed with suitable drainage behind the wall to limit the build-up of water at the back of the wall.

Pre-cast walls are typically from 1 m to 3 m in height.

62.2.3 Masonry walls

Masonry walls are made with brick, block, natural or manufactured stone, conventionally bedded with mortar and founded on mass or reinforced-concrete foundations; see **Figure 62.2**.

The walls should be provided with a coping layer to avoid water saturation and possible frost damage. Details for the use of copings, drainage, damp-proof courses and waterproofing to prevent saturation of the wall and improve durability are given in BS 5628 Part 3 (BSI, 2001). The walls should be constructed

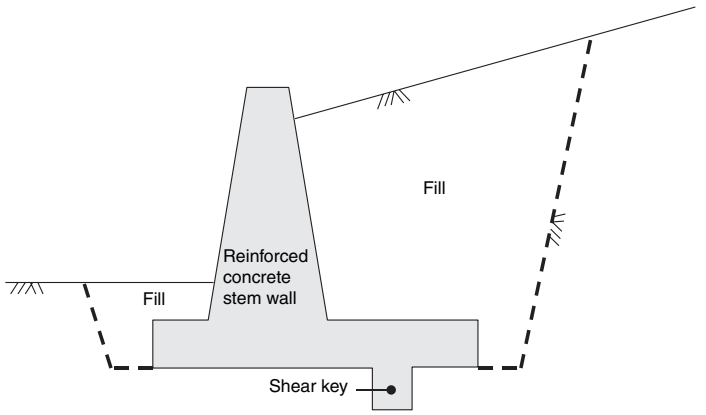


Figure 62.1 T-shaped reinforced-concrete stem wall with shear key

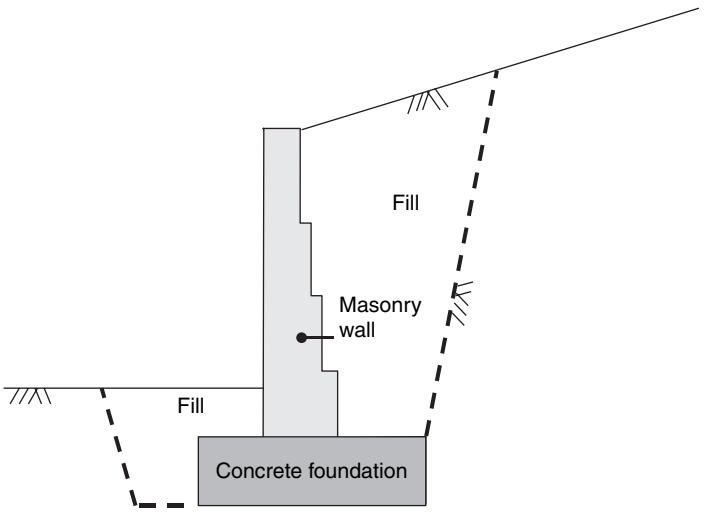


Figure 62.2 Typical masonry wall

in panels, up to 10 to 15 m in length, with movement joints at the ends.

62.2.4 Dry-stack masonry walls

Dry-stack masonry walls consist mainly of pre-cast concrete special blocks and occasionally bricks that are designed to interlock with each other and produce a solid wall face; see **Figure 62.3**. A dry-stack wall relies upon gravity to support the retained material and will have an interlock shear resistance between each layer of blocks. The presence of the interlock assists with the accurate placing of successive layers of blocks.

The wall foundations may be mass, reinforced or possibly pre-cast concrete with interlocks compatible with the wall units.

Most of the walls are relatively free draining, but some may require granular backfill material and a drain to avoid the build-up of water behind the wall.

The walls can be constructed vertically but are typically inclined or battered to provide better stability for greater

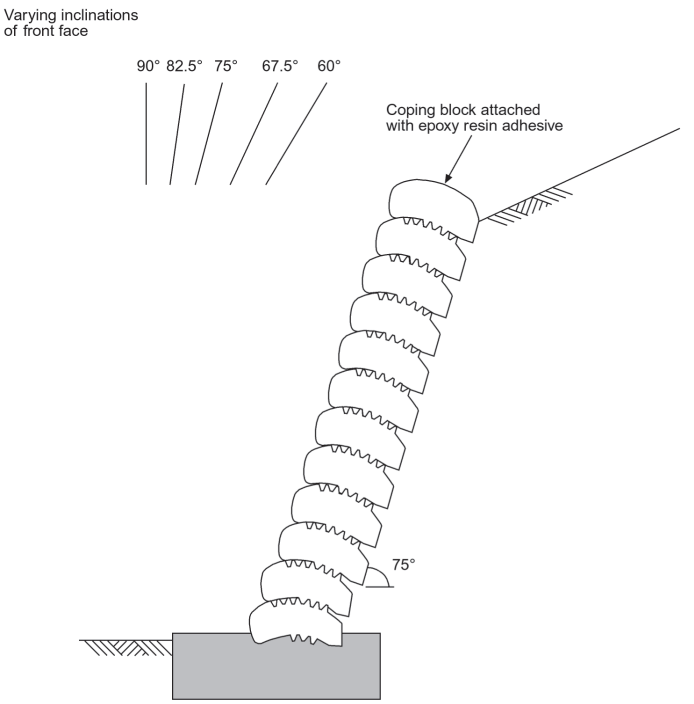


Figure 62.3 Porcupine wall: typical single-width dry-stack masonry wall
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heights. The coping blocks at the top of the wall, often attached to the rest of the wall with adhesive, are to prevent damage to the wall through vandalism.

62.2.5 Crib walls

Crib walls comprise a grillage of header and stretcher units placed on firm foundations, usually of mass or reinforced concrete. The header and stretcher elements may be made of reinforced concrete or timber and are designed to be interlocking, to give the wall continuity. Walls can be made with single or multiple rows of cribs at the base with single rows used for low walls and multiple rows for higher walls. See **Figure 62.4**.

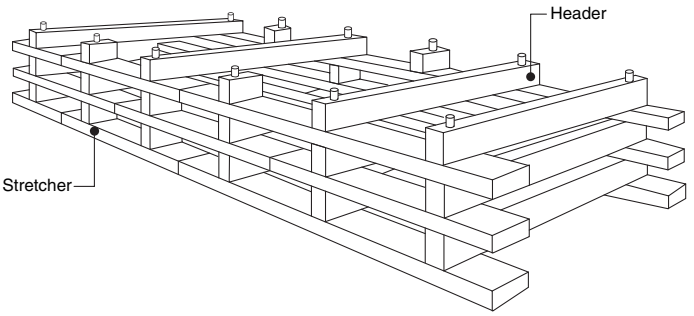
The spaces between the crib grillages are filled with a free-draining, non-aggressive coarse granular material. The spaces can also be filled with lean-mix concrete making it more akin to a masonry wall.

Even without interceptor drains, the open structure of the granular fill can allow water to flow through the wall. Interceptor drains behind the wall will be required with a lean-mix concrete fill to prevent the build-up of water behind the wall.

Crib walls are often laid to a batter, an angle corresponding to 1 horizontal to 12 vertical is typical; however, for shorter walls, generally less than 2 m, the walls are sometimes built with a vertical face provided their width exceeds their height.

62.2.6 Gabion walls

Gabion walls are free-draining walls constructed by filling rectangular mesh cages with broken stone. The cages are made



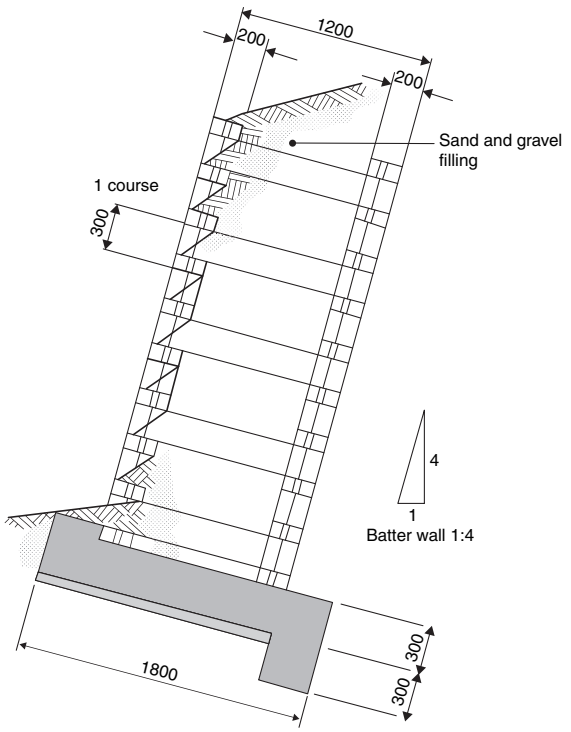


Figure 62.4 Crib wall
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from sheet mesh strips or woven wires. Differing degrees of corrosion resistance of the steel mesh can be provided by galvanising (zinc or zinc–aluminium coating) or coating with a bonded plastic, thermoplastic or epoxy resin polymer coating. They are placed side by side and laced together in a stepped course to form a gravity structure; see **Figure 62.5**. The walls are typically constructed with either a front stepped face or a back stepped face to incline the wall by at least 6°–8° from the vertical toward the retained material.

The walls are typically constructed in 0.5 m to 1.0 m heights, corresponding to the standard size of gabion-basket units. Two forms of cage are common, basket and mattress. Mattress cages tend to be long in relation to the height and are often used as a lower level to a gabion wall.

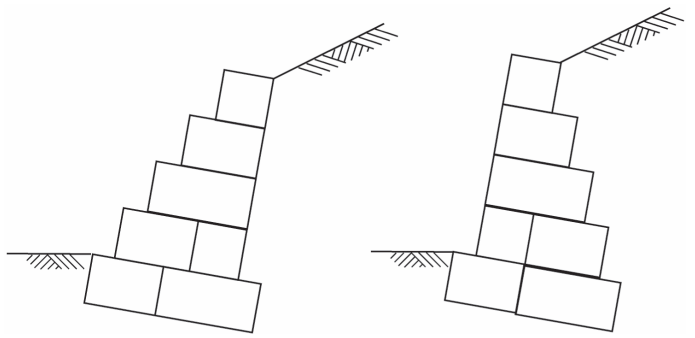


Figure 62.5 Gabion wall
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The flexibility of gabion walls makes them more suitable where the sub-grade soil is poor, as the walls can accommodate larger total and differential settlements than other wall types.

The permeability of the walls also makes them well suited to saturated conditions such as near to rivers. In such conditions, the grading of the fill behind the wall and in the wall needs to be carefully chosen so that the fines in the fill are not washed out by flowing water.

A variation of the gabion wall is a bastion. Bastion walls are collapsible wire or geotextile mesh multicellular structure systems, which are designed for rapid erection from a flat-pack delivery. When assembled, the bastion forms a series of individual cells, which are lined with a non-woven geotextile. The cells are filled with granular material, e.g. sand, ballast, earth, stone or concrete. They are most often used for emergency works such as flood protection.

To be used for permanent works, measures need to be incorporated to prevent the loss of fill material through the mesh and the use of a facing to protect any geotextile mesh against ultraviolet degradation.

62.2.7 Reinforced soil

Reinforced soil can be considered as a gravity wall. In contrast to the gravity walls described above, the gravity structure is provided by reinforcing the ground itself rather than providing a separate structure; see **Figure 62.6**.

Reinforced soil walls are described in Chapters 63 *Principles of retaining wall design* and 73 *Design of soil reinforced slopes and structures.*

62.3 Embedded walls

An embedded retaining wall penetrates into the ground at its base to gain lateral support from the passive resistance of the ground in front of the lower part of the wall. This resistance may be used as a sole resistance to provide stability for the wall or may act in conjunction with anchorages, props or other sources of lateral support for stability; see **Figure 62.7**.

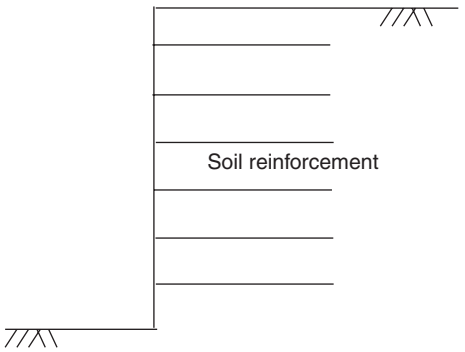


Figure 62.6 Reinforced soil wall

Embedded retaining walls are often used in conjunction with lateral support. Methods of lateral support are described in Chapter 65 *Geotechnical design of retaining wall support systems*.

In contrast to gravity walls, embedded walls require specialist plant and operations on site. Embedded walls are often adopted in preference to gravity walls when:

- deep excavations are required;
- temporary land-take is not available for a temporary open-cut excavation;
- buildings or other structures are in close proximity to the excavation and need to be supported or protected;
- high groundwater conditions require excessive dewatering for a temporary open-cut excavation.

Key references for embedded retaining walls include:

- CIRIA C580 (Gaba *et al*., 2003) – Section 3.2 and Appendix D mostly;
- BS8002 (BSI, 1994) – Section 4;
- *Piling Handbook*, 8th edition (ArcelorMittal, 2008);
- *Steel Intensive Basements*, SCI Publication P275 (Yandzio and Biddle, 2001).

62.3.1 Sheet pile walls

The key references for sheet pile walls are the *Piling Handbook* (ArcelorMittal, 2008) and *Steel Intensive Basements* (Yandzio and Biddle, 2001).

Steel sheet pile walls are constructed by driving, vibrating or pressing steel sheets into the ground without any material being removed. They are traditionally installed by means of driving methods using heavy hammers, which can result in significant noise and vibration that are often unacceptable in urban locations.

Sheet pile walls can be installed in the majority of soils. However, they can be unsuited to ground conditions with hard inclusions such as boulders or man-made obstructions, as these can lead to refusal of the pile driving or pushing. Pre-boring for the sheet piles or jetting along the toe of the sheet pile during installation may improve the penetration of sheet piles; however, these methods are often associated with additional settlement.

There are a number of standard shape profiles for sheet piles as shown in **Figure 62.8**. These can be combined together or with circular or beam steel sections to create a number of sheet pile combinations with improved strength and stiffness performance. A selection of these combinations is shown in **Figure 62.9**.

If seepage through the sheet pile interlock is controlled, a sheet pile wall can be used for water-retaining applications. Sealing systems include non-swelling sealants, hydrophilic (water-swelling) sealants, combination systems and welding of the interlocks. Details of typical construction processes are provided in Section 8.

Advantages of sheet pile walls include:

- provides an economic wall with a predictable surface finish;
- no arisings;
- suitable as water-retaining walls;
- can be used as both a temporary and permanent wall.

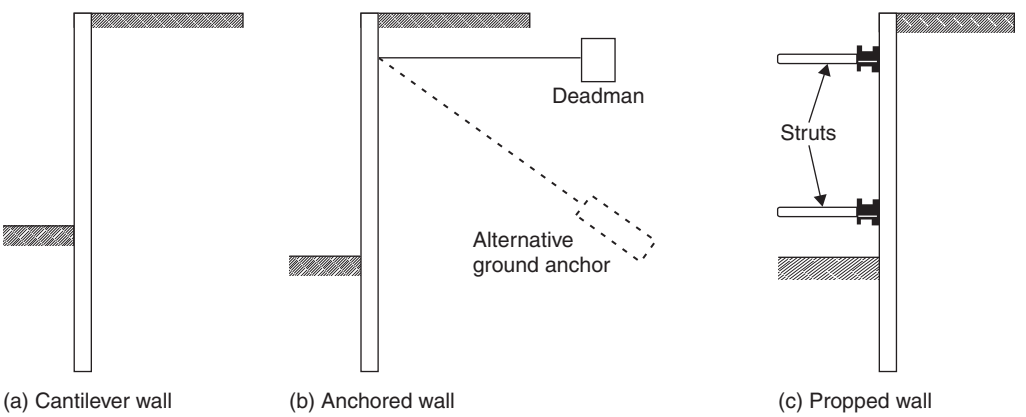


Figure 62.7 Embedded retaining wall
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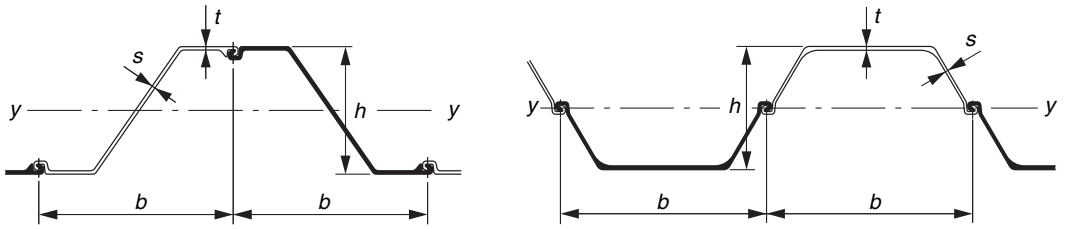


Figure 62.8 Standard sheet pile sections.
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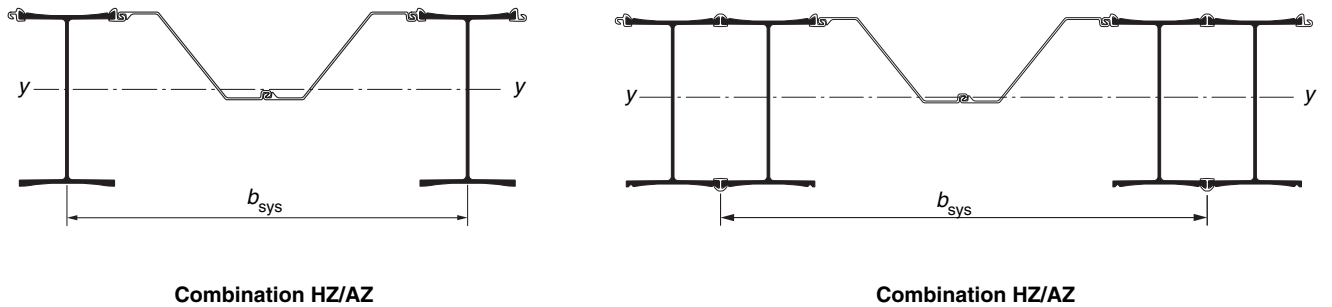


Figure 62.9 Sheet pile combinations
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Disadvantages of sheet pile walls include:

- maximum pile length approximately 30 m;
- corrosion can lead to a reduced section size;
- potential declutching in coarse-grained soils or in hard driving conditions.

62.3.2 King post walls

King post walls are also known as soldier piles with lagging or Berlin walls.

Isolated steel beams or posts are installed along the line of the wall either driven into position or placed in bored cast-in-place piles. As the excavation proceeds the space between the posts is filled using either timber railway sleepers, pre-cast concrete elements, *in situ* or spray concrete. As such, the wall is not suitable for excavation below the water table in coarse-grained soils without significant dewatering outside of the excavation. The posts are typically spaced 1 to 3 m apart and can be varied, making it easier to layout the wall to avoid localised obstructions. See **Figure 62.10**.

Propped excavations up to 20 m can be achieved; however, shallower depths of excavation are common due to the requirements for groundwater control.

62.3.3 Contiguous bored pile walls

A contiguous pile wall consists of bored cast-in-place piles installed along the line of the wall. The piles are installed with a

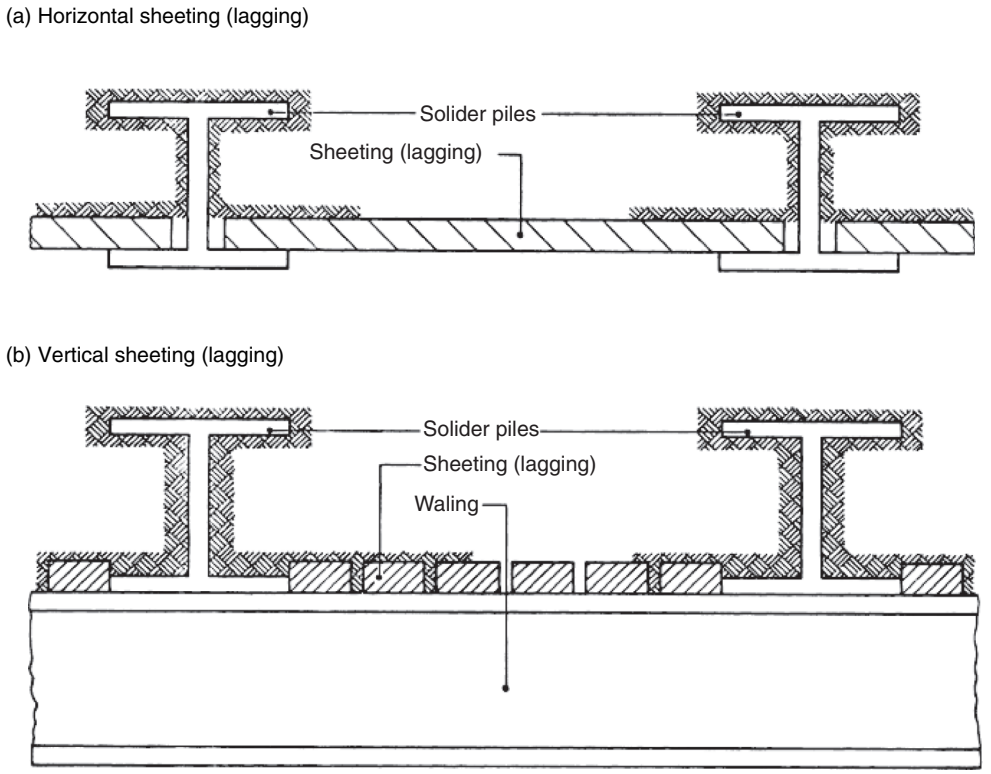


Figure 62.10 Soldier pile wall
Reproduced with permission from BS8002 © British Standards Institute (BSI, 1994)

gap between individual piles. The size of the gap is chosen on the basis of the site dimensions and also the particular ground conditions but is typically 50 mm to 150 mm; see **Figure 62.11.**

The piles can be installed as either continuous flight augur (CFA) or traditional bored piles using casings where required. The choice of pile type will depend on the ground conditions and depth of the wall. Details of the design and construction of piles are provided in Sections 5 and 8.

Due to the gap between individual piles, the wall is not water retaining and is, therefore, not suited to construction below the water table in coarse-grained ground conditions. Traditional bored piles with thick-walled casings may be required to overcome harder strata and obstructions.

Contiguous piled walls cannot be used as a permanent wall due to the gaps between individual piles; however, they can be made into a permanent solution through the use of a structural facing wall.

For CFA piles, the depth of retaining wall will be limited by the maximum depth of the CFA piles (refer to Section 8). Propped excavations up to 20 m can be achieved using contiguous pile walls.

62.3.4 Secant bored pile walls

A secant pile wall consists of intersecting bored cast-in-place piles installed along the line of the wall; see **Figure 62.12**. The pile construction may either use traditional piling rigs or CFA rigs. Refer to Chapters 79 *Sequencing of geotechnical works*, 81 *Types of bearing piles* and 82 *Piling problems* for more details on piling methods.

The female piles are installed first followed by the male piles, which cut through the female piles during installation. This overlap or secanting of individual piles provides a barrier to groundwater flow across the wall. The male piles are constructed with full strength reinforcement and provide the structural strength of the wall.

Three varieties of secant pile wall can be considered and are defined by the strength of the female piles as follows:

- hard/soft – a soft pile mix, typically cement and bentonite or bentonite and sand with a characteristic compressive strength of 1–3 N/mm^2;
- hard/firm – a 'firm' mix, with a characteristic compressive strength of 10–20 N/mm^2, and a retardant to reduce the strength of the piles during the drilling of the male piles;
- hard/hard – a full strength concrete, which can also be reinforced; see **Figure 62.13**.

The size of the piles is based on standard pile diameters.

The size of the overlap and pile spacing is chosen on the basis of the site dimensions and consideration of the installation tolerance of the piles and the depth of overlap or secanting required.

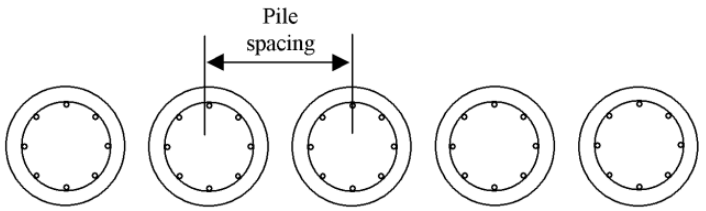


Figure 62.11 Contiguous pile wall
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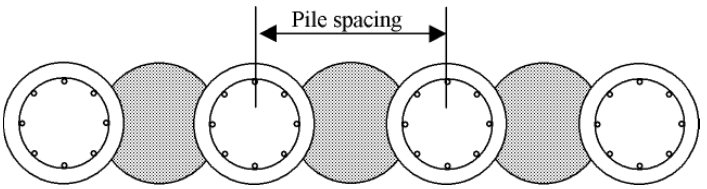


Figure 62.12 Hard/soft or firm secant pile wall
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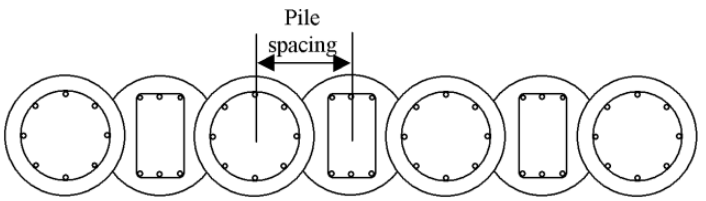


Figure 62.13 Hard/hard secant pile wall
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The hard/soft wall is not a preferred solution for permanent water-retaining walls due to shrinkage and cracking of the soft pile mix. To create permanent water retention a structural internal lining wall can be cast against the face of the wall.

For hard/firm and hard/hard basement walls, the need for an additional facing wall, e.g. a drained cavity and blockwork wall, will depend on the basement grade that is specified (refer to BS8102 (BSI, 2009)).

As a water-retaining wall, secant pile walls are suitable for permeable ground conditions where excavation is to extend below the water table. They are also suited to coarse-grained materials above the water table where the gaps in a contiguous piled wall could result in a loss of fines from the material retained behind the wall. Where permeable ground conditions are present above low permeability strata, e.g. gravels above clay, the female piles may be terminated with only limited penetration into the clays with the male piles extending lower to form a contiguous wall within the clays.

Secant pile walls are suitable for most ground conditions.

For CFA piles, the depth of the retaining wall will be limited by the maximum depth of the CFA piles (refer to Section 8). The maximum depth of a secant wall may be dominated by the installation tolerance of the wall and, therefore, the depth at which effective overlap of the piles can be achieved to provide a barrier to groundwater flow. Propped excavations up to 20 m can be achieved or up to 25 m for cased hard/hard solutions.

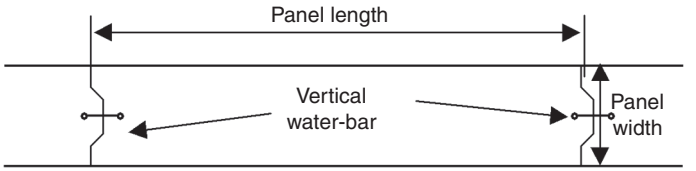


Figure 62.14 Diaphragm wall panel and joint
Reproduced with permission from CIRIA C508, Gaba *et al.* (2003), www.ciria.org

62.3.5 Diaphragm walls

Diaphragm walls are interlocking reinforced-concrete panels; see **Figure 62.14**.

The panels are formed by excavation under fluid to support the surrounding ground and lowering a prefabricated reinforcement cage into the trench excavation. Concreting is undertaken from the base of the pile using a tremie tube to displace the support fluid. Due to the requirements for a support fluid and the large volume of individual panels, diaphragm wall construction requires a larger laydown construction space than other forms of wall construction.

Panel widths and lengths are a function of the construction equipment available. Typical widths are 600, 800, 1000, 1200 or 1500 mm, which corresponds to the width of the grab or cutter used to form the hole. The typical grab length is 2.8 m but may be reduced to 2.2 m. The lengths are a function of the number of bites used to form the panel; see **Table 62.1**. For larger panel lengths, the ability of the support fluid to act in conjunction with the ground to arch around the large excavation to maintain a stable excavation should be considered.

Diaphragm walls are suitable for most ground conditions.

The depth of the wall is limited by the reach of the excavation machinery. Walls have been constructed to depths of up to 120 m; however, at this depth stringent installation tolerances are needed to provide an effective overlap of the panel width to the full depth of the walls. For such deep panels, detailed planning is required in relation to the practical difficulties of splicing cage sections together; the crane capacity to lift the cage; and the overall construction duration for the panels.

Due to the size of the individual panels, diaphragm walls are less flexible in dealing with any changes in the alignment of the wall and are, therefore, more suited to long straight walls as opposed to more complicated or changing geometries.

Water-bars constructed across the joints between individual panels make a diaphragm wall into a water-retaining wall.

Diaphragm walls are also known as slurry walls; however, care should be taken with this terminology as in the UK a slurry wall is an unreinforced wall used as a cut-off or containment wall rather than an embedded retaining wall.

Number of bites	Panel length, m Minimum	Maximum	Excavation sequence
1	W	W	1
2	W + T	2 x W – T	1 2
3	2 x W + T	3 x W – 2 x T	1 3 2

Notes:
1. W is the grab length available (typically 2.8 m).
2. T is the grab width: 600, 800, 1000, 1200 or 1500 mm.

Table 62.1 Diaphragm wall: typical panel widths
Data taken from CIRIA C580, Gaba *et al.* (2003), www.ciria.org

Variations can include:

- T-shaped panels for increased strength and stiffness;
- post-tensioned panels;
- pre-cast panels, with or without pre-tensioning.

Pre-cast panels are constructed by lowering the panel into the support fluid filled trench and sealed into the ground with *in situ* concrete or grout placed by means of a tremie pipe. The size of the panels for pre-cast walls is limited practically by the ability to lift the panels and place them into the trench.

62.4 Hybrid walls

Hybrid walls combine aspects of gravity and embedded elements to retain the ground. An example of a hybrid gravity wall is a concrete stem wall, L-shaped or T-shaped, founded on individual piles; see **Figure 62.15**. Such a hybrid solution is suited to circumstances where there is insufficient sliding or bearing capacity for a gravity wall but sufficient capacity can be achieved using piled foundations for the wall.

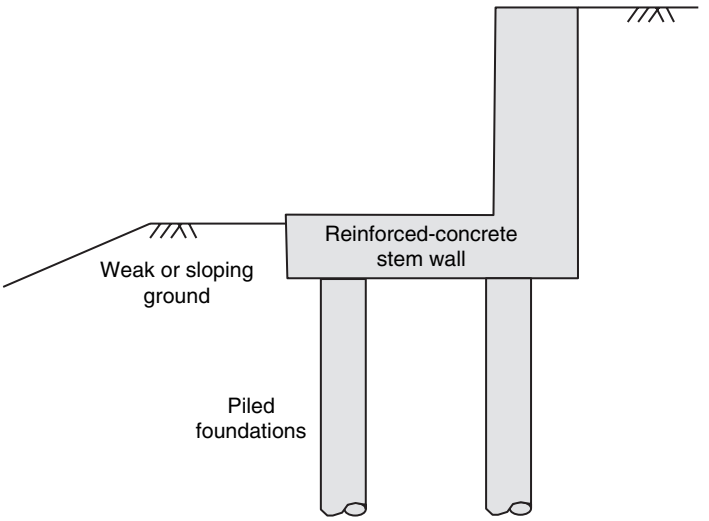


Figure 62.15 Example of a hybrid retaining wall

62.5 Comparison of walls

A comparison of the advantages and disadvantages of the different retaining wall types is given in **Table 62.2**. Chapter 63 *Principles of retaining wall design* gives further information on factors affecting the choice of retaining wall types.

Wall type	Advantages	Disadvantages
Gravity walls – general	■ Minimises construction activities on site by using pre-cast or modular systems ■ Cost-effective retaining wall solution for supporting raised ground levels	■ Requires site dewatering for construction below the water table ■ Requires temporary open-cut excavation for retaining wall construction below existing ground level
Pre-cast reinforced-concrete stem walls	■ Provides a wall with a predictable surface finish	■ Height of wall may be limited practically by transporting and lifting requirements. 1 to 3 m typical retained height ■ Changes in retained height or plan alignment need to be planned in detail for prefabrication of units ■ Non-draining, appropriate drainage required
Masonry walls	■ Can provide a wall with a predictable surface finish ■ Can be installed around obstructions at isolated points ■ Can be built to a batter	■ Construction requires bricklaying skills ■ Foundation slab required ■ Non draining, appropriate drainage required
Dry-stack masonry walls	■ Simple manual construction ■ Distinctive course effect ■ May be used with planting ■ Can produce curves in plan	■ Foundation slab required ■ Non draining, appropriate drainage required
Crib walls	■ Simple manual construction ■ Distinctive 'honeycomb' appearance ■ May be used with planting ■ Can produce curves in plan	■ Foundation slab required ■ Free draining although back drain may also be required ■ Backfill in compacted layers

Table 62.2 Comparison of retaining walls
Reproduced with permission from Gaba *et al.* (2003) and Chapman *et al.* (2000)

Wall type	Advantages	Disadvantages
Gabion walls	■ Simple construction ■ Foundation slab may not be required ■ Fully draining, no back drain required unless they are filled with non-draining materials ■ Can be used with planting ■ Can produce curves in plan	■ A flexible system which can result in an undulating wall profile
Embedded retaining walls – general	■ Minimise volume of excavation ■ Enable deep excavation adjacent to existing structures and utilities ■ Enable deep excavation below the water table (water-retaining embedded walls only)	■ Specialised construction plant and operation on site ■ Costly in comparison with open-cut construction methods
Sheet pile walls	■ Provides an economic embedded wall with a predictable surface finish ■ No arisings to be removed ■ Suitable as a water-retaining wall ■ Can be used as both a temporary and a permanent wall	■ Maximum pile length approximately 30 m ■ Potential declutching in coarse-grained soils
King post walls	■ Can be installed around obstructions at isolated points	■ Not suitable for retaining water in the long term ■ Cannot be used for excavation below the groundwater table in coarse-grained soils
Contiguous bored pile walls	■ The cheapest form of concrete piled wall	■ Not a water-retaining solution ■ Not a permanent solution in any soil due to the gaps between piles, unless a structural facing is applied
Hard/soft secant bored pile walls	■ Acts as a water-retaining temporary wall ■ The use of soft piles enables hard piles to be formed using lower-torque rigs than for hard/hard secant piles	■ Not usually a permanent solution for retaining water ■ The soft pile mix is not significantly cheaper than concrete. Local concrete plant is often unable to batch the soft material, so site batching is required ■ Depth is limited by the verticality tolerance, which may determine the depth of secanting
Hard/firm secant bored pile walls	■ A permanent water-retaining wall ■ The firm material for the primary (female) piles is either a standard concrete mix, retarded to reduce its strength when the secondary (male) piles are constructed or a reduced strength concrete mix	■ Depth is limited by the verticality tolerance, which may determine the depth of secanting
Hard/hard secant bored pile walls	■ A permanent water-retaining wall ■ Installed using standard piling plant with high-torque rigs	■ The cutting of the hard primary (female) piles requires high-torque rigs or oscillators ■ Depth is limited by the verticality tolerance, which may determine the depth of secanting
Diaphragm walls	■ A permanent water-retaining wall ■ Can be installed to great depths provided the verticality tolerances can be accepted ■ In some circumstances the face of the diaphragm wall can form the final finish subject to some surface cleaning and removal of protuberances ■ Fewer joints compared with piled walls	■ Horizontal continuity is difficult to achieve between panels ■ Cannot follow intricate plan outlines ■ The installation equipment is extensive, requiring a large site area for accommodation of the support fluid plant, reinforcement cages and the excavation plant ■ Disposal of the support fluid is costly
Hybrid walls	■ Provide a practical retaining wall solution where the site conditions will not allow a gravity wall but do not require an embedded retaining wall	■ Combination of different wall and foundation elements results in site- and solution-specific wall type

Table 62.2 (continued)

62.6 References

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62.6.1 Further reading

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NAVFAC Design Manual 7.02. *Foundations and Earth Structures*. Washington, USA: Naval Facilities Engineering Command (NAVAC), US Department of the Navy.

62.6.2 Useful websites

CIRIA; www.ciria.org

Federation of Piling Specialists; www.fps.org.uk/index.php

Hong Kong Geo Publications; www.cedd.gov.hk/eng/publications/manuals/geo_publications.htm

Piling Handbook; www.arcelormittal.com/sheetpiling/page/index/name/arcelor-piling-handbook

It is recommended this chapter is read in conjunction with

- Chapter 85 *Embedded walls*
- Chapter 91 *Modular foundations*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 63
Principles of retaining wall design

Michael Devriendt Arup, London, UK

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Prior to carrying out the design of a retaining wall, basic concepts relating to the principles of design should be understood by the designer. Considerations such as the characterisation of the wall, the construction sequence to form the retaining wall, performance criteria and the selection of an appropriate wall type should be understood. Following this appreciation of basic concepts, it is common for a designer to select design parameters for analysis in accordance with appropriate standards. The designer should also understand serviceability issues relating to the magnitude of ground movements and the potential for damage to nearby structures.

This chapter provides the reader with a basic understanding of the above conceptual issues relating to the design of retaining walls and provides references to other material where more detailed descriptions may be found.

63.1 Introduction

This chapter provides a description of the basic concepts that govern retaining wall design. The concepts described should be appreciated by designers before carrying out the design of a wall as outlined in Chapters 64 *Geotechnical design of retaining walls* to 67 *Retaining walls as part of complete underground structure*. This chapter, therefore, serves as a basis for describing many of the conceptual issues that need to be addressed when planning the design of a retaining wall.

63.2 Design concepts

63.2.1 Geotechnical characterisation of retaining walls

When designing a retaining structure, it is important to understand how the retaining wall can be characterised. As described in Chapter 62 *Types of retaining walls*, retaining structures can broadly be considered to be characterised as gravity or embedded walls or a combination of both. Methods of analysis of the wall types are given in Chapters 64 *Geotechnical design of retaining walls* to 67 *Retaining walls as part of complete underground structure*.

For analysis or back analysis of a retaining structure it is fundamental to obtain an understanding of ground and groundwater conditions. The use of soil parameters and adopting different groundwater conditions in design is discussed in greater detail later in this chapter and in Chapters 64 *Geotechnical design of retaining walls* to 67 *Retaining walls as part of complete underground structure*.

63.2.2 Construction sequence

63.2.2.1 General

The construction sequence that is followed to construct a retaining wall can have a significant effect on the design, cost and time spent constructing the wall. This section discusses the conceptual decisions that influence the selection of an appropriate construction sequence.

63.2.2.2 Design

Displacement of the retaining wall influences the *in situ* stress state in the ground around the retaining wall. Earth pressure theory was introduced in Chapter 20 *Earth pressure theory*. During installation of the wall and as displacement of the wall occurs, earth pressures will change from an *in situ* Ko condition to active and passive states depending on whether relaxation or compression of the soil occurs behind or in front of the wall respectively. Further information relating to earth pressure theory may be found in Chapter 20 *Earth pressure theory*.

Figure 63.1 illustrates this concept of the amount of displacement (or strain) governing the stress state on the active

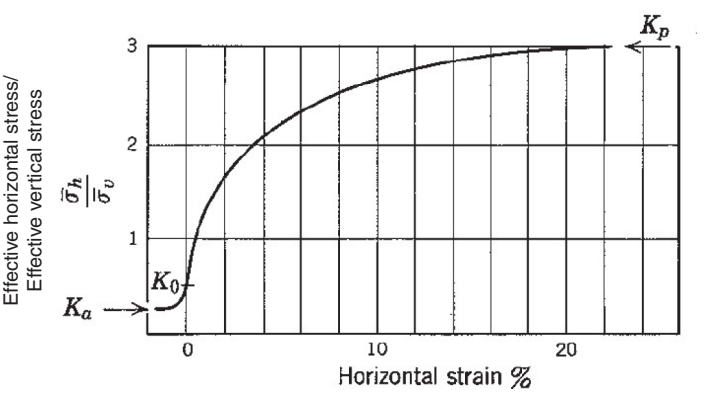


Figure 63.1 Relationship between earth pressure and horizontal strain by Terzhagi (1954) for normally consolidated sand (different scales for active and passive)

Reproduced from Simpson (1992), all rights reserved

and passive sides of the wall. An active state (K_a) is reached at relatively small strain, while larger strains are required to reach a passive state (K_p). The earth pressures in front or behind a retaining structure will influence the bending moment, shear forces and any prop forces supporting the wall. The construction sequence that is followed can also influence the displacement of a retaining structure and, therefore, affect the design of the retaining structure.

To appreciate the differences caused by construction sequence, an analysis that considers staged construction is necessary (i.e. the limit equilibrium method of analysis described in Chapter 64 *Geotechnical design of retaining walls* is not suitable). The use of more complex analysis methods such as pseudo and full finite element or finite difference programs are required to accommodate the effects of construction sequence. The use of such programs is discussed in Chapter 6 *Computer analysis principles in geotechnical engineering* and their use for retaining wall design in particular is also covered in Chapters 64 *Geotechnical design of retaining walls* to 67 *Retaining walls as part of complete underground structure*.

Displacements of the wall and those caused from construction of the wall itself will influence ground movements around the retaining wall. This is discussed in further detail in section 63.3.

63.2.2.3 Basement construction

The excavation of basements, particularly in confined urban sites, is a situation where the construction sequence has a significant impact on the successful outcome of the project. In particular, careful consideration of the construction sequence is required where there is an existing basement that occupies the site and the retention of existing basement retaining walls is necessary.

63.2.2.4 Top-down and bottom-up

Where retaining walls are used for excavation of basements and confined spaces such as station box structures, two generic forms of construction sequence are commonly used to form the basement space: 'top-down' and 'bottom-up'. Variants of these generic sequences are also commonly adopted to suit particular site constraints.

Top-down sequences involve constructing sequential permanent basement slabs to support the retaining walls as the excavation progresses. Bottom-up sequences use temporary propping to support the basement space during the excavation, then construct the permanent basement slabs after the excavation is complete.

The two generic sequences are compared in **Table 63.1** through reference to some common considerations. Photographs of construction carried out in top-down and bottom-up sequences are shown in **Figures 63.2** and **63.3**.

The construction sequence followed can have a significant effect on the programme and the cost of the proposed development. In particular, the programme for development in urban sites can have an effect on the overall successful completion of a project. The effect of delay on costs is discussed in greater detail in Chapter 7 *Geotechnical risks and their context for the whole project*.

63.2.3 Design requirements and performance criteria

63.2.3.1 General

Prior to the design of a retaining structure, the designer must first gather the generic performance criteria for the structure. The performance criteria may vary depending on the requirements of the structure during its design life (for instance, taking into account that vertical loading will be applied to the retaining structure at some time in the future).

63.2.3.2 Design life

The design life of a structure may influence several issues relating to the design of a retaining wall.

The forces exerted on a retaining wall may also be influenced by variable loads such as wave or seismic loading. The magnitude of these forces is commonly related to a return period and probabilistic approach of exceedance for a given structural design life; refer to Eurocode 8 Parts 1 (BSI, 2004b) and 5 (BSI, 2004c) and BS6349 Parts 1 (BSI, 2000) and 2 (1988) (BSI,

Property	Top-down	Bottom-up
Construction of superstructure	Allows concurrent construction of basement and superstructure	Generally basement must be constructed prior to construction of superstructure
Bearing piles used to support superstructure within plan area of basement space	Piles installed with low cut off and columns plunged into piles	Piles can be installed before or after basement excavation. If before excavation, the use of plunged tubes for reception of columns may be necessary
Spoil removal	Spoil needs to be removed from moling hole in slab	Allows 'clear space' excavation of spoil generally allowing quicker excavation of spoil provided lorry movements at surface aren't restricted
Ground movements	Generally a greater stiffness basement space is created, therefore, minimising ground movements	Temporary propping tends to be less stiff than permanent propping resulting in greater movements; however, this can be mitigated by using additional temporary propping

Table 63.1 Top-down and bottom-up considerations

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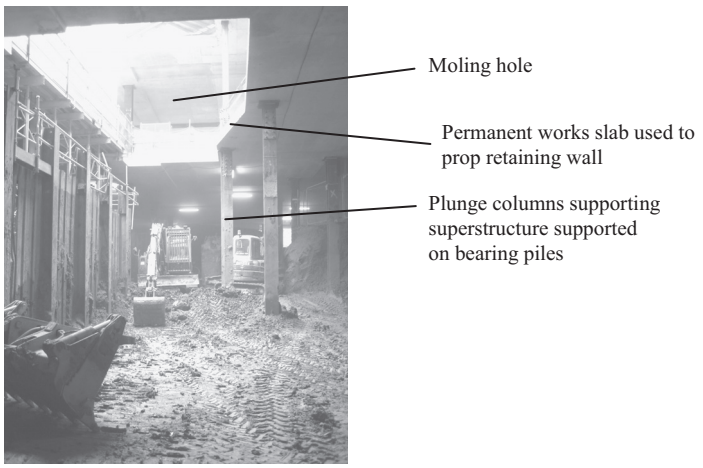


Figure 63.2 Example of top-down excavation (One New Change, London)
Photo courtesy of Arup

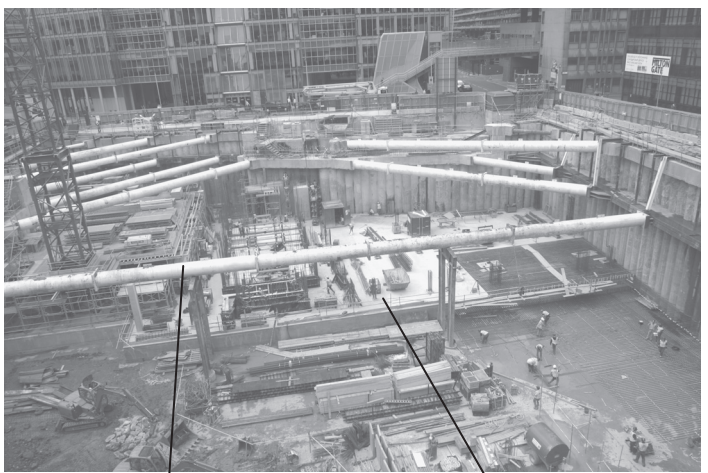


Figure 63.3 Example of bottom-up excavation (Ropemaker, London)
Photo courtesy of Arup

2000) on how to select return periods. The proposed design life of a structure can influence the return period that is considered in the design and, therefore, the magnitude of loads. In particular, for the design of retaining walls in seismic regions the size of the active and passive zones may be affected by the effect of seismic loading. Consequently the positions of any tie backs should be reviewed with respect to the seismic design.

The integrity of the overall structural form of the retaining wall and structural support also needs to be considered. Where a structure is exposed to adverse conditions (for instance sheet piles exposed to corrosion in a saline environment) the reduction in thickness of section needs to be accounted for in the design – see the *Piling Handbook* (ArcelorMittal, 2008).

63.2.3.3 Temporary and permanent works

The responsibility for design of the permanent and temporary works is often split between companies or groups of designers. As identified in section 63.2.2, the construction sequence that is followed can have a significant effect on the design of the retaining wall. It is, therefore, important that the temporary and permanent works designers share design input information and the construction sequence that each respective designer anticipates is clear. In particular consideration to respective prop and retaining wall properties and wall stiffness parameters should be considered. An analysis should be carried out to check the entire sequence (both temporary and permanent works).

63.2.3.4 Drained or undrained

When determining whether drained or undrained soil conditions apply, account should be taken of the rapidity with which drained conditions are achieved. For fully saturated soil, undrained conditions are characterised by there being no changes in volume, although a change in shape can occur. It is possible for such conditions to prevail in the short term in low-permeability soils such as high-plasticity sedimentary clays where excess water pressure dissipates slowly and suction is generated, by loading and unloading, respectively. The detailed fabric of the soil dominates the magnitude of the water pressure at equilibrium. The importance of obtaining an accurate record and gaining a good understanding of the soil fabric cannot be over-emphasised (refer to Chapters 13 *The ground profile and its genesis* to 15 *Groundwater profiles and effective stresses* and 46 *Ground exploration*).

Stiff overconsolidated clays are generally jointed, fissured and layered as a result of their depositional history. Although the permeability of the clay between the joints and discontinuities may be low, the presence of small sandy or silty partings can have a disproportionate effect on the magnitude of the coefficient of permeability of the soil. This influences the rate at which water may flow from one part of the soil mass to another and the rate at which water may be drawn into the soil. To assess whether undrained conditions are likely to prevail for a significant time the following factors should be considered:

- Soil stratigraphy and fabric. There is usually no dilation of pre-sheared surfaces, which means that shearing can occur at constant volume under drained residual conditions. This should not be confused with undrained behaviour.
- The mass *in situ* permeability of the soil (in the vertical and horizontal directions).
- The proximity and likelihood of available water.
- The soil stiffness, which affects the value of the coefficient of consolidation, c_v. This might be particularly high following a change in the direction of the stress path.
- Previous experience of construction in similar ground conditions.

The designer should evaluate drainage paths and assess the duration over which drained conditions may be restored.

As a general guide, undrained conditions may be assumed in the short term where the mass *in situ* permeability of the ground is low (i.e. a coefficient of permeability of the order of 10^{-8} m/s or less; see BS 8002 (BSI, 1994)). Where the mass permeability is not low and in coarse-grained soils, drained conditions should be assumed in design.

63.2.3.5 Cycling of loads on integral bridges

For the design of bridges, it is common to have an earth-retaining structure forming part of the bridge abutment supporting either side of the bridge deck; see **Figure 63.4**. To avoid the use of bearings to support the deck structure, which can reduce long-term maintenance costs, it is common to make the abutment integral with the deck (an integral bridge).

For integral bridges, temperature variations will induce cyclic movement of the abutments and the soil retained behind them. The effect of these cyclic movements on the earth pressure influences the design of the retaining wall. In addition, in granular soils, the cyclic behaviour may result in settlement occurring behind the retaining wall. This may have an effect on the design of the road or railway approach structure to the bridge.

Various design approaches have been proposed to account for the increased earth pressures due to cyclic loading; see, for instance, the UK's Highways Agency document, BA42/96 (1996). An appraisal of design approaches is given by Lehane (2006) through reference to centrifuge laboratory model testing.

63.2.3.6 Performance criteria for horizontal and vertical support

Although the primary function of a retaining wall is, in many instances, to provide lateral support and retention of the ground and groundwater, some retaining walls, particularly those used for basement construction, may also be required to provide vertical support. For these scenarios both the allowable vertical

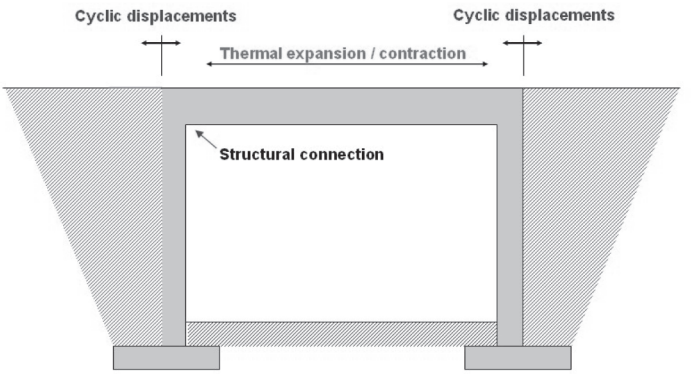


Figure 63.4 Schematic of integral bridge
Image courtesy of Arup

Factor	Description or comment
Availability of space around perimeter of site for retaining wall construction	Where there is a space constraint and the client wants to build up to the property boundary an embedded retaining wall is generally favourable over a gravity wall. The thickness of an embedded retaining wall section is also a consideration. Greater thickness walls generally increase stiffness and reduce movement; however, they will take up more space.
Unsupported height of retaining wall	Where large unsupported heights of a retaining wall are desirable, generally a thicker retaining wall section will be required to have sufficient moment capacity and limit displacements of the wall to acceptable magnitudes.
Water retention	The permeability of the ground and requirements for the basement space influence the selection of wall. Generally, the fewer the connections in the wall, the lower the leakage rate and the greater the water retention. Improved long-term basement space may be obtained by using drained cavities or inner lining walls. Guidance on how to achieve watertightness requirements for bored piles, diaphragm walls and sheet pile walls is provided in CIRIA Report 139 (Johnson, 1995) and BS 8102 (BSI, 1990).
Stiffness	Higher system stiffness (i.e. the wall and its support system) leads to reduced wall and ground movement. This is achieved by increasing the thickness of the wall or increasing the number of props or ties supporting the wall.
Cost	Cost savings may be obtained where continuous support is not required for the purpose of water retention (for instance, selection of a contiguous bored pile wall rather than a secant wall). The relative costs of materials (for instance steel for sheet piles) will also fluctuate with time.
Programme	Similar to the cost section above, savings in the programme can be made if contiguous bored pile walls are feasible rather than a secant wall. Savings can also be made by reducing the overlap for secant walls or increasing the diameter of the piles (therefore, reducing the number of piles to install).
Design life	The design life of a retaining wall is a primary consideration where the imposed forces on a retaining wall are governed by design life (for instance seismic or wave loading) or where the wall degrades or corrodes with time (for instance sheet piling in a maritime environment).
Embodied energy	The total energy used in forming the retaining wall. Further details of these considerations are given in Chapter 11.

Table 63.2 Factors influencing the selection of retaining walls

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and horizontal displacement should be considered. Further discussion regarding the design for horizontal loading is given in Chapter 64 *Geotechnical design of retaining walls* and for designing for vertical loading in Chapter 67 *Retaining walls as part of complete underground structure*.

63.2.3.7 Design of retaining structures in a marine environment

Further consideration, when designing structures in a marine environment, are the effects of tide levels, swell, storm surge, wave impact loads and sea level rise over the design life of the retaining structure. By considering these effects, overtopping of the retaining wall can be minimised. Allowance should also be made for the water level difference between the active and passive sides of the retaining wall. BS6349 Parts 1 (2000) and 2 (1988) (BSI, 2000) should be referred to for further guidance on appropriate considerations for design in a marine environment.

63.2.4 Consideration for wall selection

Chapter 62 *Types of retaining walls* describes the main types of retaining wall that are commonly constructed to retain soil and groundwater.

Table 63.2 provides a summary of the factors that influence the selection of retaining walls.

63.3 Selection of design parameters

63.3.1 Design philosophy and approach (limit states, moderately conservative, most probable)

63.3.1.1 Limit states

For the design of retaining walls, appropriate soil and groundwater parameters need to be selected for the calculations. The objective of design calculations is to demonstrate that a limit state will not be exceeded. Various limit states are generally considered. Current guidance (BS EN 1997–1:2004: Eurocode 7, BSI, 2004a) identifies that a designer must check that ultimate, serviceability or accidental limit states are not exceeded.

A serviceability limit state (SLS) defines the limit of satisfactory functioning of the structure or structural members under normal use, the comfort of those affected by the design and the appearance of the construction works. Exceedance of a serviceability limit state will typically result in inconvenience, disappointment and less manageable costs.

An ultimate limit state (ULS) is breached where one or more of the following occurs:

- loss of equilibrium of the structure or any part of it considered as a rigid body;
- failure by excessive deformation or transformation;
- deformation resulting in a mechanism or rupture;
- loss of stability of the structure or any part of it, including supports and foundations;
- failure caused by fatigue or other time-dependent effects.

Exceedance of an ultimate limit state will result in serious failure involving risk of injury or major cost. An appropriate design must render this very unlikely or make this an 'unrealistic' possibility.

63.3.1.2 Selection of characteristic soil parameters

The zone of ground governing the behaviour of a geotechnical structure at a limit state is much larger than a test sample or the zone of ground affected in an *in situ* test. Consequently the value of the governing parameter often requires careful consideration of a range of values covering a large surface or volume of the ground.

The characteristic value should be a cautious estimate. Therefore, the selection of the characteristic value should be more conservative than selecting a most probable value.

If statistical methods are employed in the selection of characteristic values for ground properties, the methods used should differentiate between local and regional sampling and should also consider any experience in using comparable ground properties. If statistical methods are used, for the limit state the characteristic value is calculated so that the probability of a worse value is not greater than 5%.Under certain circumstances the use of statistical methods may not be appropriate: an example is given in Case Study 1.

All the geotechnical expertise of the designer is incorporated in the assessment of the characteristic values of ground properties.

Reference should be made to Chapters 17 *Strength and deformation behaviour of soils* and 27 *Geotechnical parameters and safety factors* for further discussion on the selection of design parameters.

Examples of the selection of characteristic soil parameters are given in Simpson and Driscoll (1998) *Eurocode 7 – A Commentary*. An extract from this document including an example is provided in Case Study 1.

63.3.2 Case Study 1

Figure 63.5(a) shows the results of a series of undrained shear strength measurements in London Clay. The measurements were made using unconsolidated undrained triaxial tests. A statistical mean line has been drawn through the data and it is clear that undrained strength increases with depth. A characteristic line is required and this should depend on how the characteristic values will be used – what is the limit mode being considered? For example, if the undrained strength is needed for the calculation of ground movements around a retaining wall, a value such as the 'cautious (average)' value on the figure could be used. However, for a problem in which failure might take place in a small zone of soil, such as an isolated foundation placed at deep level, a more cautious value – the 'cautious (local)' value – should be adopted. From these boreholes, results from standard penetration tests were also available, as shown in **Figure 63.5(b)**. In London Clay, there is usually a constant factor between standard penetration and undrained shear strength results; the factor is about 4.5 to 5. However,

if the mean line from the SPT results is transferred onto the undrained strength plot as in **Figure 63.5(c)**, it appears that the normal correlation does not work. In fact, the measured undrained strengths are remarkably high: they are consistent with a very low water content, which was measured, but this might simply mean that the samples had dried out on the way to the laboratory, though there was no reason to suspect this. **Figure 63.5(c)** also shows lines representing mean values through the data from other nearby sites, both for undrained shear strength and SPT results. The usual close correlation applies to these, and it is clear that the undrained strengths for the new site are remarkably high.

On the basis of these inconsistent data sets, what value should be used as the characteristic undrained strength? The values measured in the triaxial tests should not be ignored, but the SPT results and data from adjacent sites should also affect the decision. The characteristic value for these data is shown in **Figure 63.5(d)**. This is less than the initial assessments in **Figure 63.5(a)**, which were based on the triaxial results only, and is closer to a lower bound of this particular set of triaxial results.

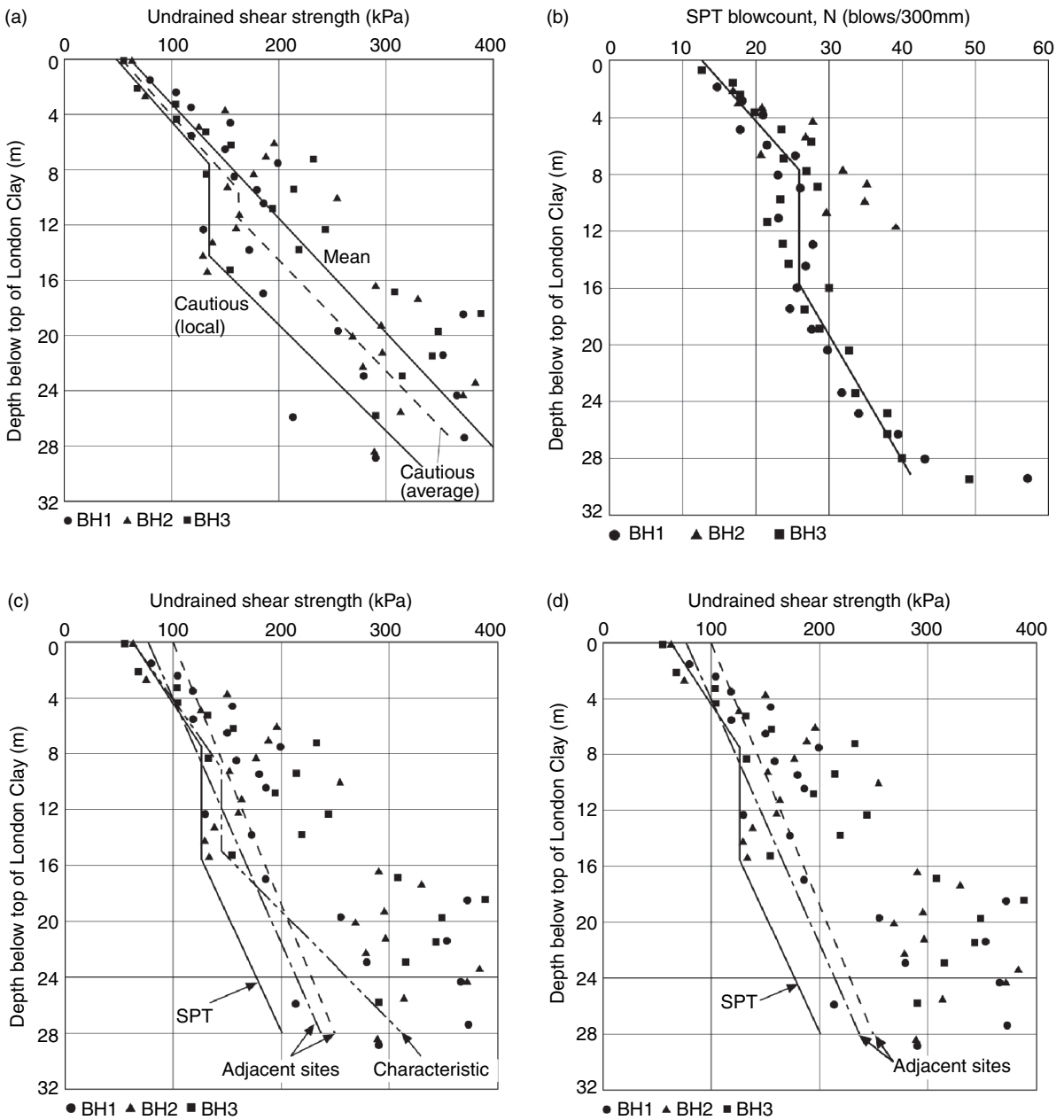


Figure 63.5 Tests in London Clay
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Engineers often need to follow this sort of process when trying to interpret real data. It may be that statistical methods could trace a similar logical sequence. However, this would require quite advanced methods and any statistical approach that failed to take account of the diverse array of data, typically available, would be harmful to the design process.

[Case study text reproduced from Simpson and Driscoll (1998), © BRE, reproduced with permission.]

63.3.2.1 Selection of characteristic groundwater parameters

General

A conservative estimate of the active and passive groundwater conditions should be taken when selecting the characteristic levels and groundwater pressure profiles on the active and passive sides of a retaining wall. The designer should check that the following have been considered:

- the proximity of sources of free water and the likelihood of such sources becoming available over the design life of the wall;
- the effects on the local hydrogeology of the site due to the construction of the wall, for example, the potential damming of natural groundwater flows or a long-term rise in aquifer groundwater levels;
- the effects of wall toe levels not reaching the target design levels due, for example, to obstructions or hard driving resistance;
- the effects of drainage or dewatering during construction and during the lifetime of the wall;
- changes to water pressures due to the growth or removal of vegetation;
- changes to water pressures due to long-term climatic variations.

Based on the above considerations, the designer should determine:

1. The water pressure and seepage forces that represent the most unfavourable values that could occur in extreme or accidental circumstances at each stage of the wall's construction sequence and throughout its design life. An example of an extreme or accidental event may be a burst water main in close proximity to the wall.
2. The water pressures and seepage forces that represent the most unfavourable values likely to occur in normal circumstances at each stage of the wall's construction sequence and throughout its design life. Extreme events such as a nearby burst water main may be excluded, unless the designer considers that such an event may reasonably occur in normal circumstances.

The groundwater pressure corresponding to (1) should be compared to the ULS while (2) should be compared to the SLS.

Undrained conditions

Where undrained conditions are considered to prevail during the construction of the wall, total stress considerations will apply. In such circumstances, there may be a tendency for tension cracks to develop in the retained soil. These tension cracks can be considered to have the potential to flood, depending upon the availability of water. Tension cracks may be less likely behind a propped wall compared with a cantilever wall. The depths of tension cracks and the groundwater pressure to assume are provided in CIRIA C580 (Gaba *et al*., 2003) and Chapter 64 *Geotechnical design of retaining walls*.

In the case of an embedded cantilever wall or where water is expected to be present, e.g. from water-bearing deposits overlying stiff clay, hydrostatic pressure should be used on the retained side of the wall.

CIRIA C580 (Gaba *et al*., 2003) proposes that where water is not expected, the total pressure acting on the retained side of the wall at any depth below the ground surface should be assumed to be not less than a minimum effective fluid pressure (MEFP) of $5z$ kN/m² (where z is the depth below the ground surface).

Drained conditions

Where the ground fabric and permeability indicate that drained conditions should be designed for, effective stress considerations will apply. The designer should evaluate the water pressure over the whole of the wall assuming steady-state conditions at relevant stages of construction and over the lifetime of the wall. Flownets are commonly used to establish the groundwater pressure distribution around the retaining wall. It should be noted that in reality, the ground is likely to be anisotropic with a non-uniform coefficient of permeability, which may greatly affect the pore water pressure distribution.

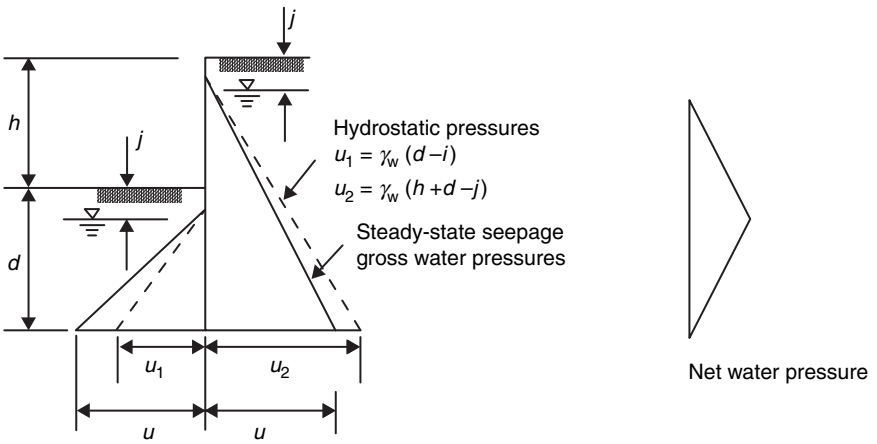
A commonly used simplified method of evaluating water pressure around an embedded retaining wall to a wide excavation in uniform isotropic conditions is shown in **Figure 63.6**.

This simplified method applies where the differential head of water across the wall dissipates uniformly along the length of the flow path adjacent to the wall and is sometimes referred to as linear seepage. This assumption tends to underestimate water pressures beneath narrow excavations. In general, the expressions in **Figure 63.6** should not be applied to excavations of widths that are less than four times the differential water head across the wall, BS 8002 (BSI, 1994). At greater excavation widths, the assumption of linear seepage provides a good approximation to the pore water pressure distribution around the wall.

In anisotropic ground conditions and in situations where the design of the wall is sensitive to small changes in earth and water pressures around the wall, water pressures should be evaluated by computer-based analytical methods or by flownets.

Mixed undrained and drained conditions

It is possible for the retained side of the wall to be considered drained (e.g. due to unloading laterally and the likelihood of open fissures allowing water into the clay and the potential for consequential softening or due to the presence of water-bearing silt and sand partings in the clay) while the restraining soil is considered to be undrained (e.g. due to an impermeable wall



= Assumption (1)
Uniform dissipation of differential head along flow path adjacent to the wall

$$u = \frac{2(d + h - j)(d - i)}{2d + h - i - j} \gamma_w$$

= Assumption (2)
Average hydrostatic pressure at wall toe

$$u = \frac{u_1 + u_2}{2} = \gamma_w \left\{ d + \left(\frac{h - i - j}{2} \right) \right\}$$

Figure 63.6 Uniform ground: simplifying assumptions for calculating water pressures for steady-state seepage conditions
Reproduced from Burland *et al.* (1981)

extending into impermeable ground thereby providing a cut-off to the groundwater recharge beneath the excavation on the restraining side). There is less likelihood of fissures opening and allowing water into the restraining clay due to its lateral confinement. In such circumstances, effective stress conditions may apply on the retained (active) side. Total stress conditions may apply in the restraining soil on the passive side and, possibly also, on the retained (active) side below the depth of the tension cracks.

The above is a very specific example used to highlight a scenario where mixed undrained and drained conditions may apply. This particular combination is quite often appropriate, though many other scenarios are possible.

The designer should carefully consider the likely ground behaviour on each side of the wall before finalising the design assumptions.

63.3.2.2 Other considerations

The designer should also include consideration of the following, prior to carrying out the retaining wall design:

- Adequate consideration of the dead and live loading that the retaining wall may be subjected to throughout the design life.
- Unplanned excavation on the passive side of the retaining wall. Generally assumed to be the lesser of 0.5 m or 10% of the total height retained for cantilever walls, or the height retained below the lowest support level for propped or anchored walls.
- Potential for softening of the soil on the passive side of the retaining wall.
- Effect of vertical forces applied to the wall through wall friction.

Further guidance on the selection of the above is given in CIRIA C580 (Gaba *et al.*, 2003) and Chapters 64 *Geotechnical design of retaining walls* to 67 *Retaining walls as part of complete underground structure*.

63.3.3 Temporary works design

The approach taken for temporary works design is a matter of risk assessment, management and mitigation. It is very important that the roles and responsibilities of each of the parties (client, owner, designer, contractor and sub-contractor) are clearly defined and fully understood by all parties in respect of the design and construction of the temporary (and permanent) works; also refer to Chapters 42 *Roles and responsibilities*, 78 *Procurement and specification* and 96 *Technical supervision of site works*.

The design of temporary works should be demonstrably robust. This robustness should be provided by identifying the hazards and risks and ensuring that these are adequately addressed by the temporary works design. The principal risks

lie in predicting ground behaviour and dealing with parameter uncertainty. For example:

- Will the ground respond in a drained or undrained manner, or a combination of both during construction?
- If undrained, will tension cracks develop? Are these likely to be dry or full of water?
- What groundwater regime will apply during construction?
- What provision should be made to allow for the deterioration in soil strength due to the installation of the wall and its support system and the disturbance due to excavation and partial drainage during the period of construction?
- What measures or controls will be applied to confirm that:
  - **(a)** The loading conditions assumed in design will not be exceeded?
  - **(b)** Unplanned excavation of the formation will not occur (see section 63.3.2.2)?
  - **(c)** The actual support (props, anchors, berms, etc.) to the wall will be as assumed in design?
  - **(d)** The actual wall performance (deflection, watertightness, etc.) will remain within pre-defined limits?

The assumptions made in temporary works design in respect of the above depend upon how the associated risks are mitigated during construction. Risk mitigation is undertaken through close control of the process of design and the construction of the temporary works on site. This requires an integrated interactive approach between design and construction on site with the ability to adapt the design quickly to suit changes in construction methods and *vice versa*. Appropriate contractual arrangements should be in place to permit this.

Special consideration should be given where there is uncertainty whether undrained or drained conditions may be assumed for the duration of the temporary works (see section 63.3.2.1 above). In terms of risk, three possible sets of ssumptions are possible for the design of temporary works in stiff clays:

- undrained conditions on both sides of the wall;
- mixed undrained and drained conditions;
- drained conditions on both sides of the wall with a steady-state groundwater regime.

The assumption of undrained conditions on both sides of the wall is associated with significant risk, while least risk is attached to the assumption of drained conditions. Further consideration of these assumptions is given in CIRIA C580 (Gaba *et al*., 2003).

63.4 Ground movements and their prediction

63.4.1 General

For the design and construction of retaining walls, it is inevitable that some movement will occur both in front and behind the retaining wall.

The amount of movement that is acceptable will depend upon the structures, utilities and infrastructure that are located within the vicinity of the new retaining structure and their tolerance to displacement.

Figure 63.7 shows the general type and form of movement that can be expected to occur.

Movements can be generally split into the following types:

- *Installation of the wall*: These could be due to a relaxation of the ground during the formation of piles or panels of an embedded retaining wall. Compression of the ground may occur, resulting in heave where sheet piles are driven into the ground. For embedded retaining walls, ground movements may occur due to the construction of the excavation because of the creation of the space where the gravity structure will be located or from settlements from loading the ground by the gravity structure.
- *Excavation related*: The excavation in front of a retaining wall will result in the retaining wall displacing laterally. This will lead to horizontal displacements and settlements behind the wall; see **Figures 63.8** and **63.9**.
- *Heave*: The removal of soil on the passive side of the wall will result in an unloading of the ground. This will lead to heave of the ground both on the passive and active sides of the retaining wall; see **Figure 63.8**.
- *Long term*: In fine-grained soils, the effects described above will result in long-term displacements arising from equalisation of the pore water pressures in the ground; see **Figure 63.8**.

It should be appreciated that during construction or installation of the wall, ground movements can also occur. These in general tend to be localised and aggravated by construction problems, for example, unacceptable vibration or overbreak. The removal of obstructions and the excavation of guide trenches before wall construction may also cause as much or more movement as wall installation itself. Further details of issues relating to construction may be found in Chapter 85 *Embedded walls*.

There are various techniques that can be used to assess the magnitude of ground deformation. The techniques broadly split into the categories of computational-based methods and physical models. For conventional design, computational

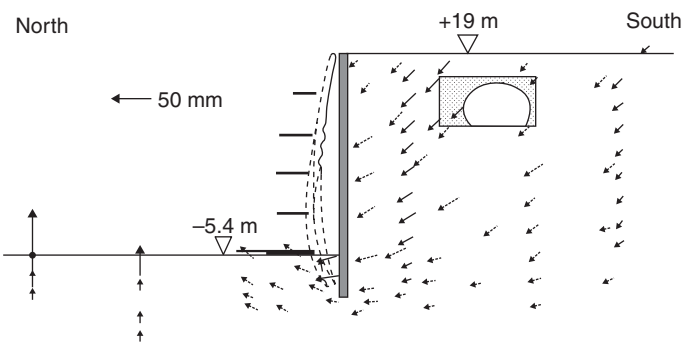


Figure 63.7 Typical ground movements around a deep excavation
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methods are generally used. Further details of these techniques are given in section 63.4.2.

Many of the methods described in this section consider only plane strain displacements (i.e. a 2D assumption). It should be realised that where retaining walls are used to form shafts or basements, the displacements will be significantly modified by the increased stiffness offered by the presence of corners around excavations. A basic consideration of how to account for these effects is given in CIRIA C580 (Gaba *et al*., 2003).

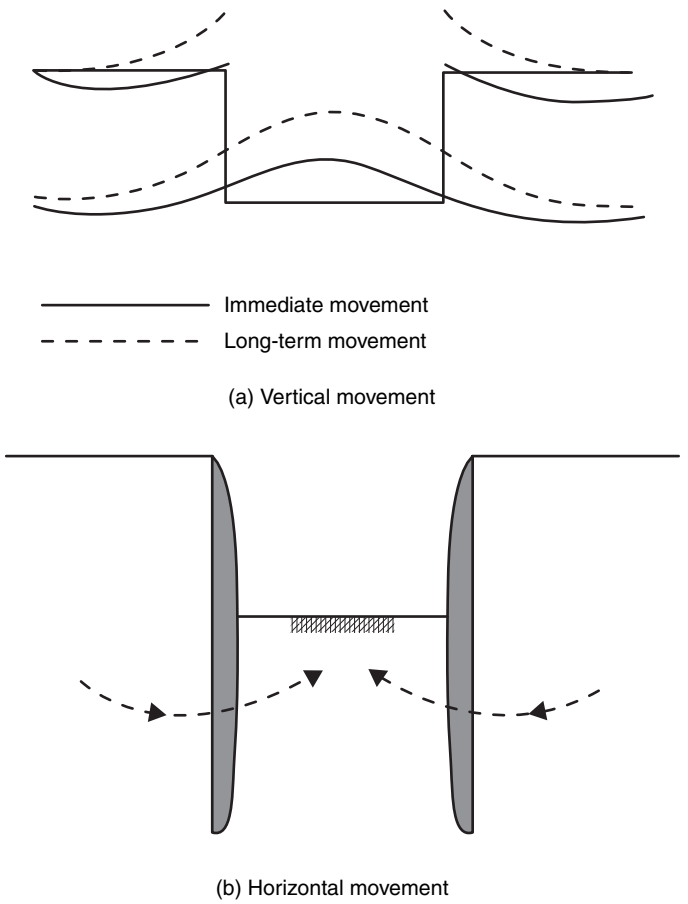


Figure 63.8 Typical ground movement pattern associated with excavation stress relief
Reproduced with permission from CIRIA C508, Gaba *et al.* (2003), www.ciria.org

63.4.2 Calculation-based methods

63.4.2.1 Introduction

Calculation-based methods can be broadly split into empirical- and numerical-based methods.

If designing in accordance with Eurocodes, calculations are carried out using characteristic soil parameters selected by the designer. As discussed in section 63.3 above, any characteristic value should be a cautious estimate. As a consequence, calculation-based methods that involve the selection of characteristic soil parameters will also result in a conservative estimate of the magnitude of ground movements.

63.4.2.2 Empirical methods

Empirical methods rely on the collation of case history data to provide an estimation of ground movement. Numerous authors have provided empirical correlations for ground movements around retaining structures. In particular, the following references are often considered when calculating surface displacements: Clough and O'Rourke (1990), CIRIA C580 (Gaba *et al*., 2003), Moorman (2004) and Long (2001). Displacements are commonly calculated as a function of the retaining wall excavation depth and are grouped with respect to the stiffness of the excavation and ground conditions where they are carried out; see **Figure 63.10**.

CIRIA C580 (Gaba *et al*., 2003) also provides an empirical method for relating wall displacements from pseudo-finite element methods, for example, those calculated by programs such as FREW or WALLAP, to the ground surface displacement behind the retaining wall.

For use of these methods the case study database should be considered and compared with the particular conditions being analysed. Further assumption or correlations with case study movements are required if estimates of subsurface ground displacements are to be made without resorting to more rigorous methods of analysis.

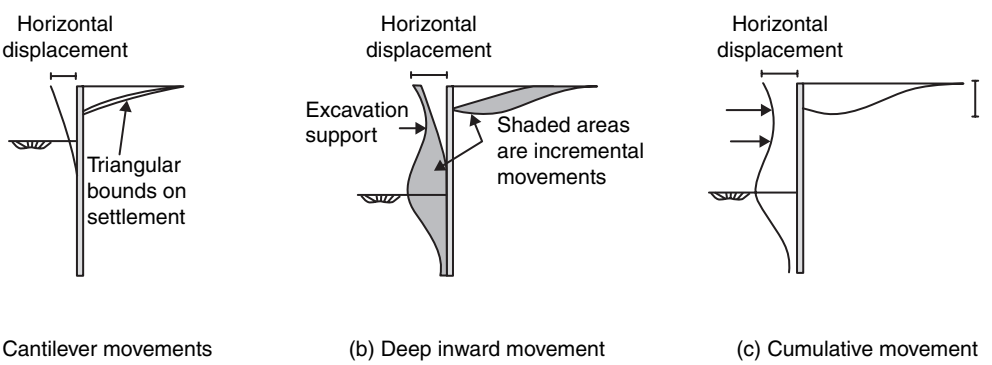


Figure 63.9 General pattern of wall movement and ground displacement
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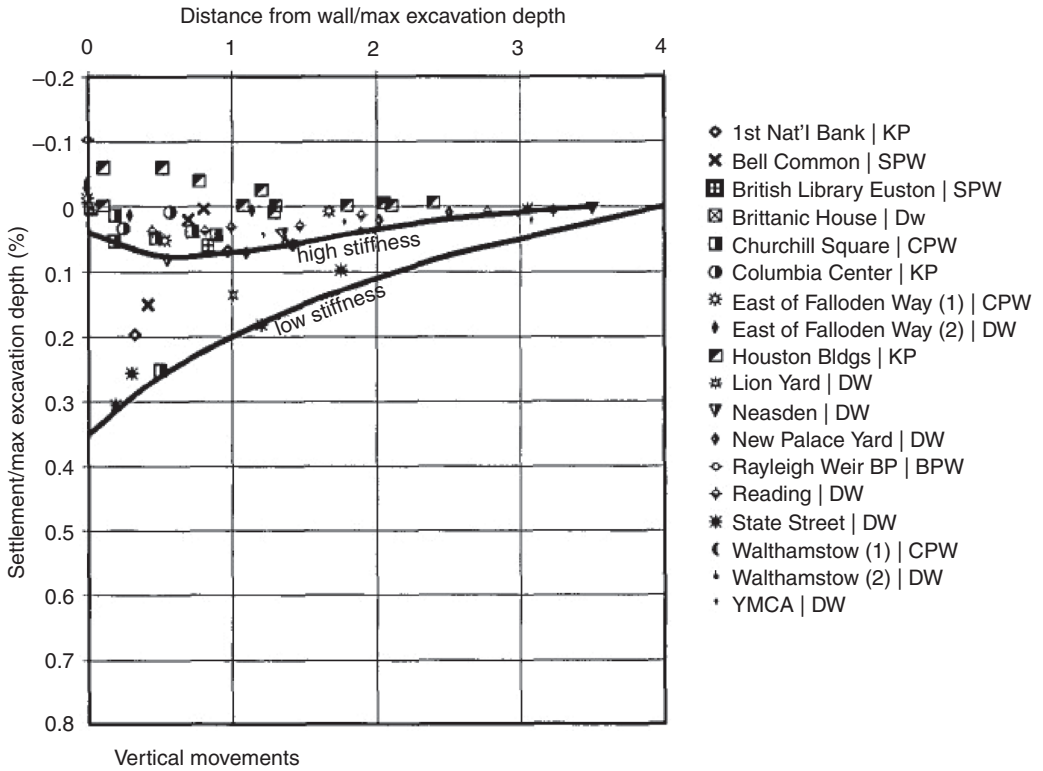


Figure 63.10 Ground surface movements due to excavation in front of wall in stiff clay
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63.4.2.3 Numerical methods

Numerical methods such as finite element or finite difference programs are commonly used for the design of retaining walls. The analysis methodology generally involves discretising the retaining wall and surrounding ground into elements within a mesh and specifying boundary conditions distant from the retaining wall. With the exception of discrete element analysis methods, the ground is modelled as a continuum. Further details on the use of numerical techniques may be found in Chapter 6 *Computer analysis principles in geotechnical engineering*. The primary reasons for using numerical methods include:

- They overcome the limitations and assumptions made by more simplistic analyses (for instance limit equilibrium or beam spring, pseudo-finite element programs).
- The use of numerical techniques can be readily extended to more complex problems (including berms).
- 3D analysis is available.
- The analysis can incorporate time-dependent consolidation.

Alongside this list of benefits, the following should also be borne in mind when considering numerical analysis:

- Application of the results of numerical analysis requires more knowledge and appropriate practical experience by the analyst.
- The active and passive limits are not easily computed.
- Data entry is more complex than other forms of computational analysis.
- Full numerical analyses carried out by a computer takes longer than limit equilibrium or pseudo-finite element of finite difference analyses.
- Interpretation of results takes longer and is often not as clearly understood.

63.5 Principles of building damage assessment

Further to carrying out an assessment of the magnitude of ground movements arising from retaining wall excavations it is important to assess whether the movements may have an impact upon adjacent structures and utilities. The principles of assessing the effect of ground movement on structures due to excavation is described in Chapter 26 *Building response to ground movements*.

It should be appreciated that lateral displacements and the resulting ground strains should also be considered as they may form a significant contribution to an affected structure's deformation. For many structures it should be appreciated that the interaction between the ground and structural deformation is complex. As a consequence either conservative assumptions or advanced forms of analysis may be required to adequately model this interaction.

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63.6.1 Further reading

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It is recommended this chapter is read in conjunction with

- Chapter 20 *Earth pressure theory*
- Chapter 64 *Geotechnical design of retaining walls*
- Chapter 85 *Embedded walls*
- Chapter 100 *Observational method*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

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Chapter 64

Geotechnical design of retaining walls

Adam Pickles Arup, London, UK

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This chapter outlines the design approach for retaining walls, following from the principles of retaining wall design outlined in Chapter 63 *Principles of retaining wall design*. The design basis is in accordance with BS EN 1997-1:2004, *Eurocode 7*. The mechanisms that typically require consideration in the design of gravity, reinforced soil and embedded retaining walls are identified and the design process for each type of wall is explained. Using the methods outlined in this chapter the reader will be able to size retaining walls to ensure adequate stability and calculate load effects for the structural design of the wall.

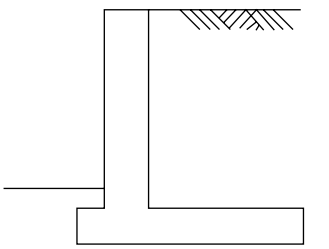
64.1 Introduction

This chapter outlines the design method for gravity retaining walls, embedded retaining walls and reinforced soil walls. The chapter develops the design concepts outlined in Chapter 63 *Principles of retaining wall design*, and identifies the limit states that should be typically considered when designing a retaining wall. The chapter describes how to apply the partial factors incorporated within BS EN 1997-1:2004 (British Standards Institute, 2004) to ensure a compliant design is achieved. Limit-equilibrium analyses that can be calculated by hand are described in addition to a design using more complex soil–structure interaction methods.

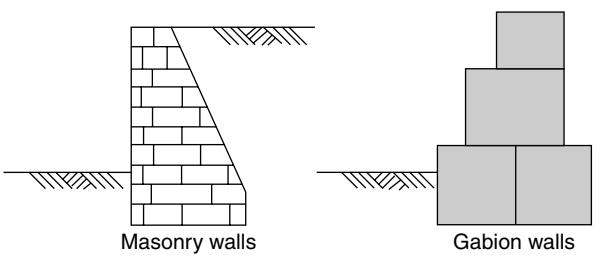
64.2 Gravity walls

64.2.1 Introduction

Gravity walls derive the majority of their stability from the self-weight of the wall, or soil placed on the toe or heel of the wall. This differs from an embedded wall, which derives significant support from the resistance of the ground in front of the wall for its stability. There are a variety of different types of gravity wall, as described in Chapter 62 *Types of retaining walls*, and the selection of a particular wall type will depend upon a variety of factors, as discussed in that chapter. There are broadly two types of gravity wall, one in which internal stability is provided by shear or bending in the structure, e.g. reinforced concrete stems, and the second type where gravity action and the self-weight of the wall provide stability, without mobilising tension in the structure, e.g. mass concrete or gabion walls. These are illustrated in **Figure 64.1**.



a) Internal stability maintained by structural stregth of reinforced concrete stem



b) Internal stability maintained by gravity action - no (or minimal) development of tension

Figure 64.1 Types of gravity wall: (a) internal stability maintained by structural strength of reinforced concrete stem wall; (b) internal stability maintained by gravity action – no (or minimal) development of tension

64.2.2 Limit states

BS EN 1997-1:2004 adopts the principle of a limit state design. The limit states relevant to gravity walls are discussed below. These are listed in Sections 9.2, 9.7 and 9.8 of BS EN 1997-1:2004.

64.2.2.1 Ultimate limit states (ULS)

BS EN 1997-1:2004 requires the following limit states to be considered for gravity retaining walls:

- bearing resistance failure of the soil below the base;
- failure by sliding at the base of the wall;
- failure by toppling of the wall;
- loss of overall stability;

- failure of a structural element such as a wall, anchor, wale or strut, or failure of the connection between such elements;
- combined failure of the ground and a structural element;
- movement of the retaining structure that may cause collapse or affect the appearance or efficient use of the structure, nearby structures or services that rely on it;
- unacceptable leakage through or beneath the wall;
- unacceptable transport of soil grains through or beneath the wall;
- unacceptable change to the flow of groundwater.

Some of these limit states are illustrated in **Figure 64.2**.

In addition to the limit states listed above, the designer should consider whether any additional limit states should be taken into account in respect of specific site conditions or other requirements not covered by this manual.

64.2.2.2 Serviceability limit states (SLS)

The following serviceability limit states should be considered:

- movement that may cause damage to structures or services, which rely on the support of the wall;
- excessive tilting, deflection or settlement of the wall. The acceptable values for these deformations should be assessed on a project-specific basis.

64.2.3 Factors of safety

The design approach outlined in this manual is based on BS EN 1997-1:2004 with partial factors being applied to characteristic values of material strengths, geometry and actions to give design values. This is a change from traditional practice for gravity-wall design where lumped factors of safety were typically applied to the different mechanisms analysed. In simple terms, the traditional methods used unfactored characteristic soil parameters and analyses were undertaken to assess factors of safety against overall stability, overturning about the toe of the wall, sliding along the base of the wall, and bearing capacity failure beneath the wall. Typical values of factors of safety adopted using such traditional design methods for designing permanent structures are given in **Table 64.1**.

Mechanism	Target factor of safety
Overall stability	1.3
Overturning	2.0
Sliding	1.5
Bearing capacity	3.0

Table 64.1 Traditional lumped factors of safety for gravity-wall design

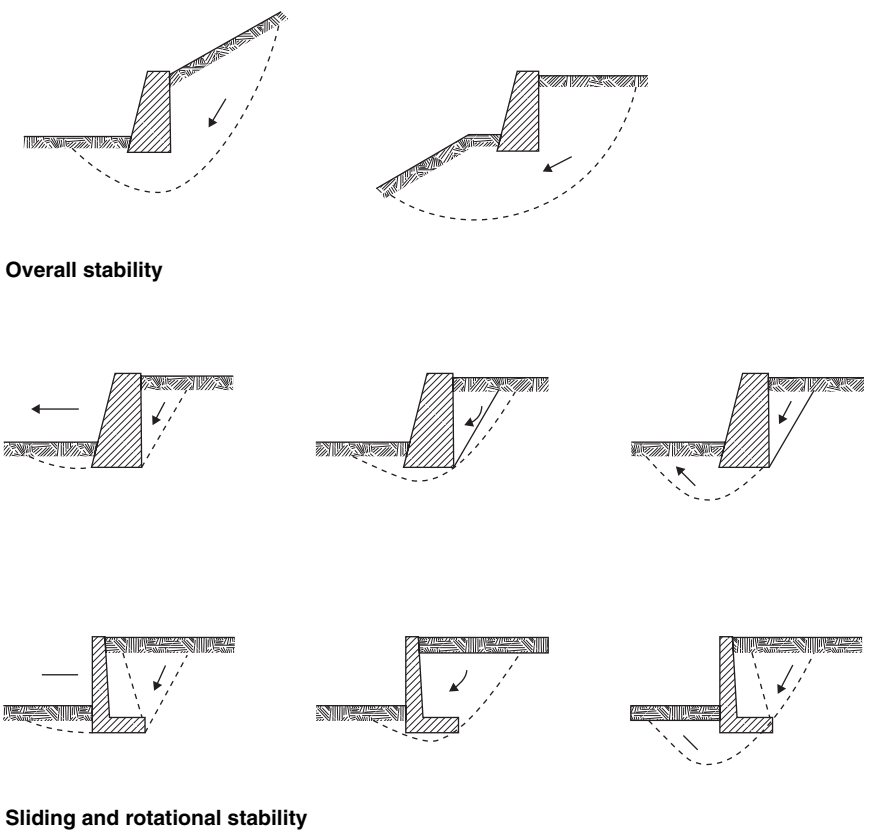


Figure 64.2 Gravity wall: ultimate limit states

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The partial factors required by BS EN 1997-1:2004 mean a different approach must be taken. Rather than seeking an overall lumped factor of safety, partial factors are applied to a number of different inputs, such as material strength, actions, wall geometry and water pressure. The size of the wall is then selected to ensure that the design actions, or action effects, are less than the design resistances for each of the limit states described above.

The design approach and values of partial factors to be used in design are stated in the relevant National Annex of BS EN 1997-1:2004. In the UK, Design Approach 1 is applicable for gravity walls. This approach comprises two combinations:

- Combination 1 (DA1-1): where the material partial factor is unity, and actions (or action effects) are factored by a value greater than 1;
- Combination 2 (DA1-2): where partial factors are applied to material strengths and variable actions, but the factor on the permanent actions (or action effects) is unity.

The reader should refer to the appropriate National Annex for confirmation of these partial factor values.

In addition it may be necessary to undertake an SLS analysis where all factors are unity. Traditionally, large lumped factors acted to reduce the proportion of strength mobilised, and hence control movement. The partial factors in BS EN 1997-1:2004, may lead to lower equivalent overall factors of safety and therefore there is potential for more movement. As a consequence an explicit check is needed for the SLS case.

64.2.4 Design situations, earth pressure, design geometry, groundwater conditions

64.2.4.1 Introduction

An illustration of the typical loads acting on a gravity retaining wall are summarised in **Figure 64.3**.

From a stability perspective, some actions tend to reduce the stability of the structure, whilst others tend to improve stability. The objective is to ensure that the design resistance exceeds the design action effects. The following considerations should be addressed:

- For *bearing capacity*: sum the vertical actions, horizontal actions and moments about the centre of the base, then undertake a check as a shallow footing – the design approach described in Chapter 63 *Principles of retaining wall design* should be followed.
- For *sliding*: calculate the net horizontal load and check that this is smaller than the sliding resistance. For free-draining granular soils, this is a function of the vertical load.

The traditional check on *overturning* is not generally necessary. If the wall was on the limit of toppling (or even past the limit of toppling), then the effective footing width from the bearing capacity calculation above would be very small or the line of action would fall outside the base of the wall. This would therefore be picked up as a bearing capacity failure. The only case where this may not be correct is for walls bearing on strong rock. In such cases the bearing capacity may be sufficient to resist very high load eccentricities and so overturning stability must be checked explicitly.

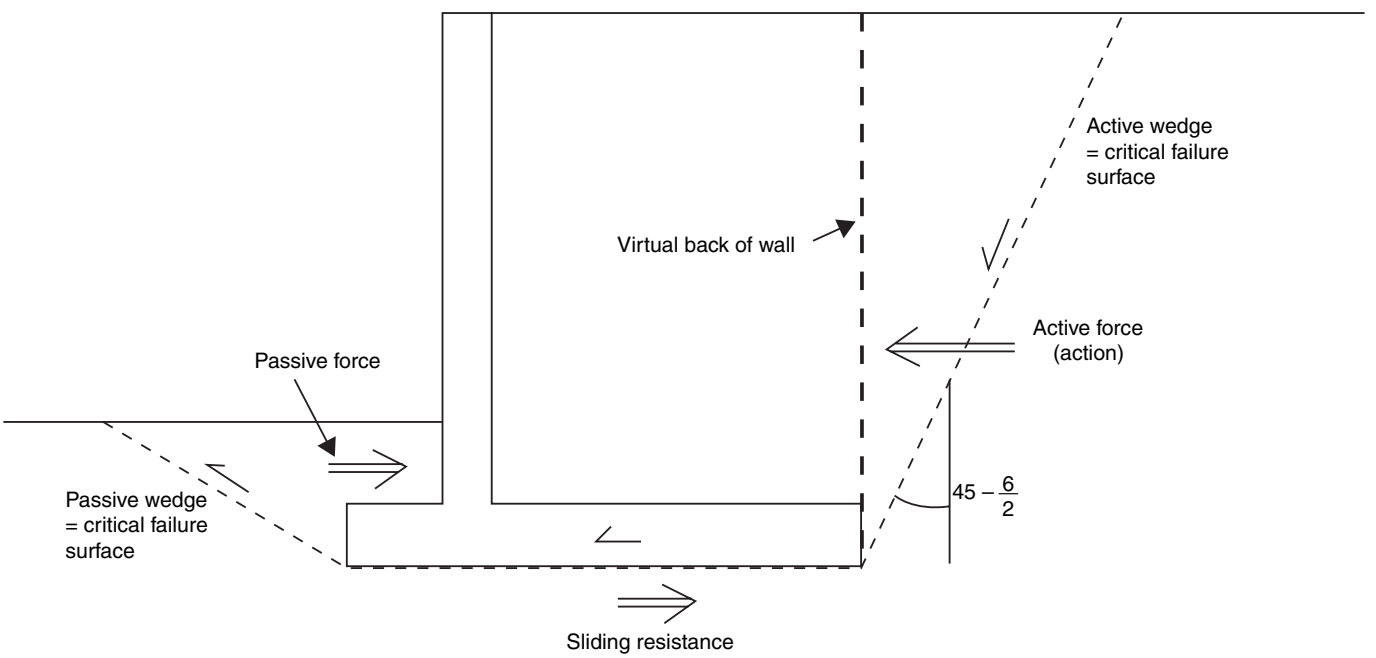


Figure 64.3 Idealised actions on a gravity wall

The sections below provide guidance on calculating the individual components of the load acting on a gravity wall.

64.2.4.2 Design situations

Before any analysis is undertaken, it is necessary to establish the appropriate design situations. Issues to consider include:

- *Design soil stratigraphy*: In variable ground conditions it may be necessary to consider more than one design soil stratigraphy. Each one would comprise a separate design situation.
- *Soil parameters*: Characteristic values should be assessed based on the approach outlined in Chapter 27 *Geotechnical parameters and safety factors*.
- *Time effects*: For example, whether the analysis will be short term (i.e. undrained parameters) or long term (i.e. drained parameters). If necessary both conditions may need to be checked as separate design situations.
- *Construction sequence*: It is necessary to ensure that all potentially critical stages are considered in the design.
- *Design geometry*: Where this varies for the structure, for example, because of changes in the overall retained height, consider how the wall will be divided up for design purposes and identify the critical section for each. Each geometry will then be considered as a separate design situation.
- *Imposed loading*: Are there any externally applied loads, such as traffic surcharges, impact loads or loads from adjacent structures that need to be considered in the analysis?

Whilst it may be necessary to calculate more than one design situation, by careful consideration of the relevant combinations that are likely to prove critical, it is possible that once a solution for the critical design situation has been determined, alternative situations may be confirmed as acceptable either by inspection, or through a reduced number of analyses and calculations.

64.2.4.3 Earth pressure

The principal actions on a gravity retaining wall are usually earth and water pressures. The earth pressure is derived from the self-weight of the soil and possibly from loads applied to the ground surface. In addition there may also be some direct loading of the wall, for example, from structures bearing onto the wall or from impact loads applied to parapets connected to the wall.

Typically, a gravity-wall analysis is carried out by assuming the limit-equilibrium earth pressure acts on the structure. It is relatively unusual for a gravity wall to be designed using a soil–structure interaction analysis. Whilst the general design principles outlined in this chapter may be applied to a soil–structure interaction analysis of a gravity wall, this section will primarily focus on the limit-equilibrium approach.

The first step in identifying the earth pressure acting on a wall is to select a virtual back (see **Figure 64.3**). Soil within this line is considered to act with the structural components of the wall as a rigid body. Normal stresses and shear stresses must then be evaluated along the virtual back. For many situations the virtual back is taken as a vertical line upwards from the heel of the wall. In this case it is assumed that a Rankine active wedge forms both within the soil mass over the heel of the wall and in the retained ground behind the virtual back. This means the force acting normal to the virtual back can be calculated using Rankine earth-pressure theory. Because the soil within the active wedge over the heel of the wall and in the retained fill are both moving downwards together, zero shear stress is mobilised along the virtual back (see **Figure 64.4**).

An alternative approach is to assume the virtual back lies on a plane joining the heel of the wall to the crest of the wall (see **Figure 64.5**).

For either of the above two cases, the active forces acting on the virtual back should then be evaluated for the chosen geometry, using either tabulated values of earth-pressure coefficients (e.g. Kerisel and Absi, 1990), the charts in BS EN 1997-1:2004 or the analytical method of calculating earth-pressure coefficients given in BS EN 1997-1:2004. The *design* material strength should be used in this calculation.

It should be noted that since DA1-1 and DA1-2 in BS EN 1997-1:2004 give different values for the partial factor on the material strength then the earth-pressure coefficients and hence the limiting earth pressures used for each design approach will also be different.

Where the base of the wall is founded below the ground level in front of the wall, passive earth pressures may be considered to act on the front of the wall. Careful consideration needs to be given as to the reliability of this pressure being mobilised. In order to form the wall base, the material in front of the wall is likely to have been excavated and then backfilled once the wall has been constructed. The quality of this material, and the level of compaction used to replace the material

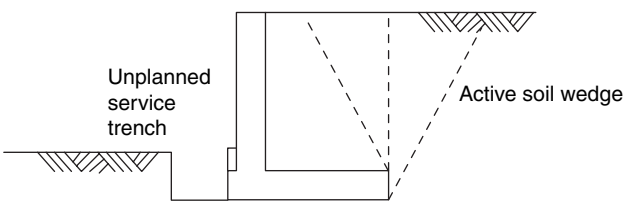


Figure 64.4 Rankine active wedge

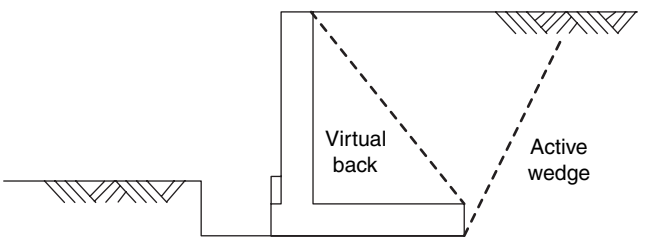


Figure 64.5 Inclined virtual back

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may therefore influence the magnitude of passive resistance that can be mobilised and relied upon. The passive resistance may also be affected by unplanned excavation (see section 64.2.4.4 below). If there is any uncertainty regarding the reliability of the passive resistance, it is recommended that it be omitted from the analysis.

Where sliding is shown to be a critical factor in design, the stability of the wall may be improved by the construction of a shear key or the use of an inclined base. Further details regarding the inclusion of the effects of shear keys in a gravity-wall analysis can be found in *CIRIA C516* (Chapman *et al*., 2000).

As required by BS EN 1997-1:2004, it is important to ensure that the assumed earth pressures for a given limit state are compatible with the anticipated movements at that limit state. For geotechnical stability (i.e. sliding, toppling, bearing) it is reasonable to assume sufficient movement will occur to mobilise active earth pressures. However, for the structural design of the wall it is possible that the wall deformation under *in situ* conditions may be insufficient to mobilise active pressures. For structural design it will therefore often be appropriate to use the 'at-rest' earth pressures. BS EN 1997-1:2004 includes information on the magnitudes of any movement that are required to mobilise active and passive earth pressures and these are also discussed in Chapter 63 *Principles of retaining wall design*. The enhanced *in situ* pressures may also be applicable to the SLS case.

Since gravity walls typically retain placed fill, the *in situ* pressures must take into account the construction processes that formed the backfill. Typically some form of compaction will be used behind the wall to achieve the required backfill strength and to limit the potential for future settlement of the backfill. Compaction tends to lock in stresses in the ground, which can exceed the values calculated based on a typical *in situ* earth-pressure coefficient, K_0. The horizontal pressure acting along the back of the wall due to compaction is given in **Figure 64.6**.

The values of σ_{hrm}, h_c and z_{cr} can be determined as follows:

$$h_c = \frac{1}{K_0}\sqrt{\frac{2P}{\pi\gamma}} \qquad (64.1)$$

where P is the effective line load per metre of the roller, and γ is the density of the material undergoing compaction.

The maximum pressure σ'_{hrm} can be calculated from

$$\sigma'_{hrm} = \sqrt{\frac{2P\gamma}{\pi}} \qquad (64.2)$$

and this occurs below a depth of z_{cr} where

$$Z_{cr} = K_0\sqrt{\frac{2P}{\pi\gamma}}\ . \qquad (64.3)$$

64.2.4.4 Unplanned excavation

When assessing the geometry for a given design situation for a wall where the passive resistance of the ground is relied upon for stability, it is important to take into account the potential for unplanned excavation in front of the wall. BS EN 1997-1:2004 requires that for normal site controls, 10% of the retained height, up to a maximum of 0.5 m, should be assumed for an unplanned excavation.

64.2.4.5 Surcharges and direct loads

The design of a retaining wall must take into account that a surcharge may be applied to the ground retained by the wall. Such a surcharge may be derived, for example, from highway traffic in the permanent condition, plant loading during the construction phase or from the placement of stockpiled materials behind the wall. The effect of such surcharges must be incorporated into the assessment of the earth pressure acting on the wall.

BS 8002 (British Standards Institution, 1994b) has a requirement to consider a minimum 10 kPa surcharge behind a wall. Although this is not an explicit requirement in BS EN 1997-1:2004, for most walls some allowance should be made and a 10 kPa uniform distributed load is considered reasonable.

Typically such surcharges would be considered as a variable action. In these cases consideration should be given to whether the effect is beneficial to the stability of the wall. In particular if a live

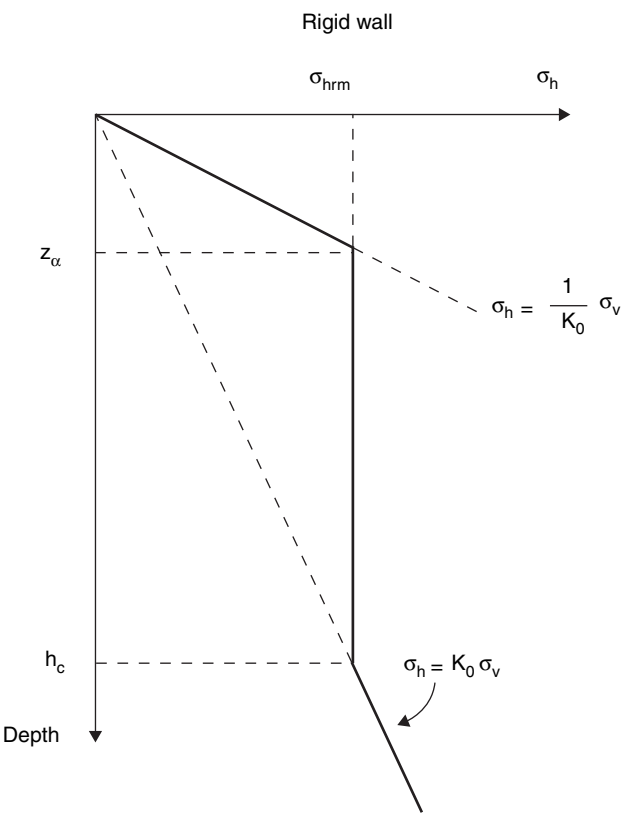


Figure 64.6 Calculation of compaction pressures

load surcharge acts on the soil directly over the heel of the wall, this would contribute additional load to resist overturning and sliding (for free-draining granular materials), which cannot be relied upon. However, for bearing capacity checks and the structural design of the wall, the additional vertical load from the surcharge over the heel may prove to be adverse. As a rule, live load surcharges are typically only applied to the ground outside the virtual back. In this way the destabilising effect of the load is considered, but no beneficial effects are relied upon; see **Figure 64.7**.

64.2.4.6 Water pressure

As is commonly the case for geotechnical design, water pressure can be a significant component of load on a gravity wall. Since gravity walls retain fill material, there is potential to install a drainage system behind the wall and hence control the water pressure. However, such a system would require ongoing maintenance to ensure that it continues to perform.

In the absence of a drainage system, BS EN 1997-1:2004 requires that the water level is assumed to be at the surface of the retained ground level for low permeability backfills. Consideration also needs to be given to the occurrence of seepage around a wall. In the absence of seepage analysis it may be appropriate to assume a linear variation in pore pressure across the base, from the pressure behind the wall to the pressure in front of the wall.

For the design groundwater conditions, the following should be determined:

1. Water pressure and seepage forces that represent the most unfavourable values that could occur in extreme or accidental circumstances at each stage of the wall's construction sequence and throughout its design life. An example of an extreme or accidental event may be a burst water main in close proximity to the wall.
2. Water pressures and seepage forces that represent the most unfavourable values likely to occur in normal circumstances at each stage of the wall's construction sequence and throughout its design life. Extreme events such as a nearby burst water main may be excluded, unless the designer considers that such an event may reasonably occur in normal circumstances.

Different approaches to groundwater are required for DA1-1 and DA1-2. In DA1-1 the actions of the water are factored by a value greater than unity, therefore it is recommended that water condition (2) is assumed, as the factor on the action of the water will give a margin of safety. For DA1-2, the design actions are derived from design material parameters, with a factor on permanent actions of unity. It is therefore recommended that the design water pressure is derived from a directly assessed design groundwater level, which is defined as (64.1) above.

64.2.4.7 Drainage systems and fill materials

There is a need to have consistency between the design groundwater assumptions and the specification of backfill or drainage. It is becoming more common to use site-won material. This may have lower strength and lower permeability than traditional backfill material so the designer needs to know in advance what is available. Without drainage, an assumption of hydrostatic conditions from the ground level is required by BS EN 1997-1:2004 for medium- and low-permeability soils.

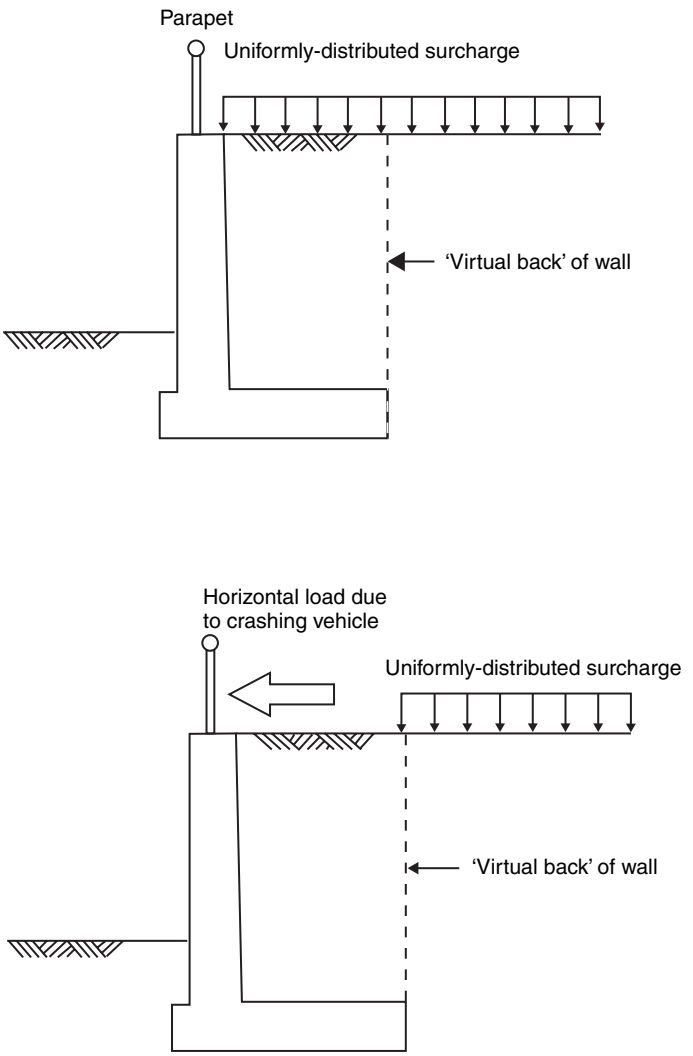


Figure 64.7 Surcharge assumptions for different design cases. (a) Load case 1: often critical for bearing pressure calculations and design for internal stability. (b) Load case 2: often critical for sliding stability and bearing pressure calculations

64.2.5 Design of structural elements

As noted above, it is necessary to ensure compatibility between the movement of the wall at a given limit state and the value of the earth pressure. This means that the magnitude of the pressure for structural strength design may be higher than for geotechnical stability. For reinforced-concrete walls, the load effects derived from the analysis are design effects to be used in the structural calculation.

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For mass gravity-type walls, e.g. mass concrete or gabion walls, it will be necessary to undertake checks at different points down the height of the wall to ensure that tension is not generated where it cannot be resisted, and that the shear forces do not exceed the sliding resistance, for example, along the boundary between two layers of gabion baskets.

64.2.6 Design method

The preceding sections outline how the various loads acting on a wall can be assessed. Once these actions have been identified, it is necessary to analyse the wall to verify that the limit states identified in section 64.2.2 above will be avoided. Typically this analysis is iterative, where an initial estimate of the wall geometry is made and stability against each failure mode is checked. The geometry of the wall may then be adjusted to optimise the solution while still ensuring stability. Such analysis and iteration may be calculated by hand, or calculated using proprietary software or a spreadsheet.

The design stages for a gravity wall are shown in the flowchart in **Figure 64.8** and described below.

(1) Confirm the relevant limit states for the problem, as identified in section 64.2.2. For serviceability limit states agree the deflection limits or other criteria that are to be adopted.

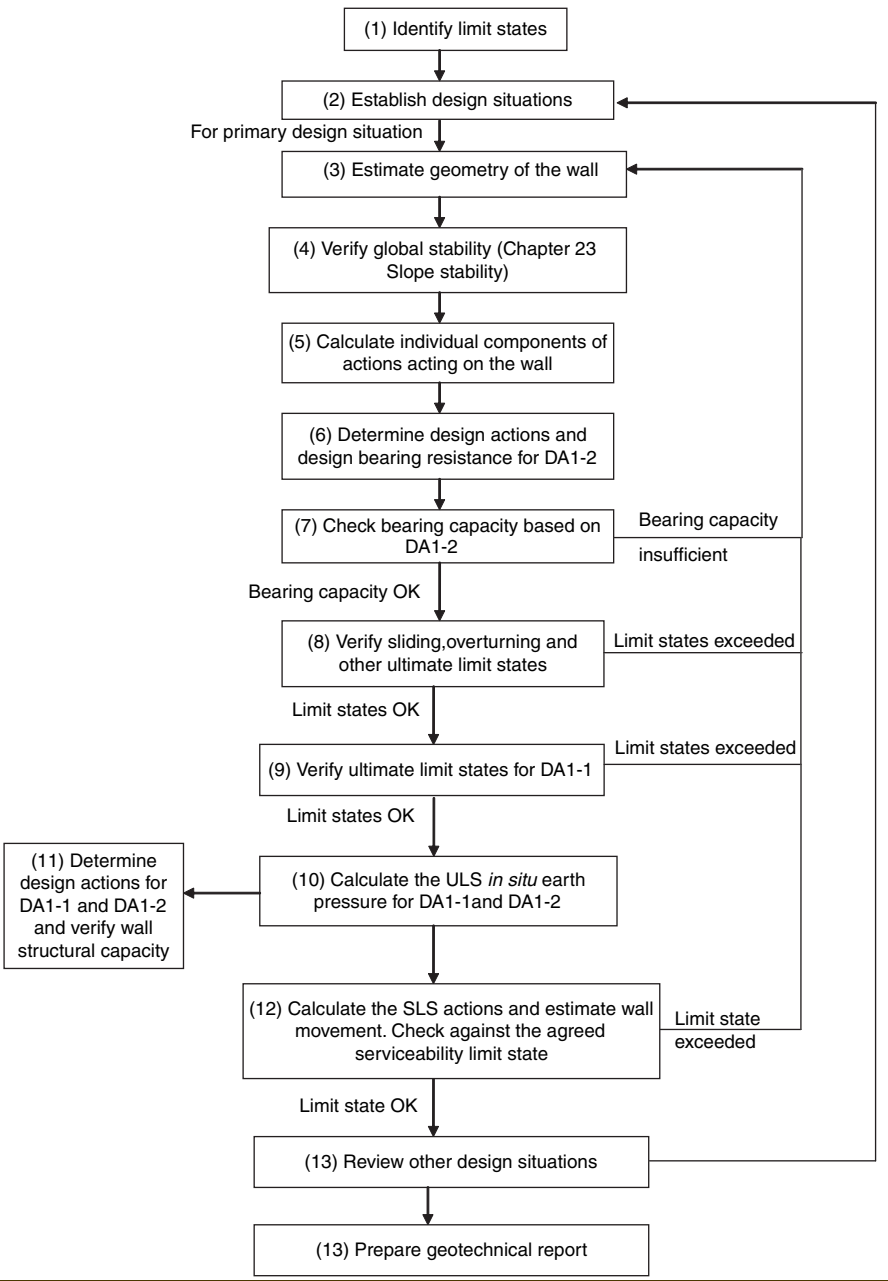


Figure 64.8 Gravity-wall design process

(2) Identify the design situations to be considered in the analyses, considering the issues described in section 64.4. Select an initial situation to analyse.

(3) For the selected design situation, estimate the initial geometry of the wall.

(4) Verify the global stability of the structure, in accordance with Chapter 23 *Slope stability.*

(5) Calculate the individual components of the actions acting on the wall for Design Approach 1 Combination 1 and 2, using the guidance of section 64.4.

(6) For Design Approach 1 Combination 2, apply the relevant partial factors to the characteristic actions to determine the design actions. On the basis of these design actions determine the design bearing resistance of the ground.

(7) Check whether the design actions exceed the design bearing resistance. If they do, return to step 3 and revise the wall geometry.

(8) Once sufficient bearing resistance has been demonstrated, verify that the sliding and overturning limit states are also acceptable. If not, return to step 3.

(9) Verify the geotechnical ultimate limit states for Design Approach 1 Combination 1. Return to step 3 if insufficient capacity is identified.

(10) Calculate the ULS *in situ* earth pressure acting on the wall, including the compaction pressure where appropriate, for DA1-1 and DA1-2.

(11) Determine the design actions on the wall for the structural design for DA1-1 and DA1-2. Verify the structural capacity of the wall is acceptable.

(12) Calculate the SLS *in situ* earth pressure acting on the wall, including the compaction pressure, where appropriate. Estimate the anticipated movement of the wall and check against the agreed serviceability limit state. If acceptable, the design situation has been verified as acceptable, otherwise return to step 3.

(13) Review other design situations: start at step 2 for each design situation.

(14) Prepare the geotechnical report, including details of maintenance requirements associated with any drainage systems included in the design.

When carrying out the different ULS checks, it is important to consider whether the actions are beneficial as this will influence the partial factor that is applied.

The procedure for verifying the bearing resistance should be undertaken in accordance with Chapter 53 *Shallow foundations*.

64.3 Reinforced soil walls

64.3.1 Introduction

Reinforced soil walls are essentially a specific type of gravity wall. Two different checks need to be made, one for external stability and the other for internal stability. The size of the block of reinforced soil is selected to provide overall stability, with the internal check being made to ensure that the reinforcement holds the block together. The external stability check should follow the procedure set out in section 64.2 above.

A discussion of the available types of soil reinforcement and a detailed explanation of the design of gravity walls is given in Chapter 73 *Design of soil reinforced slopes and structures* with a brief overview below.

It is noted that BS EN 1997-1:2004 is not applicable to reinforced soil walls and their design should be undertaken in accordance with the current British Standard BS 8006 (British Standards Institution, 1994a).

64.3.2 Design for external stability

As with any gravity wall, the overall block of reinforced soil must be selected to ensure the geotechnical limit states are met.

Slope stability, bearing capacity, sliding and overturning limit states must be checked.

For many projects it is common to design the soil block for external stability, which is typically presented as a minimum length of reinforcement.

64.3.3 Design for internal stability

Once the overall size of the reinforced soil block has been selected, the reinforcement must be designed. This typically includes parameters such as the gauge of reinforcement and the spacing of layers. On occasion it may also be necessary for the straps to be longer than the minimum required for external stability to ensure that pull-out failure is avoided.

There are a number of different proprietary soil reinforcement systems available, and whilst they generally work on the same principles, the detail of the design will vary. In general the design for internal stability must ensure that a soil block remains intact. Typically this requires checking the pull-out resistance and whether the reinforcement ruptures, which are described in Chapter 73 *Design of soil reinforced slopes and structures*.

Since the various systems differ, it is common for a specialist reinforced soil sub-contractor to undertake the detailed design.

64.4 Embedded walls

64.4.1 Introduction

As in Chapter 62 *Types of retaining walls*, an embedded retaining wall penetrates into the ground at its base to gain lateral support from the resistance of the ground in front of the lower part of the wall. This resistance may be used as the sole resistance to provide stability for the wall or it may act in conjunction with

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anchorages, props or other sources of lateral support for stability. The structural strength of the wall in bending and shear is usually critical.

Unlike gravity walls, which support backfilled material, embedded walls will usually be installed into natural ground, which is then excavated in front of the wall. This means there is no control over the quality of the ground that the wall is supporting nor is there an opportunity to install drainage.

In general, embedded walls can be categorised as cantilever or propped. Whilst the overall principles for design are comparable between these two categories, there are differences in the specifics of the design approach and these are identified in the following sections.

64.4.2 Limit states

As with gravity walls, BS EN 1997-1:2004 adopts the principle of a limit state design. The limit states relevant to embedded walls are discussed below. These are listed in Sections 9.2, 9.7 and 9.8 of BS EN 1997-1:2004.

64.4.2.1 Ultimate limit states (ULS)

BS EN 1997-1:2004 lists the following ultimate limit states as relevant to embedded walls:

- loss of overall stability;
- failure by rotation or translation of the wall or parts thereof;
- failure of a structural element such as a wall, anchorage, wale or strut or failure of the connection between such elements;
- failure by lack of vertical equilibrium;
- combined failure in the ground and in the structural element;
- failure by hydraulic heave and piping;
- movement of the retaining structure, which may cause collapse or affect the appearance or efficient use of the structure or nearby structures or services, which rely on it;
- unacceptable leakage through or beneath the wall;
- unacceptable transport of soil particles through or beneath the wall;
- unacceptable change in the groundwater regime.

Some of these limit states are illustrated in **Figure 64.9**.

The design of an embedded wall must ensure that these limit states are avoided.

64.4.2.2 Serviceability limit states (SLS)

The primary serviceability limit state noted by BS EN 1997-1: 2004 for embedded walls is movement of the retaining structure, which may cause collapse or affect the appearance or efficient use of the structure or nearby structures or services, which rely on it. This may involve both lateral and vertical movement of either the wall or the soil around the wall.

64.4.3 Factors of safety – partial factors and lumped factors

As discussed in section 64.2.3 above, BS EN 1997-1:2004 adopts a partial factor approach to design. For embedded walls this concept has been in common use for a number of years, and should be readily adopted in design. There are, however, a number of traditional methods of embedded wall design that use a lumped factor approach. Typically these either sought a sufficiently large ratio of the restoring moment to the overturning moment in a rotational stability check, or applied a single factor to a measure of the passive resistance of the soil. *CIRIA C580* (Gaba *et al*., 2003) provides a detailed discussion of these approaches.

The ultimate limit states identified above typically relate to either the BS EN 1997-1:2004 GEO limit state ('failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance') or STR ('internal failure or excessive deformation of the structure or structural elements in which the strength of structural materials is significant in providing resistance'). Partial factors to be used in the design of embedded walls are provided in BS EN 1997-1:2004 for these limit states. Design Approach 1 has been adopted in the UK: walls must be checked against both Combination 1 and Combination 2.

In Combination 2, factors greater than 1 are applied to the characteristic soil parameters to derive the design values to be used in the analysis. This is usually the critical case for stability and will establish the required wall geometry. The load effects in the wall from this analysis are used to check the structural capacity of the system, along with the output from Combination 1.

For the serviceability limit state check, partial factors of unity are used.

64.4.4 Design situations, design geometry, groundwater conditions

64.4.4.1 Introduction

The general aim of an embedded wall design is to select a wall length that is sufficient to ensure stability and to derive load effects so that the adequate structural performance of the wall and any support system can be ensured. In addition it is common that some form of movement analysis is undertaken to ensure that the structure meets the design performance requirements and to ensure nearby structures and utilities are not adversely affected.

The following sections outline the key considerations that the designer should address in designing an embedded retaining wall.

64.4.4.2 Design situations

Design situations are discussed in BS EN 1997-1:2004. When designing an embedded wall it is important to consider whether more than one design situation is applicable. For example,

where construction is staged, such as for multiple-propped walls, it is necessary to consider each stage of the construction sequence individually to ensure that the relevant limit states are avoided.

For each design situation, the designer must consider the following:

- *Design soil stratigraphy*: In variable ground conditions it may be necessary to consider more than one design soil stratigraphy. Each would comprise a separate design situation.
- *Soil parameters and ground model*: Characteristic soil parameters should be assessed based on the approach outlined in Chapter 27 *Geotechnical parameters and safety factors*. Walls are linear structures and therefore have the potential to encounter variations in the design stratigraphy and soil parameters. The designer must consider this variability and if necessary analyse the different ground models.
- *Time effects*: For example, whether the analysis will be short term (i.e. undrained parameters) or long term (i.e. drained parameters). If necessary both conditions may need to be checked as separate design situations.
- *Construction sequence*: It is necessary to ensure that all potentially critical stages are considered in the design.
- *Design geometry*: Where ground levels or retained heights vary along the length of the wall, it may be appropriate to divide the

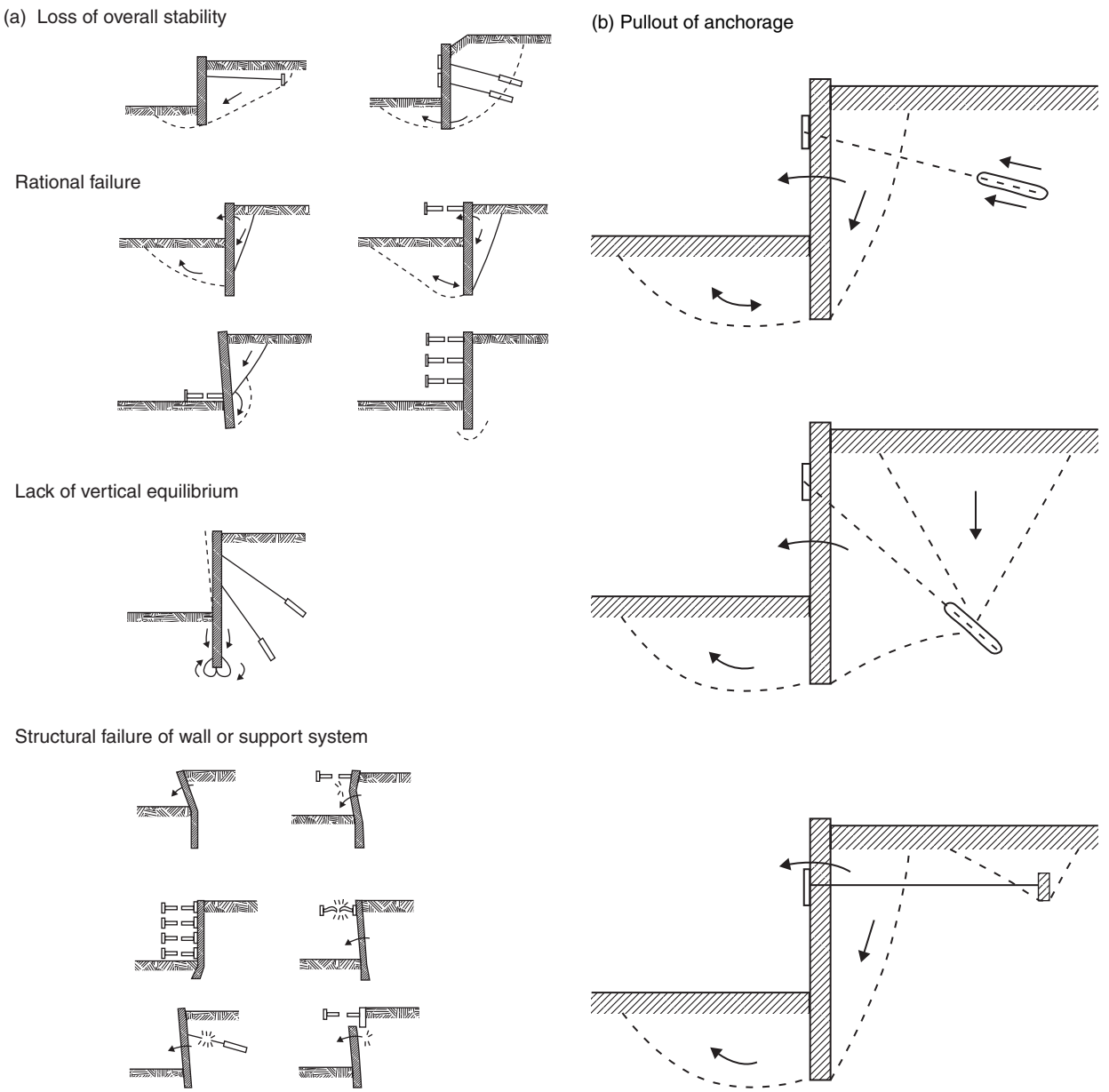


Figure 64.9 Ultimate limit state failure modes for embedded walls

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wall into sections and select a representative geometry for each section. Each geometry will then be considered as a separate design situation.

- *Groundwater and drainage*: This is a key consideration when developing the design situations for a retaining wall. Depending upon the soil type, the construction sequence and the rate of construction, it may be necessary to analyse situations which consider that the ground is undrained in some cases and drained in others. This is discussed in detail in Chapter 63 *Principles of retaining wall design*.
- *Imposed loading*: Consider surcharges due to building loads or traffic and their effect.

Whilst it may be necessary to calculate for more than one design situation, by careful consideration of the relevant combinations that are likely to prove critical, it is possible that once a solution for the critical design situation has been determined, alternative situations may be confirmed as acceptable either by inspection, or through a reduced number of analyses and calculations.

64.4.5 Earth pressure

In order to verify the stability and movement of a wall, the earth pressure acting upon it must be calculated. There are two general analysis methods that are typically used to evaluate this pressure:

- limit equilibrium;
- soil–structure interaction.

Limit-equilibrium methods of calculation are based on conditions at collapse, when the full strength of the soil is mobilised around the retaining wall based on assumed failure mechanisms. Calculations are typically based on simple assumed linear lateral stress distributions, which represent a particular collapse mechanism. Collapse mechanisms are well developed for cantilever and single-propped walls, but less so for other forms of embedded retaining walls, such as multi-propped walls and walls propped significantly below the top. Limit-equilibrium analyses may be calculated using computer software or by hand.

Soil–structure interaction analyses require the use of specialist software. In such analyses the initial stresses in the ground are established, and the sequence of construction is modelled. At each stage, changes are made to the model to disrupt the equilibrium of the system, for example soil is excavated or a support removed, and the calculation models the deformation of the system and the mobilisation of stresses in the ground and the structure as the analysis attempts to restore equilibrium. These analyses therefore directly calculate the earth pressure that is generated around the wall and the associated wall and soil movement.

A discussion of the relative merits of limit-equilibrium and soil–structure interaction analyses are given in *CIRIA C580* (Gaba *et al.*, 2003).

64.4.5.1 Limit-equilibrium analysis

For a limit-equilibrium analysis, earth-pressure profiles are based on established mechanisms, and the selection of the appropriate earth-pressure distribution depends upon the design situation. The established mechanisms for two common situations: cantilever and propped walls are illustrated in **Figure 64.10**.

For the fixed-earth mechanism the wall is assumed to rotate about some pivot point in the ground. Above this point, the earth pressure on the retained side is assumed to be at the active limit and the earth pressure on the excavated side is assumed to be at the passive limit. To ensure rotational and lateral stability can be achieved, below the pivot point this is reversed so that

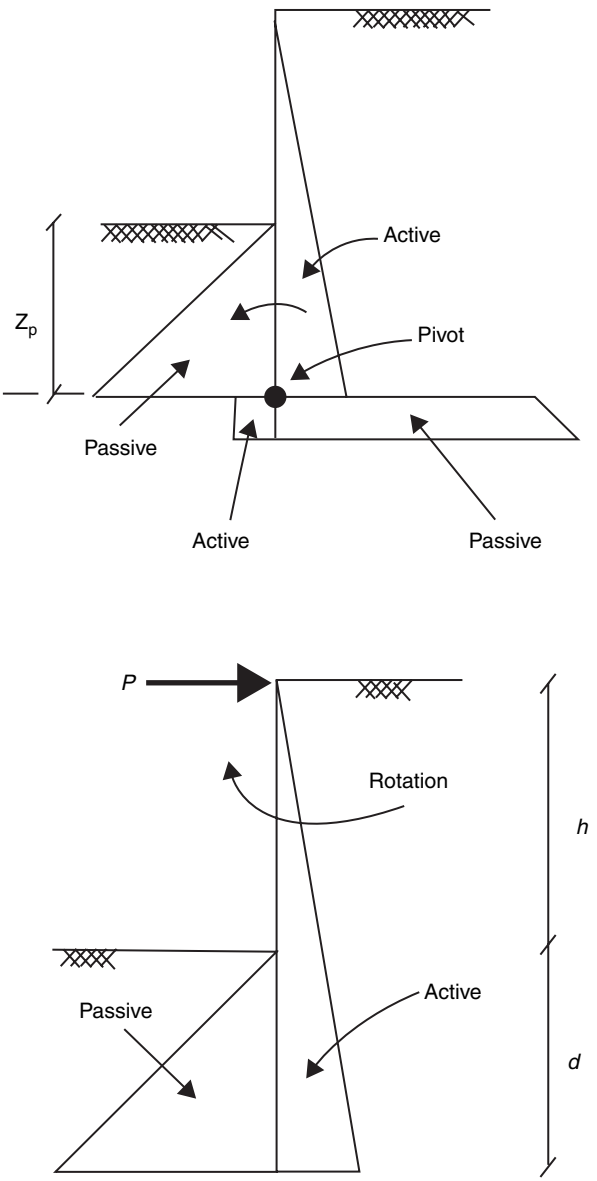


Figure 64.10 Limit-equilibrium earth-pressure distributions. (a) Fixed earth – cantilever. (b) Free earth – propped wall

on the retained side the pressure is at the passive limit and the pressure on the excavated side is at the active limit. The stability of the wall is verified by considering the moment equilibrium about the assumed pivot point, and the equilibrium of the assumed horizontal forces. To achieve equilibrium, an iterative calculation is required, where the toe and pivot points are varied until equilibrium is achieved. This process is best performed by specialist wall analysis software or a spreadsheet.

A simplification of the calculation is often adopted for hand calculations. In this case the length of wall below the pivot point is replaced by an assumed resultant force, R, as illustrated in **Figure 64.11**.

The depth to the pivot point, d_0, is then varied until moment equilibrium is achieved and the magnitude of the resultant, R, that is required for horizontal stability can be calculated. To ensure the wall is long enough to ensure stability, the toe of the wall is generally assumed to be 20% deeper than the pivot point, i.e. $1.2d_0$. It is recommended that this length of wall below the pivot point is sufficient to mobilise a reaction that is greater than the assumed resultant force, R.

The free-earth condition is simpler to assess as it is assumed that the wall rotates about the level of the prop. The earth pressure on the retained side is therefore assumed to be at the active limit and the earth pressure on the excavated side is assumed to be at the passive limit. To verify stability, moments are taken about the assumed pivot point and the toe of the wall varied until equilibrium is achieved. Any imbalance in the lateral pressure acting on the wall is taken up as a reaction in the prop.

For either mechanism, once the stability of the system has been verified, the earth pressure arising from that analysis is used to derive the action effects (e.g. bending moments and shear forces) in the wall.

Design values of soil parameters should be used when determining the values of the limiting earth-pressure coefficients that are inputs into the calculation. DA-1, Combination 1 and Combination 2, will therefore yield different results for the wall length, pivot points and prop forces. This is because the different partial factors on material properties and actions will lead to different limiting earth-pressure coefficients and imposed load effects. In terms of wall length, the longer wall from the two combinations should be adopted in the design.

The use of limit-equilibrium methods to derive design action effects where the length of the wall is fixed by criteria other than rotation or lateral stability, such as to achieve water cut-off or for vertical load capacity, requires further consideration. The toe level required for the limit-equilibrium mechanisms to achieve an equilibrium condition will be different to the wall toe level required to satisfy the other criteria. Load effects will therefore not be calculated for the full length of the wall. This may be overcome by deriving the equilibrium earth pressure and load effects in the wall based on the limit-equilibrium analysis, and then extrapolating the load effects down to the actual required toe level, as illustrated in **Figure 64.12**. This approach also applies to the SLS condition, where the toe level from the limit-equilibrium analysis will be shorter than from the ULS analysis, therefore the SLS load effects need extrapolating to the ULS toe level.

If this extrapolation of the load proves to be a critical consideration in the wall design, it may be appropriate to consider using soil–structure interaction methods as described below.

A further limitation of the limit-equilibrium method is the lack of arching or earth-pressure redistribution in the assumed earth-pressure profile. This is particularly relevant for a wall propped at or near the top where there is a risk of underestimating the prop loads. This is discussed further in section 64.4.7 below.

64.4.5.2 Soil–structure interaction analysis

As noted above, in a soil–structure interaction analysis, the earth pressure acting on a wall is calculated using computer

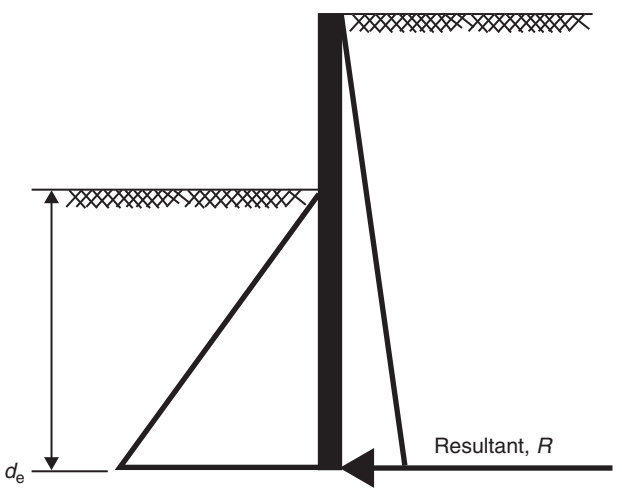


Figure 64.11 Simplification of the fixed-earth mechanism

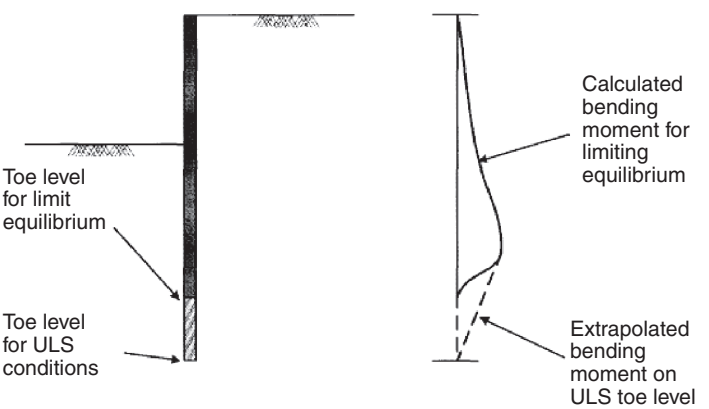


Figure 64.12 Extrapolation of load effects from limit-equilibrium analysis

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software. A number of different methods of soil–structure interaction analysis are available:

- Sub-grade reaction (e.g. Geosolve WALLAP);
- Pseudo-finite element (e.g. Oasys FREW, Geosolve WALLAP);
- Finite element and finite difference (e.g. Plaxis, FLAC, Oasys SAFE).

This chapter does not go into the detailed theory behind the different methods and the method of software operation, as these are dependent upon the program used. However, the main considerations to be taken into account for a soil–structure interaction analysis are presented below.

Unlike a limit-equilibrium analysis, the *in situ* stresses are an important input for a soil–structure interaction analysis. Typically the software will require a value of the at-rest earth-pressure coefficient, K_o, for each soil layer.

One benefit of a soil–structure interaction analysis is the potential for modelling the sequence of construction. The sequence in which a wall is constructed will affect the earth pressure that is generated in the soil adjacent to the structure and therefore the action effects acting on the wall. The stages of the construction sequence that are to be modelled should therefore be established when the design situations are considered.

During analysis of the construction sequence, time effects can also be taken into account. This would include allowing fine-grained materials to behave in an undrained manner in the short term, but as drained materials in the longer term. Creep and corrosion of structural components may also be modelled and it is important that such effects are represented in the software after following the recommendations in the user manual.

It is important to read and understand the technical basis upon which any software carries out calculations to ensure that the correct assumptions and inputs are made for the analysis. It is also crucial that the output is checked carefully. With a complex analysis, it can be difficult to understand its theoretical basis and there is a danger that output is accepted without being critically reviewed. Before accepting the results of a soil–structure interaction analysis the user should consider whether the results are as expected – does the earth pressure look reasonable? How much of the soil is at the active or passive limit? Does the mode of deformation look reasonable? It is useful to relate the results back to a limit-equilibrium analysis of the same problem, where possible.

One particular difficulty with a soil–structure interaction analysis is the determination of the limiting stability, such as would be required when shortening a wall to find the minimum length that is stable. Whether or not an analysis converges will depend upon the number of iterations specified and the magnitude of any convergence tolerances specified in a particular program. Often an analysis may indicate that it has converged, which could lead the user into thinking that the wall is stable, however, if excessive deformation has occurred in order to reach convergence, it may be considered that the wall has actually failed, despite reaching convergence. Again this should be picked up when checking the detailed output of an analysis.

64.4.5.3 Other influences on earth pressure

The preceding sections outline the basic failure mechanisms assumed in limit-equilibrium analysis, and the general approach to soil–structure interaction analysis necessary to calculate the earth pressure acting on a wall. However, there are a number of additional factors that will influence the basic approach that need to be taken into account when assessing the actions acting on an embedded wall. These are discussed below.

64.4.5.4 Wall–soil friction

The interface friction angle between the soil and the wall structure will affect the limiting earth pressure that is mobilised adjacent to a wall. The magnitude of the soil–wall interface friction angle will depend upon the ground conditions and the wall type. The direction of the wall–soil interface friction angle will depend upon whether the wall is carrying a vertical load. For the conventional case, on the retained side the soil slumps downward relative to the wall, giving an upward reaction on the wall–soil interface, whereas on the excavated side the soil heaves upward relative to the wall giving a downward reaction. See **Figure 64.13**. Useful guidance on the use and interpretation of a soil–structure interaction analysis is given in Chapter 6 *Computer analysis principles in geotechnical engineering*.

If, however, the wall is carrying a vertical load, upward shear stresses need to be mobilised on the wall–soil interface in order to support the applied loads. Over the retained height of the wall it may be considered that the soil and wall move downwards together – generating no net friction at the interface. The resistance to the applied load therefore comes from the embedded portion. On the excavated side this corresponds to a reversal in the direction of the interface friction direction in relation to the conventional case. This is important as it can significantly reduce the limiting values of the passive resistance

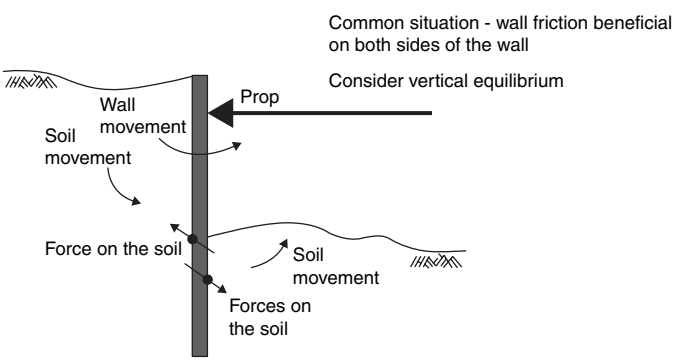


Figure 64.13 Common situation for wall friction

and may therefore increase the length of wall required for stability. Further details of walls carrying vertical loads are given in Chapter 62 *Types of retaining walls* as part of complete underground structures.

64.4.5.5 Passive softening

During excavation of the ground in front of a wall, the soil providing lateral stability for the wall is unloaded vertically. If this soil is undrained fine-grained material, there is a risk that the unloading will lead to a softening of the soil near the surface of the excavation as the excess pore pressure is dissipated. Any such softening will reduce the passive resistance available for wall stability and may therefore increase the required wall length. It is recommended that the value of the undrained shear strength at the surface of an excavation in fine-grained soils is taken as zero, rising to the design value of undrained shear strength at some depth, L, within the soil block, as illustrated in **Figure 64.14**. Where there is no potential for groundwater recharge this depth may be taken as 0.5 m. In other situations it may be taken as:

$$L = \sqrt{(12 c_v t)} \tag{64.4}$$

where recharge occurs at the excavation level but where there is no recharge within the soil. c_v is the coefficient of consolidation and t is the elapsed time.

Consideration should also be given to the potential effects of excavation unloading deeper in the soil mass.

64.4.5.6 Tension cracks

In fine-grained materials modelled with undrained shear strengths, there is a theoretical potential that on the retained side the active limit will be negative. Since soil cannot sustain tensile stresses, there is the potential for a crack to open up between the soil and the wall. In this zone the pressure upon the wall would be zero. Consideration must be given to such tension cracks becoming flooded with water, for example due to perched water on top of a clay layer. If a tension crack becomes flooded the pressure acting on the wall will be the hydrostatic pressure of the water in the crack. Even if there is no obvious source of water, it may not be appropriate to rely on a tension crack exerting zero pressure on the wall. In such cases it is common to limit the minimum pressure that is assumed to act on a wall to a minimum equivalent fluid pressure (MEFP). In dry conditions, the MEFP is typically taken as $5z$, where z is the depth below the retained ground surface. Water-filled tension cracks and the assumption of MEFP are illustrated along with passive softening of the ground in **Figure 64.14**.

64.4.5.7 Sloping ground

Sloping ground, either in front of or behind a wall, can significantly affect the pressure acting on the wall. BS EN 1997-1:2004 gives analytical and graphical methods for calculating the earth-pressure coefficients for sloping ground. A difficulty arises when the slope is uneven or of limited extent. In such situations, if approximating the profile to an infinite slope is unreasonably conservative, one of the following methods may be considered:

- Carry out a Coulomb wedge analysis to determine the active lateral thrust at different depths down the wall.
- Model the effect of the slope as a series of surcharges. It is noted that this method does not model the active thrust within the slope above the top of the wall. The active thrust should therefore be quantified separately and applied as an imposed force at the top of the wall.

64.4.5.8 Reverse passive

During excavation of the ground in front of a wall propped close to the top, the section of wall above the prop tends to rotate backwards into the retained soil. In a soil–structure interaction analysis this can mobilise an earth pressure up to the passive limit close to the top of the wall. In this zone, however, the soil is generally moving down relative to the wall (giving a negative value for the wall–soil interface friction, based on the standard sign convention for K_p), therefore it is important that the value of K_p applied over the soil zone near the top of the

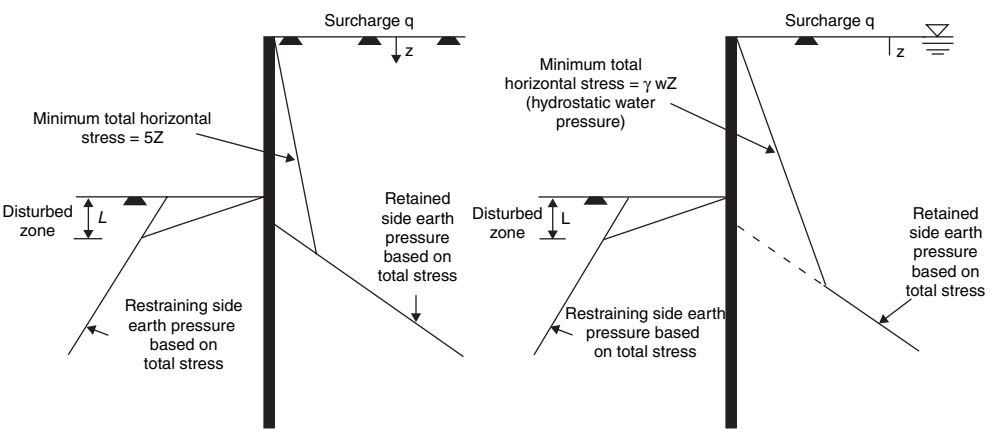


Figure 64.14 Passive softening and tension cracks. (a) Undrained conditions: dry tension cracks. (b) Undrained conditions: flooded tension cracks

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wall takes this into account. It is generally found that a value of $K_p = 1$ is appropriate.

A limit-equilibrium analysis will not incorporate the reverse passive pressure near the top of the wall as the assumed pressure profile does not include it. The absence of this increased pressure is the main reason that limit-equilibrium analyses underestimate prop forces in a wall support system. A model factor should be applied to the calculated limit-equilibrium prop force to ensure the design is robust. Further details are given in section 64.4.7 below.

64.4.5.9 Multi-propped walls

Where walls are propped by more than one level of support, instability of the wall due to rotation or translation is not a concern, provided the structural design of the wall and the support system is adequate. However, to design the wall and the support system it is still necessary to calculate the earth pressure acting on the wall. Limit-equilibrium analysis is not particularly well suited to multi-propped situations, though some approaches are discussed in *CIRIA C580* (Gaba *et al*., 2003). In general, it is recommended that for multi-propped walls a soil–structure interaction analysis is undertaken, with the sequence of prop installation and excavation being modelled.

64.4.5.10 Wall and soil stiffness

Whilst not required for a limit-equilibrium analysis, wall and soil stiffness are important inputs in a soil–structure interaction analysis. It is important that the stiffness adopted is appropriate for the level of strain anticipated in the soil. Generally for retaining wall design this strain will be lower than for foundation design, therefore higher values of stiffness may be justified. In all cases the resulting wall and soil movements arising from the analysis should be verified against comparable experience, such as the wall and soil movement case histories presented in *CIRIA C580* (Gaba *et al*., 2003) or other technical papers, to ensure that the assumptions are reasonable.

Modelling wall and prop stiffnesses should take into account any time related effects. For example the bending stiffness of a reinforced-concrete wall is given as E_oI, where E_o is the short-term Young's modulus of the reinforced concrete and I is the second moment of area. However, as soon as a wall is loaded by bending, cracking will occur on the side of the wall that is in tension, which acts to reduce the stiffness of the wall that is bending. In the absence of a detailed calculation it is recommended that a value of $0.7E_oI$ is taken as the initial stiffness of the wall. In the long term, reinforced-concrete walls have the potential to creep due to strain at constant stress. Typically the long-term bending stiffness is taken as $0.5E_oI$.

In steel retaining walls, a reduction in stiffness may occur due to corrosion of the steel over the lifetime of the structure.

Modelling changes in wall stiffness between the short and long term needs to be appropriate to the software being used, and the user should consult the specific guidance for the software. In general, it is not acceptable to simply change the wall stiffness at any stage of the analysis.

64.4.5.11 Thermal effects

Temperature variations can lead to expansion and contraction of support systems, which may induce additional loads on the structure. In general this is more likely to be significant where the support system is exposed to atmospheric conditions, such as during construction, especially where temporary steel props are used. A method for calculating thermal effects on props is given in section 64.4.7 below.

Thermal effects may be significant for integral bridges (bridges without movement joints between the deck and the abutments). In these cases the abutment walls are subject to cyclic expansion and contraction that can lead to strain ratcheting – an effect that increases the earth pressure behind the wall. Specific guidance for the analysis and design of integral bridges is given in the *Design Manual for Roads and Bridges* (Highways Agency, 2000).

64.4.5.12 King post walls

King post walls are a specific type of embedded wall. It is generally assumed that retaining walls are a plane-strain problem and therefore the calculations are generally carried out on the basis of a 'unit length' of wall. King post walls, however, derive support from discrete elements embedded at a fixed spacing, s. Where s is less than three times the diameter of the support element, it may be appropriate to assume plane-strain conditions, however, for wider spaced supports a different approach is required, which is set out in *CIRIA C580* (Gaba *et al*., 2003) and summarised below.

The individual support elements should be designed individually to resist the calculated lateral loads. Full active conditions should be assumed behind the retaining wall. The passive resistance in front of the wall may be taken as follows (taken from Gaba *et al*., 2003; Equations (4.14) to (4.17)):

'The king posts should be designed as piles in lateral loading, with an ultimate net effective resisting force P'_a per metre length of

$$P'_a = K_p.b.p'_v/s \quad at\ embedment\ depths\ z \le 1.5b \qquad (64.5)$$

and

$$P'_a = K_p^2.b.p'_v/s \quad at\ embedment\ depths\ z \le 1.5b \qquad (64.6)$$

where b is the king post width, s is the spacing of the king posts ($s > 3b$), K_p is the passive earth-pressure coefficient ($K_p > 3$) defined as $(1 + \sin \varphi') / (1 - \sin \varphi')$, and p'_v is the effective overburden pressure at depth (Fleming *et al*., 1994).

The above expressions for P'_a are based on the work of Barton (1982) as reported by Fleming *et al*. (1994) and are considered applicable for K_p values of between 3.0 and 5.3 (i.e. $30° \le \varphi' \le 43°$).

In a total stress analysis, the ultimate net lateral resisting force per metre length P_u is given by Fleming *et al*. (1994) as:

$$P_u = \left[2 + \left(7z/3b\right)\right].s_u.b/s \quad at\ embedment\ depths\ z \le 3b \qquad (64.7)$$

and

$$P_u = 9.s_u.b/s \quad at\ embedment\ depths\ z \le 3b \qquad (64.8)$$

where b is the king post width, s is the spacing of the king posts ($s > 3b$), and s_u is the undrained shear strength at embedment depth z.'

This is illustrated in **Figure 64.15**.

64.4.5.13 Unplanned excavation

When assessing the geometry for a given design situation, it is important to take into account the possibility of an unplanned excavation in front of the wall. This is particularly critical for an embedded wall, where the passive resistance of the ground is necessary for stability. BS EN 1997-1:2004 requires that for normal site controls, 10% of the retained height, up to a maximum of 0.5 m, should be assumed for unplanned excavations.

64.4.5.14 Surcharges and direct loads

The design of a retaining wall should consider the possibility of surcharges being applied to the ground retained by the wall. Such surcharges may arise, for example, from traffic in the permanent condition, plant loading during the construction phase or from the placement of stockpiled materials behind the wall. The effect of such surcharges must be incorporated into the assessment of the earth pressure acting on the wall.

The method of modelling a surcharge will depend on how the wall is analysed. Where specialist software is used, the user manual should be consulted to ensure the surcharges are properly modelled. Uniformly and non-uniformly distributed loading, such as loading over a limited area, line load and point loads, can be modelled directly in a limit-equilibrium analysis. *CIRIA C580* (Gaba *et al*., 2003) gives different methods for determining the limiting lateral pressure due to non-uniformly distributed loads.

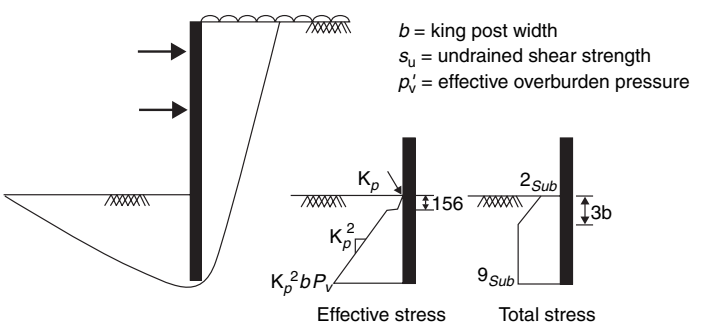


Figure 64.15 King post wall design. (a) Overall stability. (b) Lateral loading of individual king posts

BS 8002 (British Standards Institution, 1994b) has a requirement to consider a minimum 10 kPa surcharge behind a wall. Although this is not an explicit requirement in BS EN 1997-1:2004, for most walls some allowance should be made and 10 kPa is considered a reasonable value.

64.4.5.15 Groundwater

Understanding the groundwater conditions is crucial to embedded retaining wall design. Since the water pressure directly influences the effective stresses, and hence strength, of the ground, applying a partial factor on the water pressure may lead to unreasonable design situations. In order to ensure a margin of safety is achieved, it is recommended that a margin is applied to the groundwater levels upon which the pore pressures are based. The approach for selecting the groundwater level differs in DA1-1 and DA1-2. In DA1-1, the actions of water on the structure are factored by a value greater than unity. Whereas in DA1-2, the factor on permanent actions is unity. This means that for DA1-2, no factors are applied to the effect of water so it is recommended that a suitably cautious design water pressure is derived from a directly assessed design groundwater level, defined as:

> (1) Water pressure and seepage forces that represent the most unfavourable values, which could occur in extreme or accidental circumstances at each stage of the wall's construction sequence and throughout its design life. An example of an extreme or accidental event may be a burst water main in close proximity to the wall.

For DA1-1, a factor greater than unity is applied to water effects, so it is considered that a less cautious assessment of the groundwater level may be used than for DA1-2. The recommended approach is defined as:

> (2) Water pressures and seepage forces that represent the most unfavourable values likely to occur in normal circumstances at each stage of the wall's construction sequence and throughout its design life. Extreme events such as a nearby burst water main may be excluded, unless the designer considers that such an event may reasonably occur in normal circumstances.

When selecting the design groundwater levels, consideration must be given to any potential effect the construction of the embedded wall may have on the groundwater regime. For example, if the wall penetrates a layer of flowing groundwater, there may be a 'dam' effect that leads to a rise in groundwater levels adjacent to the wall. Conversely the installation of drainage within a basement may lead to a lowering of the equilibrium groundwater level. Such effects need to be considered in the design of the wall, and possible effects on adjacent structures must also be considered.

Where differing groundwater levels are identified on each side of a retaining wall, the potential for seepage around the wall must be investigated, as this will have an effect on the pore pressures assumed to act along the length of the wall. Where an impermeable wall penetrates an impermeable stratum, the assumption of unbalanced pore pressures at the toe of the wall

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may be acceptable. However, in many situations, especially over the design lifetime of a structure, since the wall and the soil have a finite permeability there will be seepage around the wall and generation of non-hydrostatic pressure profiles.

In order to assess the pore pressures acting on the wall accurately, it may be necessary to undertake a seepage analysis. However, there is also a commonly adopted simplification to the seepage problem that has been developed on the basis of isotropic permeability and wide excavations. For this to be applied, the design must verify that these simplifying assumptions are reasonable. The method of calculating this simplified linear seepage profile is given in **Figure 64.16**. Further details of the effect of seepage on retaining wall design are discussed in *CIRIA Special Publication 95* (Williams and Waite, 1993).

64.4.6 Design of structural elements

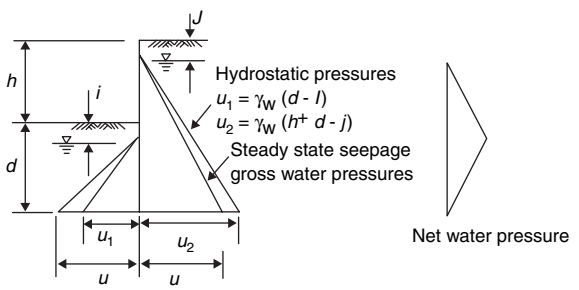
The load effects from either the limit-equilibrium or soil–structure interaction analyses should be used in the structural design of a wall.

64.4.7 Design of propping systems

The design of propping systems is covered in Chapters 65 *Geotechnical design of retaining wall support systems* and 66 *Geotechnical design of ground anchors*. Careful consideration must be given to determining the design load effects that are to be used in the propping design. In general these will be derived from the reactions calculated in the retaining-wall analysis. However, there may be additional factors that need to be taken into account to ensure a robust solution is achieved.

64.4.7.1 Limit-equilibrium prop loads

As noted in section 64.4.5 above, the limit-equilibrium method for propped walls does not consider soil arching or redistribution of the earth pressure. This can lead to a significant underestimate of the prop loads that may be appropriate for use in design. Whilst this is conservative for the wall design, it is not conservative for prop design. *CIRIA C580* (Gaba *et al.*, 2003), therefore, recommends that a model factor of 1.85 be applied to the support reactions derived from a limit-equilibrium analysis to ensure a robust design is achieved.



= Assumption (1)
Uniform dissipation of differential head along flow path adjacent to the wall

$$u = \frac{2(d + h - j)(d - j)}{2d + h - i - j}$$

= Assumption (2)
Average hydrostatic pressure at wall toe

$$u = \frac{u_1 + u_2}{2} = \gamma_w \left| d + \left(\frac{h - i - j}{2} \right) \right|$$

Figure 64.16 Simplified approach for calculating the linear steady-state seepage water pressure

64.4.7.2 Distributed prop load method

A method for deriving design actions for a temporary prop design is the distributed prop load (DPL) method. This method is outlined in *CIRIA C517* (Twine and Roscoe, 1999) and is based on extensive field measurements of prop loads for flexible and stiff walls and for the range of ground conditions commonly encountered in the UK. Where the DPL method is adopted, the results should be compared against the findings of the wall analysis for the comparable temporary case. Where the prop loads differ between the two methods the designer should consider the reasons for the difference and adopt appropriate values in the design of the temporary props

64.4.7.3 Thermal effects

As noted in section 64.4.5 above, thermal effects can increase the design load effects for props and retaining walls. The effect on props due to thermal variations can be calculated as follows:

An increase or decrease in the temperature of a prop from its installation temperature will cause the prop to expand or contract according to the relationship:

$$\Delta L = \alpha . \Delta t . L \qquad (64.9)$$

where: ΔL = change in prop length

α = thermal coefficient of expansion for the prop material

Δt = change in prop temperature from the installation temperature

L = prop length

If the prop is restricted or prevented from expanding freely, an additional load is generated in the prop. The magnitude of this addition load is:

$$\Delta P_{temp} = \alpha . \Delta t . E . A . (\beta/100) \qquad (64.10)$$

where: E = Young's modulus of the prop material

A = cross-sectional area of the prop

β = percentage *degree of restraint* of the prop

(70% for stiff walls in stiff ground and 40% for flexible walls in stiff ground).

The designer should select the appropriate value of β to suit particular project circumstances. Values other than those generally

recommended above may be applied where the designer is confident that such values can be justified, e.g. on the basis of comparable experience.

64.4.8 Ground movement

Excavation in front of an embedded wall will lead to movement of the wall and the ground around the excavation. Where sensitive infrastructure or buildings are in close proximity to the wall, there is a risk of damage if movement is not limited. A check of the ground movement caused by the construction of an embedded wall is often required as part of the verification that SLS criteria have not been exceeded.

Methods for predicting ground movement are discussed in detail in Chapter 63 *Principles of retaining wall design*.

64.4.9 Design method

The preceding sections outlined how the various loads that may act on a wall can be assessed. Once these actions have been identified, it is necessary to analyse the wall to verify that the limit states identified in section 64.4.2 above will be avoided. For a soil–structure interaction analysis, an iterative approach is necessary, where an initial estimate of the wall geometry is made and stability against each failure mode is checked. The geometry of the wall may then be adjusted to optimise the solution while still ensuring stability. Such analysis and iteration is likely to require proprietary software.

This section outlines the stages in the design process for an embedded wall. These stages are illustrated by the flowchart in **Figure 64.17** and described below.

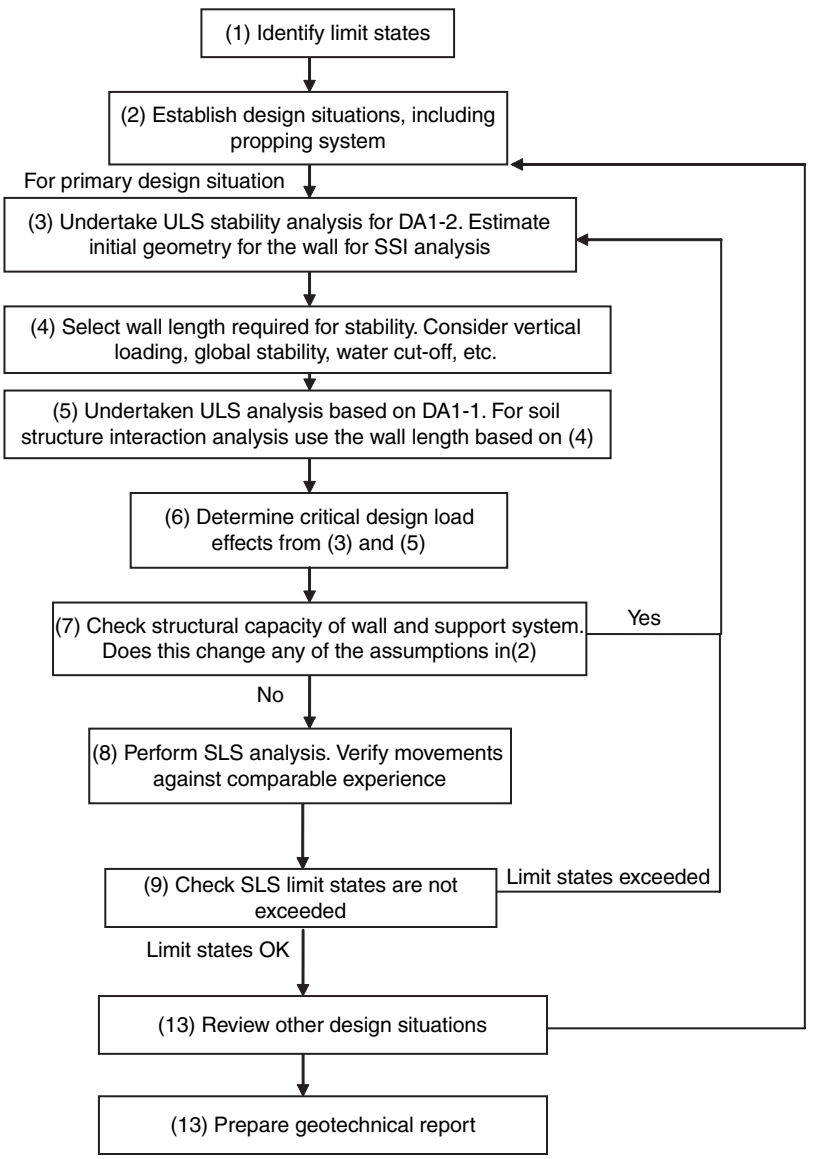


Figure 64.17 Embedded wall design process

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(1) Confirm the relevant limit states for the problem, as identified in section 64.4.2. For serviceability limit states agree the deflection limits or other criteria that are to be adopted.

(2) Identify the design situations to be considered in the analysis, considering the issues described in section 64.4.4, and make an assumption regarding the anticipated support system for the wall. Select an initial situation to analyse.

(3) Carry out a ULS analysis of the wall stability (Design Approach 1, Combination 2). For a soil–structure interaction analysis, an initial size for the wall must be assumed. In such cases it may often be appropriate to carry out a limit-equilibrium analysis to make an initial estimate of the required wall length.

(4) On the basis of (3), select the design length of the wall. For a soil–structure interaction analysis, step (3) may need to be iterated to optimise the solution. Consider whether other factors, such as global stability, vertical stability or water cut-off requirements will be critical for the wall length.

(5) Undertake a further ULS analysis (DA1-1) based on the geometry from (4).

(6) Consider the load effects from (3) and (5) and determine the design load effects in the wall and the support system.

(7) Undertake the structural design of the wall and support system. Consider whether this changes any of the initial assumptions (e.g. wall stiffness, support levels or support stiffness). If so, return to step (3) and re-analyse.

(8) Undertake SLS analysis to estimate wall movements. Validate against comparable experience or case history data, such as that presented in *CIRIA C580* (Gaba *et al.*, 2003).

(9) Check SLS limits are not exceeded. If they are, return to step (3).

(10) Review other design situations.

(11) Prepare the geotechnical report.

64.5 References

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64.5.1 Further reading

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Padfield, C. J. and Mair, R. J. (1991). *Design of Embedded Retaining Walls in Stiff Clay.* Publication R104. London: CIRIA.

Puller, M. (2003). *Deep Excavations: A Practical Manual* (2nd Edition). London: Thomas Telford.

64.5.2 Useful websites

Design Manual for Roads and Bridges (DMRB) from the Highways Agency; www.standardsforhighways.co.uk/dmrb/

It is recommended this chapter is read in conjunction with

- Chapter 20 *Earth pressure theory*
- Chapter 64 *Geotechnical design of retaining walls*
- Chapter 85 *Embedded walls*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 65

Geotechnical design of retaining wall support systems

Sara Anderson Arup, London, UK

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As identified in Chapter 62 *Types of retaining walls*, embedded retaining walls gain lateral support from passive resistance of the soil in front of the embedded section of the wall, while gravity walls gain lateral support from passive resistance and friction of the soil on the underside of the wall. These walls may also rely on anchorages, props or other sources of additional lateral support for stability.

Many different forms of support systems are available with different properties; these vary from strength, stiffness and constraints to working conditions within the excavation. An understanding of these different properties as well as the site-specific constraints and characteristics is required to determine the most suitable support system for a project.

This chapter provides the reader with an overview of different methods of providing lateral support to both gravity and embedded retaining walls.

65.1 Introduction

Embedded retaining walls gain lateral support from passive resistance of the soil in front of the embedded section of the wall, while gravity walls gain lateral support from a combination of passive resistance and friction of the soil on the underside of the wall. Under many circumstances it is necessary to provide additional lateral support to the wall, to restrict the lateral movement of the wall, or to provide a more economical retaining wall solution.

This chapter presents the design and performance criteria to be considered by the designer as well as the advantages and disadvantages of adopting different methods of lateral support.

Key references for retaining wall support systems include:

- CIRIA C517 (Twine and Roscoe, 1999);
- CIRIA C580 (Gaba *et al*., 2003);
- *Piling Handbook* (ArcellorMittal, 2008);
- BS 8002 (1994); and
- Eurocode EC7 (2004).

65.2 Design requirements and performance criteria

Support to a retaining wall must be considered during all stages of the life of the retaining wall. In many cases, the temporary conditions during construction, which are dependent upon the sequence of construction adopted, can be more onerous than the permanent conditions and therefore critical for the design of the wall.

65.2.1 Temporary situations

Temporary situations predominantly occur during construction but can also relate to later activities such as major remedial or refurbishment works. It is very common for the stability of a basement embedded retaining wall to be dominated by the temporary condition, as the excavation reaches its maximum depth – but before the lowest permanent basement slab is constructed. Similarly the construction or maintenance of drainage in front of gravity retaining walls can be a dominant condition for stability.

65.2.2 Permanent situations

In the permanent situation or condition, retaining walls may be provided with any permanent support systems – such as slabs or permanent anchors – which will need to be designed to resist the lateral loading appropriate to the full design life of the wall.

65.2.3 Bottom-up/top-down construction sequences

For basement excavations using embedded retaining walls, two different sequences of construction may be adopted, as identified in Chapter 63 *Principles of retaining wall design*.

The *bottom-up construction sequence*, in its simplest form, consists of a cantilever retaining wall supporting the ground surrounding the excavation, followed by construction of the permanent works within the cantilever excavation. However there are many situations when additional temporary support, often in the form of temporary props or ground anchors, are utilised to supplement the earth pressures on the passive side of the wall during excavation. Following excavation, the permanent works are constructed within the excavation; the temporary support is subsequently removed once the permanent works are in place and to provide sufficient lateral restraint.

Top-down construction is defined by the use of the permanent internal structure of a basement to prop the retaining wall during excavation without the reliance on temporary propping

systems. To achieve this, the higher level slabs are constructed before the excavation progresses below the slab level and before casting the lower level slabs.

65.2.4 Control of ground movements

Ground movements result from both the installation of the retaining wall and the excavation. Control of ground movements is important in urban settings where buildings and infrastructure such as utilities are in close proximity to an excavation and will incur damage from ground movements. The magnitude of ground movements due to excavation in front of a retaining wall is dependent upon the stiffness of the wall and support systems. As such, the requirements for the control of ground movements may be critical in the selection and design of retaining wall support systems.

65.2.5 Accidental conditions

Accidental conditions, for example, leading to the loss of a support element such as a prop or anchor, should be considered in the design. The loss of one support element will shed loading into adjacent support elements. If these are not designed to resist this additional loading, they too may fail – leading to progressive loss of support elements and therefore excessive deflection if not progressive collapse of the retaining wall.

65.2.6 Design responsibility

The responsibility for the design of both the retaining wall and any lateral support will not always lie with the same party and therefore must be clearly defined. Situations where there is likely to be more than one design party involved include the following:

- *Temporary retaining wall* A temporary wall may be used to form an excavation, into which the permanent structure will be constructed. Performance criteria, such as the maximum wall deflection to ensure that the wall does not impinge on the permanent works, need to be clearly defined for the designer of the temporary wall. The temporary wall design will also need to consider the proposed construction sequencing to minimise constraints placed by the temporary works on the construction of the permanent works. The staging of the temporary works, for example the removal of temporary propping, can affect the permanent works. As such, the design of both the temporary and permanent works must fully account for any restrictions or reliance placed on one system by the other.
- *Permanent retaining wall with temporary support* For basement excavations it is not uncommon to utilise the permanent embedded retaining wall with temporary propping or anchoring during excavation and construction of the basement. The design loading acting on the retaining wall during construction will be very dependent on the strength, stiffness and sequence of installation and removal of the temporary support system. In this situation, the designer of the permanent wall will need to inform the designer of the temporary support system of the assumptions made regarding the strength, stiffness and staging of the temporary works in the design of the permanent works. If the temporary support designer is unable to satisfy these requirements, then the design of the permanent works, temporary works or construction staging will need to be modified to achieve a coordinated temporary and permanent works design solution.
- *Specialist lateral support* The permanent works designer may not possess the relevant specialist design skills for some of the methods of lateral support. Alternatively, a specialist contractor may provide additional value to the project by undertaking the design of the permanent lateral support to a wall. For this situation, key design assumptions of strength, stiffness, maximum wall deflection, construction staging and design life must be clearly communicated by the permanent works designer to the specialist designer to achieve a coordinated design solution.

65.3 Types of wall support systems

Many different forms of wall support systems can be used for retaining walls. Some of the more common types of wall support systems are briefly described below:

- *Cantilever* The retaining wall relies only on the passive ground resistance or sliding resistance of the wall and therefore has no additional wall support system.
- *Single support* A single level of support is provided to the retaining wall. The introduction of additional support will reduce the loading in the wall and therefore reduce its size, compared to a cantilever wall. It will also increase the stiffness of the excavation support system and therefore reduce ground movements behind the wall. As such, support is typically adopted either where the excavation depth is too deep for an economic cantilever wall with where a cantilever wall would result in unacceptably large wall deflections and associated ground movements, or where the introduction of support results in an overall saving for the project.
- *Multiple support* For deeper excavations, a single level of support is not sufficient to provide the strength and stiffness required to retain the ground behind the wall with acceptable wall deflections and associated ground movements; therefore several levels of support are required.
- *Circular construction* Retaining walls, constructed in a circular layout, derive support from hoop action due to the shape of the wall layout and excavation. For walls which provide a solid cross-section such as secant walls or diaphragm walls, support is likely to be provided by hoop action through the wall itself. In other situations, a continuous ring waling beam will be required to provide the hoop action.
- *Buttressed walls* A buttress is a local thickening of a gravity retaining wall. It acts to increase the stiffness of the wall and also to improve the resistance of the wall, particularly to overturning. Buttresses are more commonly provided to masonry walls and form an architectural feature to many historical masonry buildings and walls.
- *Temporary props* Props consist of steel sections, typically (but not always) orientated perpendicular to the line of a retaining wall. They provide lateral support by acting in compression, transmitting loading from the retaining wall on one side of the excavation onto a retaining wall on the opposite side, or onto another source of resistance.
- *Ties* Support to a retaining wall can also be provided by tying it back to another structure using a steel rod. The ties can be

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connected back to an object such as a mass concrete block which relies on friction and passive resistance of the ground between the block and the wall to resist the tie force. They can also be connected back to embedded elements such as piles or other nearby embedded walls which utilise their stiffness and the passive resistance of the ground in front of the piles/embedded wall section to resist the tie loads.

- *Berms* The use of an earth berm in front of, and immediately adjacent to, a retaining wall reduces the effective depth of excavation felt by the retaining wall whilst allowing a deeper excavation within the centre of the site. Raking props from the top of the berm to a thrust block at the bottom of the excavation in the central area of the site can then be used to replace lateral support provided by the berm when excavation proceeds to full depth adjacent to the wall. Alternatively, removal of the berm down to the final excavation level and construction of the lowest slab level can be carried out in isolated sections or bays. Although there is a reduction in the lateral support provided to the retaining wall, arching in the soil and distribution of loading through the retaining wall capping beam reduces the loading and deflection that occurs on the short section or bay of wall.
- *Ground improvement* Strengthening or stiffening of the ground, particularly within the passive zone below the excavation level, can be utilised to provide support. Ground improvement methods include:
  - *Soil mixing, jet grouting or other strengthening measures* Such measures can be used to create a 'slab' of improved soil spanning between retaining walls (retaining an excavation) to provide enhanced passive restraint.
  - *Crosswalls* These are constructed using unreinforced slurry or diaphragm wall trenches and can enhance the passive strength and stiffness of the main retaining wall. Crosswalls provide enhanced passive performance at their locations, but little improvement between crosswalls. As such, they are most effectively used with diaphragm walls where a single crosswall can be provided for each diaphragm wall panel.
- *Ground anchorages* Anchors provide support to a retaining wall by transferring tension loading to a load bearing stratum behind the wall through friction over a grouted length of the anchor. Ground anchors are discussed further in Chapter 66 *Geotechnical design of ground anchors*.

65.4 Props

65.4.1 General

Temporary props are typically formed from steel sections but can also be formed from reinforced concrete. Ideally the props should be located at regular intervals along the length of the excavation using a waling beam to distribute the restraint between the individual props. Both the strength and stiffness of the propping system will be dependent on the properties of the waling beam, as well as the size and spacing of the props.

Temporary props will span across an excavation and provide a physical restriction on the working methods that can be adopted within the excavation. From program and ease of construction perspectives, there is a desire to have propping at larger intervals to minimise the restrictions on working methods. This desire needs to be balanced not only with the strength and stiffness of the propping system, but also with the practical issues related to installing and supporting large prop and waling sections.

65.4.2 Propping layout

The layout of propping will depend on the shape of the excavation (**Figure 65.1**). For long thin trough excavations, propping at regular intervals can be provided across the excavation. For rectangular basement excavation, propping will be required in two directions. This can be achieved using cross-bracing and diagonal propping in the corners.

Diagonal bracing can also be utilised to provide effective propping with an enlarged opening between props, which is often used as the location for mucking out of the excavation (**Figure 65.2**).

65.4.3 Inclined struts

Propping is typically installed horizontally. However, inclined struts can be used as raking props in conjunction with a berm solution, as described in section 65.3 above (**Figure 65.3**). They can also be used for unbalanced excavations where the excavation depth and loading is different on the opposite sides of an excavation.

65.4.4 Prop stiffness/pre-load/pre-stress

The stiffness of a propping system per metre length of wall is dependent on the Young's modulus of the propping member (E), the cross-sectional area of the member (A), the effective length of the prop (L), the angle of inclination of the prop from the horizontal (α) and the spacing of the individual props (s), and is given by:

$$k = EA\cos^2\alpha/Ls.$$

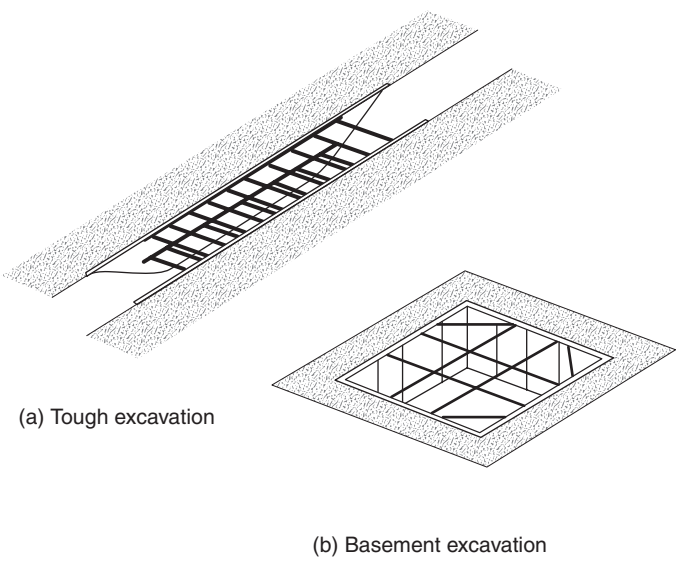


Figure 65.1 Typical excavations with temporary propping
Reproduced with permission from CIRIA C517, Twine and Roscoe (1999), www.ciria.org

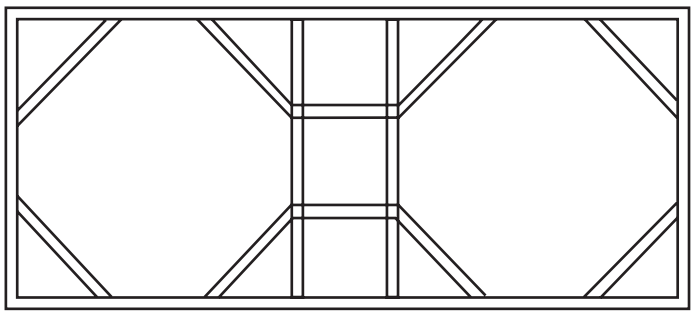


Figure 65.2 Use of diagonal bracing to provide enlarged opening for access
Reproduced with permission from CIRIA C517, Twine and Roscoe (1999), www.ciria.org

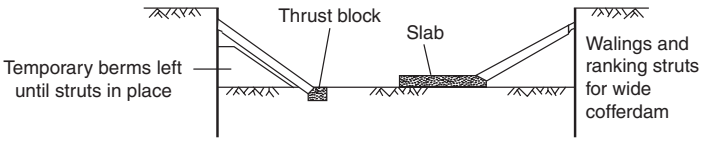


Figure 65.3 Use of inclined struts with temporary berm
Reproduced with permission from CIRIA C517, Twine and Roscoe (1999), www.ciria.org

For 'balanced' excavations, where excavation depth and loading on both ends of the prop are similar, L can be taken as the half of the excavation width. However for unbalanced or sway excavations, where the excavation depth and loading are much larger on one end of the prop, the length should be taken as the full excavation width.

Although this assessment of propping stiffness is commonly used, it is an approximation as it does not consider the influence of the waling beam on the effective stiffness of the propping system (**Figure 65.4**).

The application of pre-load or pre-stress into the propping system – typically around 10% of the design (working) load – can be used to take up the slack in the system. Higher preloading can be used to stiffen the support system to reduce wall deflections and hence movements behind the wall.

65.4.5 System stiffness

Available case history data indicate that the magnitude of horizontal wall deflection of embedded retaining walls is dependent on the effectiveness of the support system.

Clough *et al*. (1989) define system stiffness as $EI/(\gamma_w h^4)$, where I is the second moment of area of the wall section, γ_w is the bulk unit weight of water and h is the average vertical prop spacing of a multi-propped support system. Case history data indicate that wall deflections and associated ground movements are relatively insensitive to variation in wall thickness and stiffness, provided the overall system stiffness is not significantly reduced. Because of this, CIRIA C580 (Gaba *et al*., 2003) comments that economies in wall type and size can be achieved by adopting flexible walls (e.g. sheet pile walls) in stiff soils, without greatly increasing ground movements. Thus,

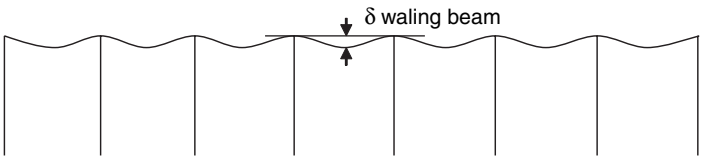


Figure 65.4 Waling beam deflection

in general, flexible walls with many props (smaller h) will give similar displacements to stiff walls with fewer props (larger h). However, it should be noted that the cost of additional propping should be offset against the cost benefit of using a more flexible wall. More extensive propping may be counterproductive, as it may slow the rate of construction and thereby increase ground movements.

65.4.6 Temperature effects

The change in length of a prop due to temperature effects depends upon the thermal coefficient of expansion (α), increase or decrease in the temperature of a prop from its installation temperature (Δt) and prop length (L):

$$\Delta L = \alpha \, \Delta t \, L.$$

In practice the waling beam, wall and retained ground at the end of the props, will restrict or prevent free expansion leading to an increase in loading within the props. The effects of temperature will be more significant for steel props in comparison to reinforced concrete due to higher values of coefficient of expansion in steel. Refer to CIRIA C517 (Twine and Roscoe, 1999) for further guidance. Thermal effects will be more pronounced for props in direct sunlight and less pronounced for more shaded lower levels of propping.

65.4.7 Accidental loading

The design of individual props should be robust and include consideration of accidental loading and the possible loss of a prop. The design needs to:

- incorporate the loss of a prop in the design of the wall and support systems; or
- provide adequate mitigation of the risk of accidental damage or loss of a prop through design changes and robust construction management strategy.

65.4.8 Buckling

Props are slender axially loaded elements. Therefore the design of props needs to consider buckling due to excessive or eccentric axial loading. For wider excavations or highly loaded props it may be necessary to provide mid-span restraint to the props. This can be provided using temporary king post piles within the centre of the excavation.

65.4.9 Determination of prop load for design

Refer to Chapter 64 *Geotechnical design of retaining walls* for the determination of serviceability limit state and ultimate

limit state prop loads using limit equilibrium and soils structure interaction methods.

Designers of temporary props may also make use of the distributed prop load (DPL) method presented in CIRIA C517 (Twine and Roscoe, 1999). The DPL method for calculating prop loads is based on the back analysis of field measurements of prop loads relating to 81 case histories, of which 60 are for flexible walls (steel sheet pile, king post walls) and 21 are for stiff walls (contiguous, secant, diaphragm walls). The case histories relate to excavation ranging in depth from 4 m to 27 m in stiff to very stiff clays, and 10 m to 20 m in coarse-grained soils.

Based on the case history data, a family of distributed prop load diagrams have been determined and are presented in CIRIA C517 (Twine and Roscoe, 1999) and summarised in CIRIA C580 (Gaba *et al*., 2003) (**Figure 65.5**). Further reference should be made to these publications for the details of the method as well as the considerations to be satisfied when using it.

65.4.10 Permanent structure propping

Permanent propping of embedded retaining walls is typically provided by permanent reinforced concrete slabs. Although the Young's modulus of reinforced concrete is lower than that of steel, the cross-sectional area of a solid slab is significantly higher than for temporary steel propping. Therefore, a reinforced concrete slab provides a stiffer support solution than temporary steel propping. This enhanced stiffness is one of the benefits of a top-down construction sequence to minimise wall deflections and hence ground movements.

The stiffness (k) of the slab to axial loading is dependent upon the Young's modulus of concrete (E), thickness of the slab (t) and effective length of the slab perpendicular to the wall (L):

$$k = Et/L.$$

As for the stiffness of temporary props, the effective length is dependent upon whether the excavation is balanced, or subject to sway loading, see section 65.4.4 above.

Large holes in the slab, for example for services, can locally influence the stiffness of the slab in supporting the retaining wall. The effect is most significant for holes adjacent to the retaining wall.

65.5 Tied systems

65.5.1 Ties and deadman anchors

Deadman anchors can be formed of either a plate section (e.g. sheet pile section) or a mass concrete block and provide restraint to a wall by means of passive resistance in front of the plate section or mass concrete block (**Figure 65.6**).

Where the passive soil zone in front of the anchor does not interfere with the active soil zone behind the main wall, the design of the main wall and the anchor can be undertaken independently of each other (**Figure 65.7**).

Ties can also be used to connect or tie a wall back to embedded elements for support; for example, raking tension piles (**Figure 65.8**). Embedded elements are able to gain passive resistance over a deeper extent than the plates or mass concrete plates used in deadman anchors. For raking elements, the resistance is provided by tension within the piles.

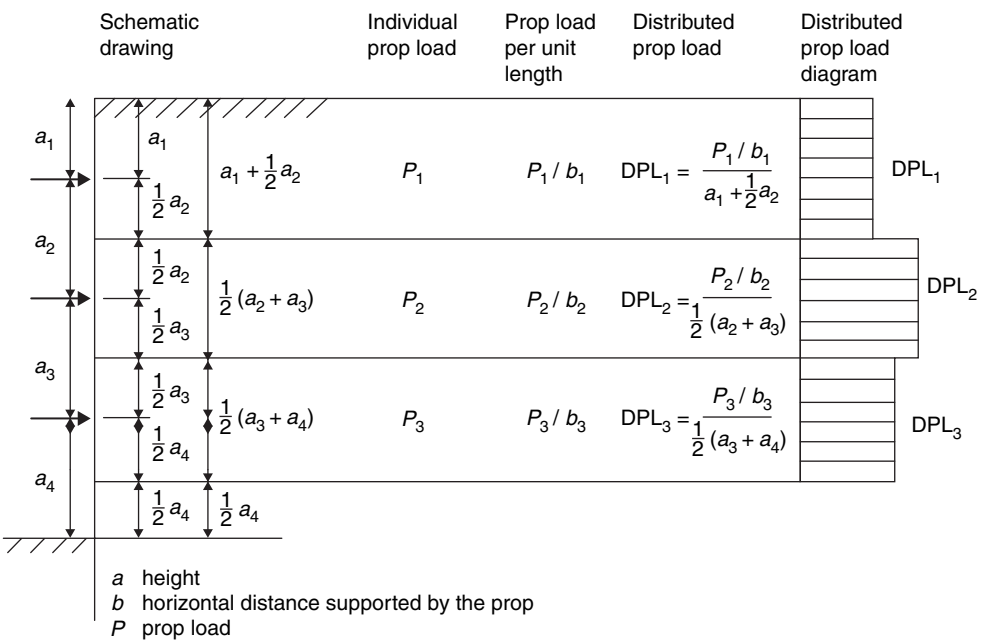


Figure 65.5 Method for calculating the distributed prop load. In order to compute prop loads from a distributed load diagram, the procedure is reversed

Reproduced with permission from CIRIA C517, Twine and Roscoe (1999), www.ciria.org

The effectiveness of a tied system will depend on the depth to which the ties and deadman anchors can be installed. This will be affected by practical considerations of excavation for the anchors. As such, tied systems are unlikely to be feasible where multiple levels of support are required; for instance, where the ground rises behind the retaining wall or where the groundwater level is high.

65.5.2 Earth-filled cofferdams

Earth-filled double-wall or cellular cofferdams are another form of tied structure. They are formed from two parallel lines of sheet pile walls connected to each other by means of waling beams and tie rods. The gap between the two lines of piling is backfilled with sand, gravel, broken rock or other coarse-grained material. Cellular cofferdams are formed by interlocking sheet pile sections to form cells which are then backfilled with coarse-grained material (**Figure 65.9**).

Earth-filled cofferdams are typically used in marine applications. The sheet piles are installed through the water and into the underlying ground before placing backfill between the rows or within the cells to above the surrounding water level. The cofferdams are designed to allow dewatering and excavation from within the area they enclose.

Further design consideration of tied systems including earth-filled cofferdams is given in BS8002 (1994), Piling Handbook (ArcellorMittal, 2008), Williams and Waite (1993) and for ground anchors, guidance is provided in Chapter 66 *Geotechnical design of ground anchors*.

65.6 Soil berms

Berms can be used to provide support to an embedded retaining wall by reducing the effective depth of the excavation.

The degree of support provided by a berm will depend on the ground conditions, the height of the berm (H), the width of the bench of the berm (B) and the angle of the berm slope (S) (**Figure 65.10**).

The slope of the berm will be governed by the soil and groundwater conditions, and for soils of low permeability, will depend on the duration for which the berm is to be relied upon

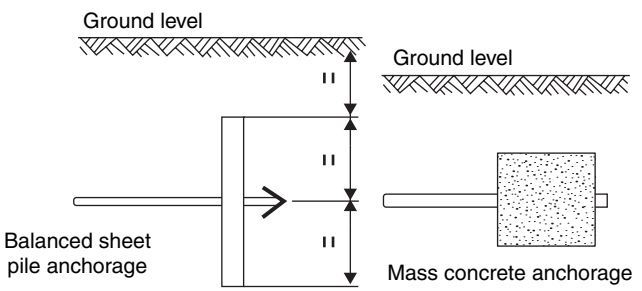


Figure 65.6 Deadman anchors
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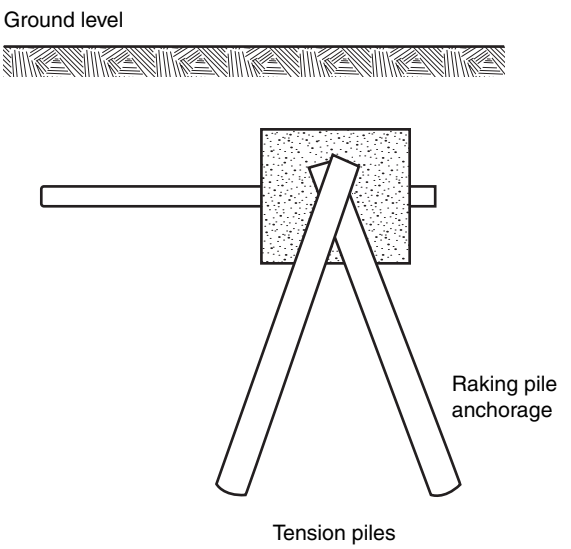


Figure 65.8 Ties and raking pile anchorage
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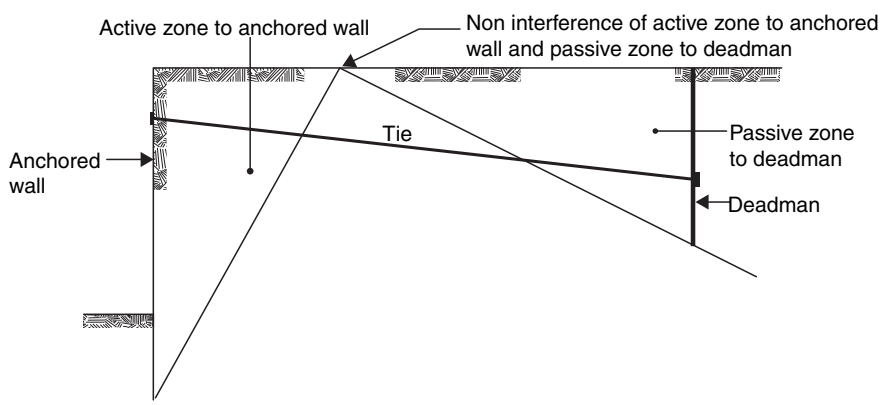


Figure 65.7 Non interference zone for anchors
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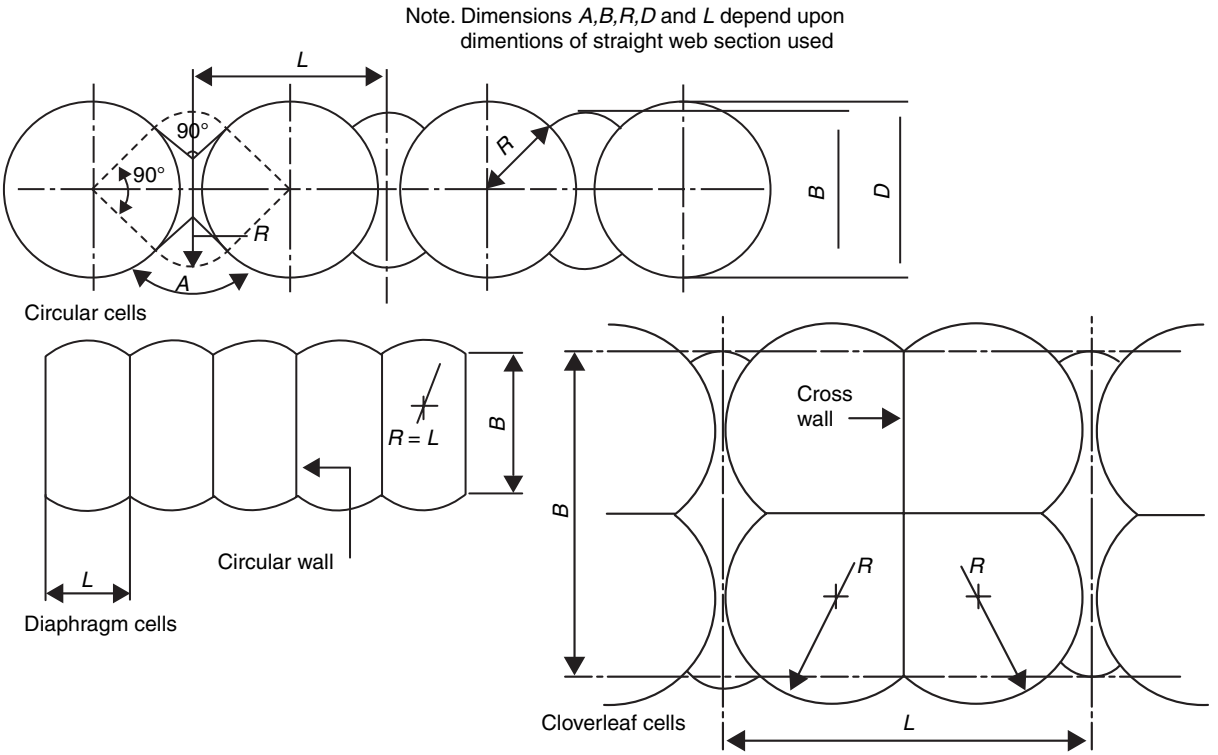


Figure 65.9 Types of cellular cofferdam
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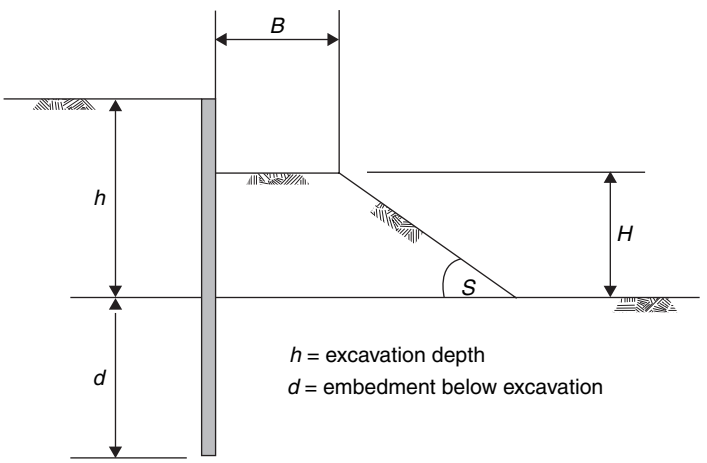


Figure 65.10 Definition of berm geometry
Reproduced with permission from CIRIA C580, Gaba *et al.* (2003), www.ciria.org

to support the wall. The height of the berm, bench width and also the slope of the berm will be influenced by the practical considerations of space and access. As such, the design of a berm will need to seek a balance between providing one of a suitably large size to provide meaningful support to the excavation, and yet be suitably small to minimise the restrictions on space and access within the excavation.

CIRIA C580 (Gaba *et al.*, 2003) comments that most methods of representing the effects of a berm in a limit equilibrium or soil-structure analysis, using sub-grade reaction and pseudo finite element methods, are semi-empirical and that this difficulty of analysis may explain why berms have often been used in conjunction with the observational method (Chapter 100 *Observational method*).

Several methods are commonly used to represent an earth berm in limit-equilibrium or soil-structure interaction analysis methods. CIRIA C580 (Gaba *et al.*, 2003) recommends the adoption of a raised effective formation level (after Fleming *et al.*, 1994) for routine analyses.

The raised effective formation level proposed by Fleming considers the berm as having a slope of 1:3 without any bench and with the full width of the berm. The effective formation level to be used in design analyses is determined as half of the reduced berm height which is equivalent to one sixth ($b/6$) of the full berm width. The benefits of berm structure beyond the assumed 1:3 berm profile and the effective formation level (shown as shaded in **Figure 65.11**) are incorporated into the analyses as a surcharge representing the weight of this additional berm structure applied at the effective formation level.

For information on other methods of modelling earth berms for consideration of wall stability or wall deflections, refer to CIRIA C580 (Gaba *et al.*, 2003, chapter 7).

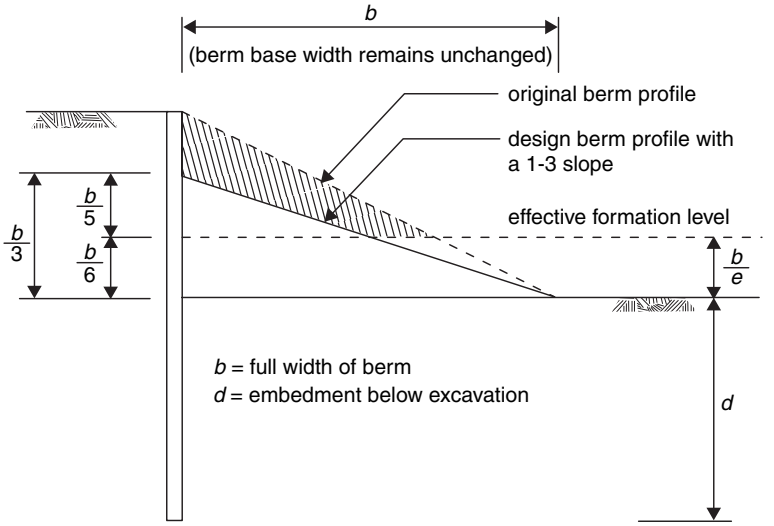


Figure 65.11 Representation of a berm by means of a raised effective formation level
After Fleming *et al.* (1994). Reproduced with permission from CIRIA C580, Gaba *et al.* (2003), www.ciria.org

65.7 Other systems of wall support

65.7.1 Circular construction

For circular excavations where the wall is not suitable to provide sufficient support through hoop action, support is typically provided in the form of a continuous ring beam. Due to the shape, a reinforced concrete ring beam is typically adopted.

For circular excavations, BS8002 (1994) identifies that earth pressures should be calculated as for straight-sided excavations; refer to Chapter 64 *Geotechnical design of retaining walls*.

In practice, the profile of the excavation and ring beam will deviate from a true circle and checks should be made for buckling in the ring beam with a radial waling load, W (kN/m), which is dependent upon the Young's modulus of the waling material, E (N/mm^2) and the moment of inertia about the x–x axis, I (cm^4) and the radius of the circular excavation, R (m), and is defined by:

$$W = 1.5\frac{EI}{R^3 \times 10^5}. \qquad \text{(BS 8002:1994, Equation 34.)}$$

65.7.2 Buttressed walls

The design of masonry walls to resist horizontal loading is covered by BS 5628 Parts 1 and 2. Buttresses act to improve the internal stability of the masonry wall structure as well as to increase the effective width of the wall footing (**Figure 65.12**). As such, they improve the ability of the walls to transmit horizontal pressures acting on the back of the wall down to the foundation level, where sliding resistance is provided through friction on the underside of the wall, and overturning resistance is provided through eccentric bearing resistance on the underside of the wall footing.

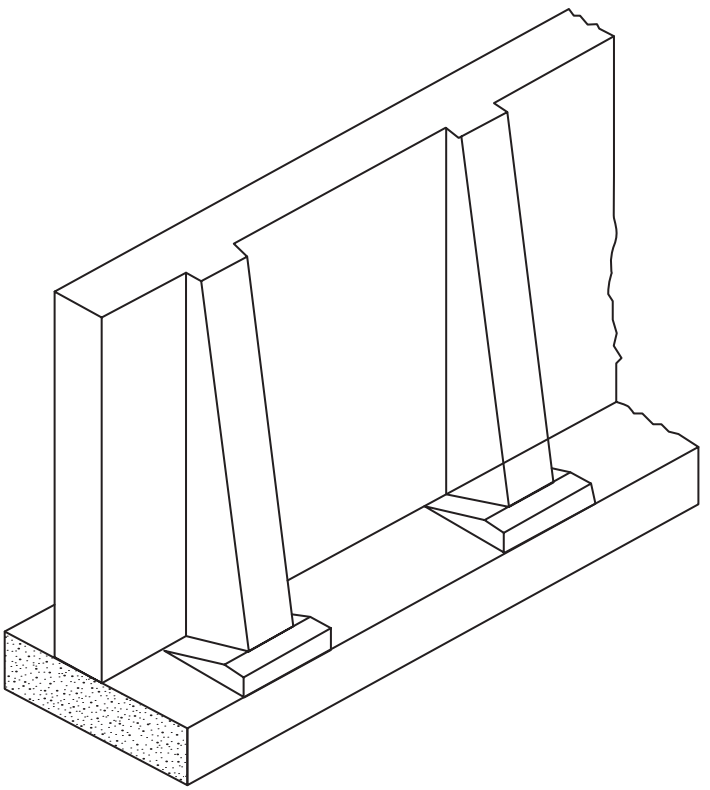


Figure 65.12 Buttressed masonry retaining wall on concrete footing
Reproduced with permission from BS 8002 © British Standards Institute (1994)

65.7.3 Ground improvement

Ground improvement methods such as soil mixing, jet grouting and crosswalls act by increasing the lateral support to an embedded retaining wall; they are more specialised than the

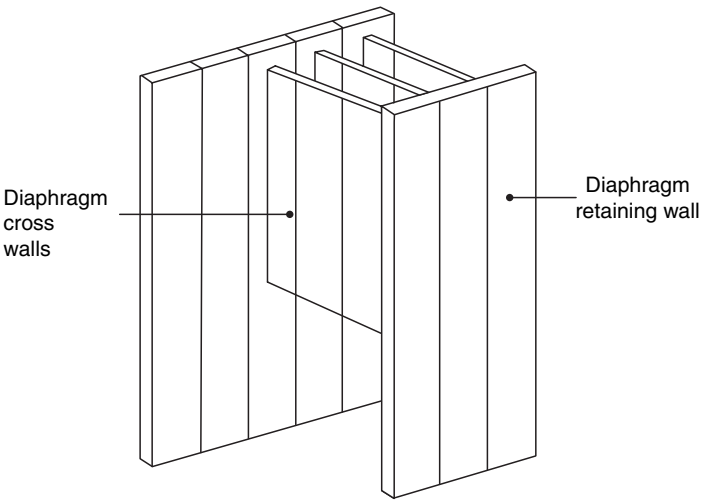


Figure 65.13 Use of crosswalls

other techniques discussed in this chapter (**Figure 65.13**). Such methods are generally only considered for excavation within soft soils where the benefit of improving the ground before excavation can outweigh the costs and time required for such ground improvement.

Guidance on the design and implementation of deep soil mixing and jet grouting is included in the following publications:

- CIRIA C573 (Mitchell and Jardine, 2002);
- EuroSoilStab Design Guide (2002);
- BS EN 14679 (2005a); and
- BS EN 12716 (2001).

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Mitchell, J. M. and Jardine, F. M. (2002). *A Guide to Ground Treatment*. London, UK: CIRIA, Publication C573.

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Williams, B. P. and Waite, D. (1993). *The Design and Construction of Sheet Piled Cofferdams*. London, UK: CIRIA, Special Publication SP95.

65.8.1 Further reading

Geoguide 1 (1993). *Guide to Retaining Wall Design*. Hong Kong: Hong Kong Government Geotechnical Control Office.

65.8.2 Useful websites

Construction Industry Research and Information Association (CIRIA); www.ciria.org

Hong Kong GEO Publications; www.cedd.gov.hk/eng/publications/manuals/geo_publications.htm

Piling Handbook; www.arcelormittal.com/sheetpiling/page/index/name/arcelor-piling-handbook

It is recommended this chapter is read in conjunction with

- Chapter 67 *Retaining walls as part of complete underground structure*
- Chapter 85 *Embedded walls*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 66

Geotechnical design of ground anchors

Michael Turner Applied Geotechnical Engineering Limited, Steeple Claydon, UK

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Ground anchors (also sometimes referred to as ground anchorages) are used to provide restraint to all forms of retaining structures, including embedded retaining walls, gravity walls (especially when such structures are being repaired, modified or upgraded) and some forms of hybrid wall, where additional lateral restraint is required.

In the wider civil engineering field, ground anchors are used to resist uplift, tension or other destabilising forces. These include structures subject to hydrostatic uplift or flotation such as dock structures or stormwater tanks and tension loads such as those exerted by transmission towers and masts, suspension bridges and tension roof structures. In addition, ground anchors are very commonly employed in the stabilisation of soil and rock slopes, landslides and cliff stabilisation works.

This chapter deals with the design of ground anchors as part of the restraint system to retaining walls.

It also covers general features concerning temporary and permanent anchors, load transfer mechanisms, tendon design and selection and overall stability considerations. These are applicable to ground-anchor design for other structural uses.

66.1 Introduction

This chapter should be read in conjunction with Chapter 89 *Ground anchors construction*, because the design aspects of ground anchors are intimately linked with their construction.

66.1.1 Definitions

66.1.1.1 BS EN 1537:2000

The execution standard for ground anchors, BS EN 1537:2000, defines an anchor as an installation capable of transmitting an applied tensile load to a load-bearing stratum. Such an anchor consists of an 'anchor head', a 'free anchor length' and a 'fixed anchor length', which is bonded to the ground by grout. The term 'ground' is taken to include both soil and rock. Ground anchors are normally pre-loaded to a predetermined service load. Such loads are applied to the anchor by tensioning the anchor tendon at the anchor head, usually by means of a hydraulic jack. The applied load is transferred from the anchor head, along the tendon, through the debonded free tendon length and into the ground through the fixed anchor zone.

The anchor head transfers the applied anchor force onto the structure, the fixed anchor length transfers the anchor force into the ground, and the free anchor length spans the gap between the anchor head and the fixed anchor length.

For reference, these terms are illustrated in **Figure 66.1**.

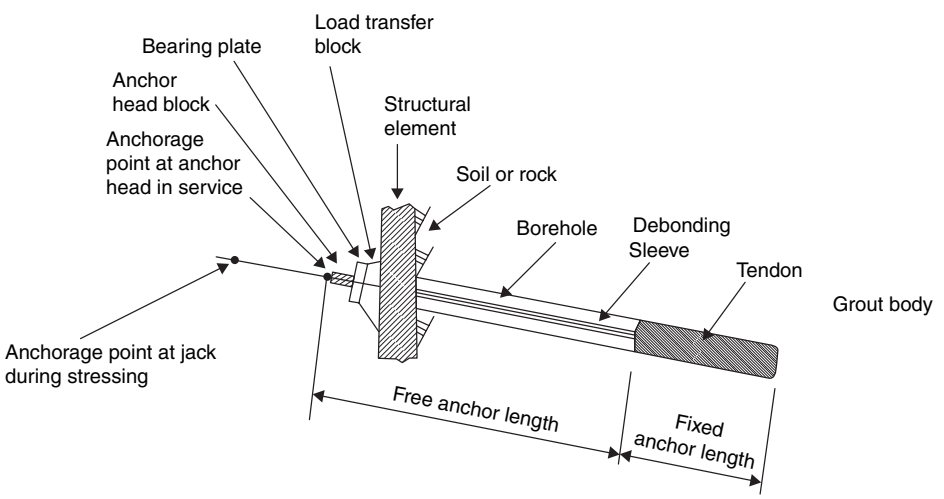


Figure 66.1 Sketch of a typical ground anchor
Reproduced with permission from BS EN 1537 © British Standards Institution 2000 (as published in CIRIA C580)

66.1.1.2 Temporary anchors

The execution standard defines a temporary anchor as one which is required to be in service for no more than two years and a permanent anchor as having a design life in excess of two years.

The execution standard requires that for a 'temporary' anchor the steel components of the anchor shall be provided with protection that will inhibit or prevent corrosion over a minimum design life of two years. To this end, the standard requires that as a minimum the exposed steel in the tendon bond length should be provided with a minimum grout cover of 10 mm from the borehole wall. In aggressive ground conditions, protection might need to be enhanced by use of a single corrugated-plastic duct.

EN1537 also draws attention to the fact that the least-protected zone of an anchorage defines the class of protection provided. Thus, attention to the anchor head and to joints and boundaries within the system are important, if not paramount, to the integrity of the anchor tendon in terms of assuring its level of corrosion protection. This requirement also applies to temporary anchors.

EN1537 also states that the two-year design life for a temporary anchor is mandatory. If the service life exceeds this, the anchor should be considered 'permanent'.

66.1.1.3 Permanent anchors

For a 'permanent' anchor EN1537 requires that corrosion protection to the steel tendon(s) should consist of at least a single continuous layer of corrosion protective material that will not degrade during the design life of the anchor. Acceptable systems within EN1537 (Cl. 6.9.3) are identified as those with:

(a) Two protective barriers to corrosion such that if one barrier is damaged during installation or anchor loading, the second barrier remains intact.

(b) A single protective barrier to corrosion, the integrity of which shall be proven by testing each anchor *in situ*. Such testing consists of some form of electrical resistance testing to demonstrate that the anchor tendon is electrically isolated from the ground and the structure.

(c) A corrosion protection system provided by a steel duct tube-a-manchette anchor.

(d) A corrosion protection system provided by a corrugated plastic duct tube-a-manchette anchor.

(e) A corrosion protection system provided by a steel duct compression tube anchor.

Examples of corrosion protection that may be considered to satisfy the above principles of protection for permanent anchors are described in Table 3 of EN1537.

66.1.2 Nomenclature

In the UK, currently the most common protective systems comply with Type (a) as described above. At the current time of writing, systems using Types (c) or (d) have also been used on some projects in the UK, but are less common.

Both EN1537 and the now partially superseded UK code BS8081 (BSI, 1989) pay attention to, and attempt to codify, the nomenclature of ground anchorages and techniques. In this regard, for example, **Figure 66.2(a)** illustrates the nomenclature and major features of a typical temporary anchor and **Figure 66.2(b)** shows a typical permanent anchor as described in (a) above.

As illustrated in **Figure 66.2(b)**, a typical permanent anchor in the UK will consist of an encapsulated tendon bond length (which transfers the anchorage force into the fixed anchor length), a free tendon length and an anchor head. It will be noted that the fixed anchor length is not necessarily the same as either the tendon bond length or the encapsulation bond length. Correspondingly, the free anchor length is not necessarily the same as the free tendon length. This differentiation is intentional within EN1537 and BS8081, and allows flexibility of design by the designer.

In the case of a temporary anchor the tendon bond length and the fixed anchor length are usually, but not always, the same.

66.1.3 Classification of ground anchors

As detailed in Chapter 89 *Ground anchors construction* the sequence of constructing a ground anchor is usually:

- drill;
- install the tendon;
- place grout (extracting temporary drill casing if used).

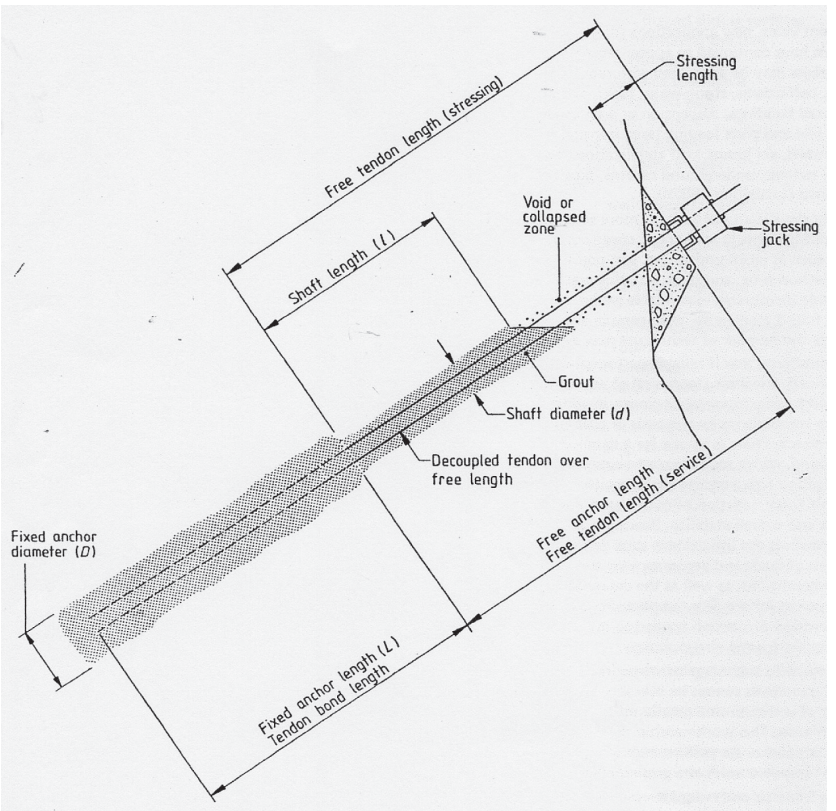
Whilst it can be appreciated that the drilling method and technique can affect the magnitude of the grout-to-ground bond that can be mobilised, it is generally considered that the method of grouting has the most influence on the load-carrying capacity of a grout anchor (and similarly for micro-piles: as noted in FHWA, 1997, for example).

BS8081 (BSI, 1989) proposed a general classification of cement-grouted ground anchors into four groups, based primarily upon the type and pressure of the grouting. BS8081 termed these as Types A to D anchors, as described below and illustrated in **Figure 66.3**.

- Type A anchor: Grout is placed in the borehole under gravity head only, in straight-shafted boreholes, which may be temporarily lined or unlined, depending on hole stability.
- Type B anchor: Grout is injected into the borehole under low pressure, typically of less than 10 bar (1000 kN/m^2), using temporary or permanent casing, or an *in situ* packer at the top of the fixed anchor.
- Type C anchor: Grout is injected into the borehole under a high pressure, typically in excess of 20 bar (2000 kN/m^2) using a lining tube or *in situ* packer. Typically the injection process uses a tube-a-manchette system, or similar, which allows multiple phased injections if required.

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(a)



(b)

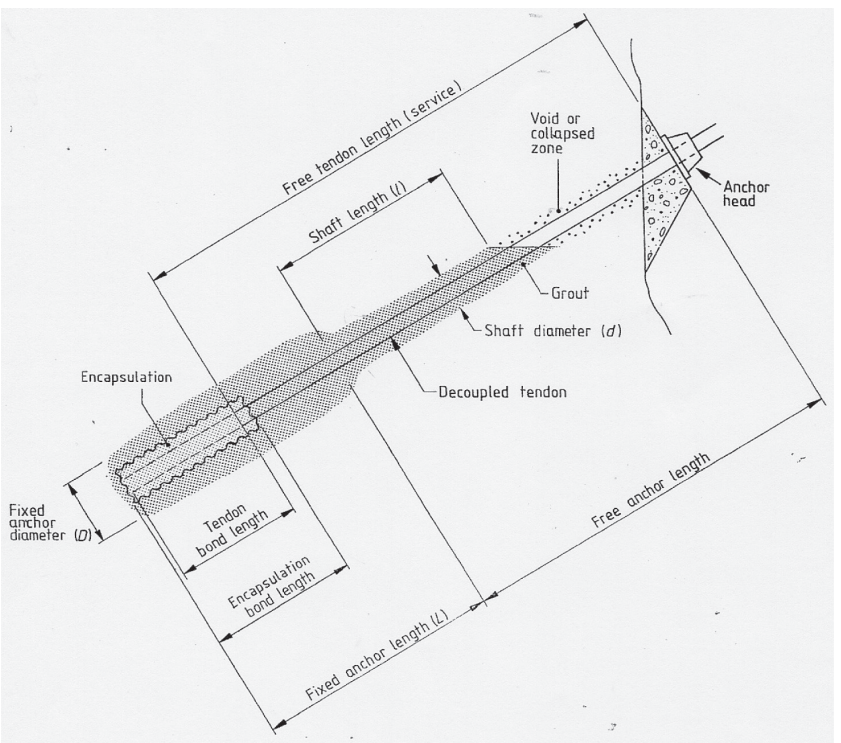


Figure 66.2 Typical ground anchor nomenclature: (a) temporary ground anchor; (b) permanent ground anchor

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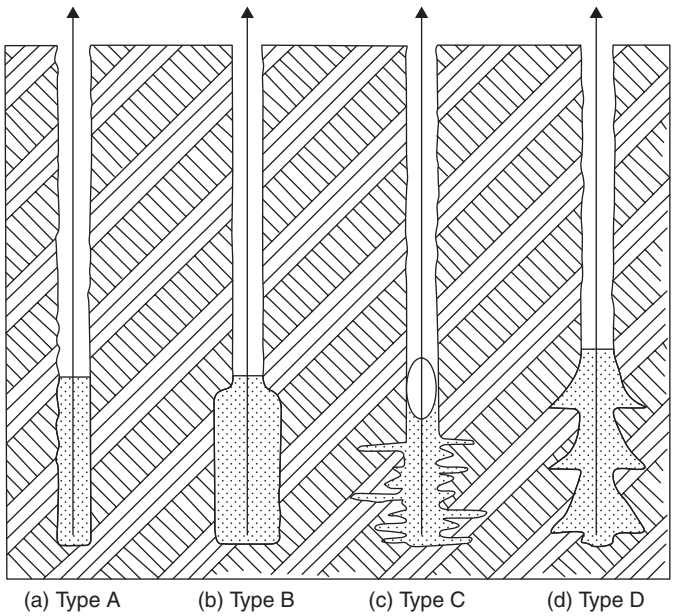


Figure 66.3 Classification of ground anchor Types A, B, C and D
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- Type D anchor: Grout is placed under gravity head only, but the borehole is formed with a series of enlargement: either bells or under-reams.

Further reference to these four groups is made in Chapter 89 *Ground anchors construction*.

It is worth noting that European practice identifies two techniques of forming anchors that would fall into the category of Type C anchors. In French practice, these are termed, respectively, IGU techniques, where only a single 'global' high-pressure injection is undertaken through a non-reusable packer system, or IRS techniques, where repeated injections at specific horizons can be undertaken if necessary.

In addition since the publication of BS8081:1989 and EN1537:2000 there has been a rapid growth in hollow-core 'self-drilling' systems. With such systems the hollow bar is fitted with a drill bit and is used as both the drill rod and the anchor tendon. Grout is pumped down the hollow rods to exit out of the drill bit, either during or after drilling to the required depth. These can be equated to either BS8081 Type A or Type B anchors depending on their details of use.

66.2 Review of design responsibilities

EN1537 emphasises that the planning and design of ground anchors, together with their installation and testing, all require experience and knowledge of this specialised field. Additionally, in common with other such specialist operations, the technique requires skilled and qualified operatives and supervision in the field.

The design responsibilities for works involving the installation of ground anchors can often be spread among several parties, including the client, the overall project designer and specialist designers working for one or more of the contractors. It is important, therefore, that the responsibilities of all parties involved in the design, execution and maintenance of such ground anchors are properly identified and defined.

EN1537 includes a guide identifying an appropriate separation of 'design' and 'execution' activities, a copy of which is attached as **Table 66.1**.

From **Table 66.1**, the detailed 'design' of a ground anchor falls within the ambit of Activities 1 to 4 of the specialist execution activities. In addition, other aspects of anchor design are included within the overall design activities. In the context of anchors supporting a retaining wall, EN1537 identifies two different roles:

(a) The wall designer, who is responsible for the design of the retaining wall itself, and who would expect, or would be expected, to identify not only the requirement for ground anchors to support the structure, but also the details of individual anchor loads, their spacing and orientation, and their minimum length to ensure the stability of the structure (although, in practice, these design decisions might well be undertaken in conjunction with the anchor designer).

(b) The anchor designer, who is responsible for the selection and design of the anchor type and its component parts, and who is also responsible for the overall design of the ground anchor system, taking full account of the ground conditions anticipated at the site.

The following text centres upon the goals identified in (b) above. It should be borne in mind, however, that it is perfectly possible (and not unusual) for the wall designer to identify the magnitude of the horizontal restraint required to be provided to the wall, and its level or levels of action, leaving the anchor designer to determine the detail of individual anchor loads and their spacing and orientation. It should remain the wall designer's responsibility, however, to identify a minimum free anchor length or the extent of the zone beyond which the designed fixed anchor length must be placed, in order to ensure the stability of the structure. This is on the basis that only the wall designer is in a position to identify the full range and combination of forces and effects touching upon the design, construction and performance of the retaining structure.

The following text therefore assumes that the overall design activities 1 to 13 in **Table 66.1** are undertaken by the wall designer, albeit in possible conjunction with the anchor designer in an advisory or support capacity. The anchor designer in turn is responsible for specialist execution activities 1 to 4, where they affect 'design' matters. The assessment of site investigation data, for instance, is not limited to a 'design' issue, but is also a necessary part of the execution phase in determining the means to be adopted to install an anchor.

Overall design activities	Specialist execution activities
1. Provision of site investigation data for construction of ground anchors	1. Assessment of site investigation data with respect to design assumptions
2. Decision to use ground anchors, required trials and testing and provision of a specification	2. Selection of ground anchor components and details
3. Acquisition of legal authorisation and entitlement to encroach on third-party property	3. Determination of fixed anchor dimensions
4. Overall design of anchored structure, calculations of anchor force required. Definition of safety factors to be employed	4. Detailing of the corrosion protection system for the ground anchor
5. Definition of ground anchor life (permanent/temporary) and requirement for corrosion protection	5. Supply and installation of the ground anchor system
6. Specification of anchor spacing and orientation, anchor loads and overall stability requirements	6. Supply and installation of the ground anchor monitoring system
7. Specification of minimum distance from the structure to mid fixed length to ensure stability of the structure	7. Quality control of works
8. Specification of load transfer mechanism from the anchor to the structure	8. Execution and assessment of anchor tests
9. Specification of any sequence of anchor loading required of the structure and the appropriate load levels	9. Evaluation of on-site anchor tests
10. Specification of systems for monitoring ground-anchor behaviour and for interpretation of results	10. Maintenance of ground anchor as directed
11. Supervision of the works	
12. Specification of maintenance for ground anchors	
13. Instruction to all parties involved of key items in the design philosophy to which special attention should be directed	

Table 66.1 Design and execution activities

As examples of the above division of the duties undertaken by the various parties responsible for design, it is not uncommon in a project containing an anchored retaining wall that the design responsibilities are divided in one of the following ways:

(a) A project designer employed by either the client or the principal or main contractor will identify the need for such a retaining structure within the overall project scheme. A specialist foundation engineering contractor may be employed by the principal contractor, or directly by the client, to be responsible for the design and installation of the retaining wall, which would typically be an embedded retaining wall. The foundation engineering contractor may, in turn, sub-contract the design and execution of the ground anchors to a specialist geotechnical contractor, who may also design the anchors, or who may, in turn, employ a specialist consultant to design them on their behalf.

(b) Alternatively, the project designer may undertake the design and specification of the retaining wall, and also identify the need for ground anchors as part of the applied restraint to the wall at levels and spacings identified by the project designer. The construction of the retaining wall might be undertaken by a foundation engineering contractor to a specification prepared by the project designer, with the ground anchors being designed and installed by the foundation engineering contractor, or by an anchoring specialist. The anchors must provide the loadings specified by the project designer.

It will be appreciated that there are many such possible variations of the division of responsibility for the design and execution of the anchored structure, which underscores the necessity of ensuring that those responsibilities are clearly understood and accepted by all parties.

66.3 The design of ground anchors for the support of retaining walls

66.3.1 Introduction

As an essential precursor to design, a substantial retaining wall, requiring the use of ground anchors for support, should be accompanied by an adequate ground investigation.

It is often forgotten, or not recognised, that such ground anchors may extend beyond the confines or ownership of the site, and that in such circumstances any ground investigation should be expected to extend beyond the site boundaries. In addition, questions of wayleaves or permissions to install the anchors beyond the site boundary have to be addressed.

As a principle, any such site investigation should ideally extend beyond and below the anticipated limits of the anchors. Where this is not possible, or practicable, it should

be understood that the design involves additional risk by extrapolating geotechnical and geological data beyond known boundaries in order to formulate a predicted ground model.

Whilst the requirement for adequate ground information is similar in essence to the requirements that would be expected for the ground investigation for the design of the wall itself, it is not unusual in the design process that the need for a retaining structure is identified at some time before the requirement for ground anchors emerges. It might be only at this latter stage that the need for ground information at some distance behind the wall becomes clear, and this might have to be obtained as a second investigation phase.

The ground information should be sufficient to allow the anchor designer to understand the range, sequence and properties of the strata behind the retaining wall into which, or through which, the anchors will be installed. It should allow the anchor designer to illustrate this understanding with adequate and sufficient cross-sections through the wall and its attendant ground anchors. This appreciation of the 3D geological picture, including groundwater information, and the potential variation across the site, is vital to the successful design and execution of the anchors, as it is, indeed, for any other geotechnical design.

It also has to be appreciated, however, that the scope of the ground investigation may well depend to some extent upon the size and type of the project, and the practicalities of gaining access for an adequate investigation. It also may depend upon a degree of local knowledge and experience of ground conditions. It is possible and permissible, of course, to take an informed view of the degree of risk and uncertainty implied by the relative lack of ground information when undertaking design and construction. At the least, however, this must imply that the robustness of the ground model developed by the anchor designer must be adequate and flexible for the task in hand, and any potential shortcomings are sufficiently understood and capable of being addressed within the design and execution of the anchors.

66.3.2 Matters that have to be assessed at the design stage of the retaining wall.

In the context of the above discussion, it is probably useful at this stage to step back and review how these matters equate within the framework of the activities outlined in **Table 66.1**. With reference to the design activities, Activities 1 to 5 encompass both the site investigation phase and the design of the retaining wall, which will have led to the decision to use ground anchors, including decisions on their design life.

The next phase of the design, Activity 6, is to review the anticipated layout and 'sizing' of the anchor system in relation to the structural design of the wall and the ability of the ground within any potential fixed anchor length zone to transfer the projected design loads, and the ability of the wall to accept them. An example would be the consideration of whether it was preferable to use a large number of lower capacity anchors, or to adopt a smaller number of anchors with higher individual loads. As a guide, at this stage, for a typical 'average' embedded retaining wall, ground-anchor design working loads would typically be between 200 and 600 kN, with a mean typically in the range of 300 to 400 kN, and with horizontal and vertical anchor spacings of between 2.0 and 4.0 m. Designs requiring anchors outside this range would often be concerned with larger, deeper retaining walls and more arduous conditions.

Anchor inclinations typically vary between 20° and 45° below the horizontal, although inclinations outside these limits are not uncommon. Both the wall designer and the anchor designer need to bear in mind that the more steeply an anchor is inclined, the greater is the vertical component of the anchor force exerted on the wall. This can be particularly relevant for steel sheet piles, with their relatively low cross-sectional area.

It is not unusual for an anchored retaining wall to have corners or bends, to suit site constraints, ownership or the like. When corners become re-entrant (i.e. at an angle of greater than 180° relative to the site), there is an increasing tendency for anchors to clash with one another. A typical situation is illustrated in **Figure 66.4**. Measures have to be identified during design to ensure in such circumstances that anchors are positively designed to avoid intersecting one another, and that sufficient clearance is left between passing anchors. This usually has to be done by varying both the inclination and skew (horizontal angle to the wall) for some or all of the affected anchors. Even relatively simple intersection problems often become sufficiently complex that they are best solved by simple modelling of the layout, either by computer or by constructing a physical model.

After Activities 6 to 8, with adequate ground information in place, and ideas of anchor size having been developed, the detailed design of individual anchors can be undertaken.

66.3.3 Notes on the use of BS8081

It is intended that EN1537 should take precedence and eventually replace the UK ground anchorage code BS8081 from 2010. However, BS8081 contains much advice and guidance on best practice and the development and use of design parameters, particularly in terms of bond values for the various interfaces involved in ground-anchor design. Hence, for the purposes of this current document, BS8081 has been identified as a source reference for much of the basic design features of ground anchors. It should be understood, however, that where there is any potential conflict between any recommendations or advice within BS8081 and BS EN1997–1 (EC7–1) and EN1537, the latter two documents would normally take precedence. A feature of EN1537, in particular, is that many of its requirements are mandatory principles, *which have to be followed*, rather than recommendations or guidance.

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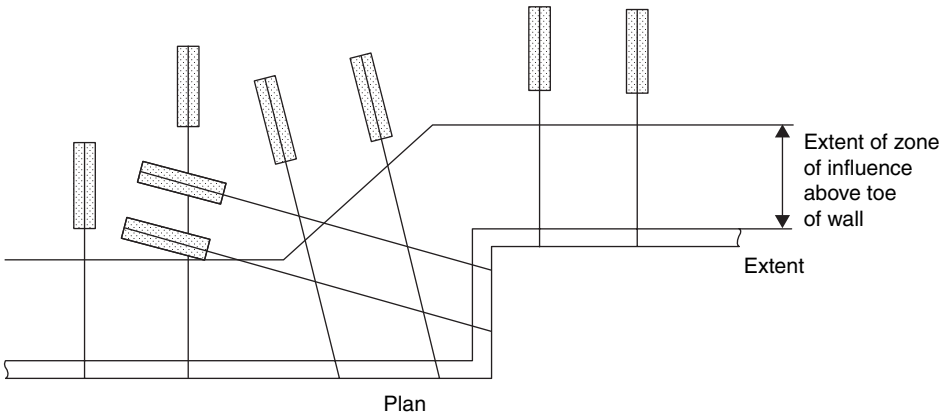


Figure 66.4 Typical intersection of anchors at re-entrant corners
Reproduced with permission from BS8081 © British Standards Institution 1989

66.4 Detailed design of ground anchors

66.4.1 Introduction

Four main components of design have to be considered by the anchor designer:

(1) load transfer into the ground;

(2) the type and strength of the tendon;

(3) the location of the fixed anchor zone, together with uplift or pull-out capacity;

(4) load transfer into the structure.

It can be seen that items (1) and (3) depend upon a knowledge of the characteristics of the ground, as revealed by the ground investigation.

66.4.1.1 Factors of safety

Ground-anchor design in the UK has traditionally been based on a global factor of safety or permissible stress.

The introduction of Eurocodes and, in particular, Eurocode EC7 (Geotechnical Design; BSI, 2004) requires design to be undertaken on a partial factor limit-state design basis. At the time of writing all of the 'wrinkles' associated with the adoption of this design philosophy have not been finally satisfactorily addressed and ironed out, particularly for ground-anchor design (see Bond and Harris, 2008, for example).

At present, the best advice would seem to be to retain the BS8081 recommendations for global factors of safety, as summarised in **Table 66.2**.

In such designs:

$$T_{\text{design}} = \frac{T_{\text{ult}}}{F}$$

where T_{design} is the design working load of the anchor, T_{ult} is the ultimate load capacity of the anchor by calculation or test and F is the 'global' factor of safety.

66.4.2 Load transfer into the ground

The load transfer between the ground anchor tendon and the soil or rock within which it is embedded depends upon the design of three boundaries:

- anchorage grout to ground (fixed anchor length);
- encapsulation to anchorage grout, if appropriate (encapsulation length);
- tendon to anchorage grout or encapsulation grout (tendon bond length).

66.4.2.1 Bond between anchorage grout and the ground (fixed anchor length)

General

The relationship between the design anchorage force, T_{design}, and the required fixed anchor length, L, is usually determined by a skin friction equation of the form:

$$T_{\text{design}} = \pi\, d\, L\, \tau_{\text{design}} \qquad (66.1)$$

where d is the diameter of the borehole and τ_{design} is the design value for the bond or skin friction at the grout–ground interface. It is not usual that an end-bearing component at the proximal end of the fixed anchor length, due for instance to a possible enlargement of the fixed anchor diameter by pressure grouting, is included in the design. However, some design techniques do attempt to take some account of this aspect, as will be discussed below. Under-reamed anchors in granular soils, however, usually take some account of such end-bearing.

Values for τ_{design} are derived from a wide variety of sources, such as pile-related bearing capacity formulae, back analysis of pull-out tests on specially constructed test anchors and the like.

With regards to design parameters, it is quite important to understand that ground anchors, together with, micropiles, 'conventional' piles, soil nails and other similar inserts, all have design and construction aspects in common. In particular, anchors and micropiles are usually quite similar dimensionally

Anchorage category	Minimum safety factor			
	Tendon	Ground–grout interface	Grout–tendon or grout–encapsulation interface	Proof load factor
Temporary anchorages where the service life is less than six months and a failure would have no serious consequences and would not endanger public safety, e.g. short-term pile test loading using anchorages as a reaction system	1.40	2.0	2.0	1.10
Temporary anchorages with a service life of say up to two years where, although the consequences of failure are quite serious, there is no danger to public safety without adequate warning, e.g. retaining wall tie-back	1.60	2.5 (1)	2.5 (1)	1.25
Permanent anchorages and temporary anchorages where corrosion risk is high or the consequences of failure are serious, e.g. main cables of a suspension bridge or as a reaction for lifting heavy structural members	2.00	3.0 (2)	3.0 (1)	1.50

[1] Minimum value of 2.0 may be used if full-scale field tests are available.

[2] May need to be raised to 4.0 to limit ground creep.

In current practice the safety factor of an anchorage is the ratio of the ultimate load to design load. This table defines minimum safety factors at all the major component interfaces of an anchorage system.
Minimum safety factors for the ground–grout interface generally lie between 2.5 and 4.0. However, it is permissible to vary these, should full-scale field tests (trial anchorage tests) provide sufficient additional information to permit a reduction.
The safety factors applied to the ground–grout interface are invariably higher compared with the tendon values, the additional magnitude representing a margin of uncertainty.

Table 66.2 Minimum safety factors recommended for design of individual anchors
Reproduced from BS8081 © British Standards Institution 1989

and in load capacity terms, and the design techniques and values obtained from field testing are often essentially interchangeable between the two, with appropriate allowances.

Similarly micropiles and conventional piles have common design and performance aspects. With regards to bond design, however, there are often considerable differences in size and construction technique that have to be factored into any comparisons on performance.

The load-carrying capacity of anchors in suitable soils or weak rocks can also be enhanced by under-reaming or by post-grouting techniques. In the latter technique, additional grout is injected into the fixed anchor length at high pressure after the initial anchorage grout has set.

In addition, higher anchor loads can often be developed using 'multi-stage' anchor tendons (also known by the proprietary name of Single Bore Multiple Anchors), a technique whereby a number of essentially single-strand anchors are placed within the same borehole. Each strand anchor is designed to transfer load over a separate section of the anchor hole. Hence, with such anchors, it can be seen that the free tendon length of each strand is different, as illustrated in **Figure 66.5**.

For further details on such techniques and systems, see Chapter 89 *Ground anchors construction*.

Guidance on fixed anchor length design

Fixed anchor length design generally focuses on three convenient 'groups' of anchoring material: granular soils, clayey soils and rocks. It will be realised that there can be some blurring of the boundaries between these idealised groups, as intact rocks weather towards 'soils', for instance.

As a general rule (e.g. BS8081) the fixed anchor length should be:

(a) A maximum of 10 m long (because of a trend of decreasing capacity per metre run, as discerned from back analyses of load testing). However, multi-stage anchors are able to employ longer load transfer lengths by means of 'stacking' fixed anchor lengths, as described below.

(b) Not less than 3.0 m; although for design working loads below 200 kN, a minimum fixed anchor length of 2.0 m may be acceptable in suitable circumstances.

Fixed anchor length design in granular soils

A typical design relationship based upon an expression developed by Littlejohn (1970) is shown in equation (66.2) below. Note that, contrary to equation (66.1) and its accompanying note, equation (66.2) does include consideration of an end-bearing component. This is discussed further in the accompanying text.

$$T_{ult} = A\overline{\sigma'_v}\pi kdL\tan\phi' + B\sigma'_h\frac{\pi}{4}[(kd)^2 - d^2] \qquad (66.2)$$

(shaft friction) + (end bearing)

where T_{ult} is the ultimate load holding capacity; L is the length of the load transfer length (fixed anchor length) within the

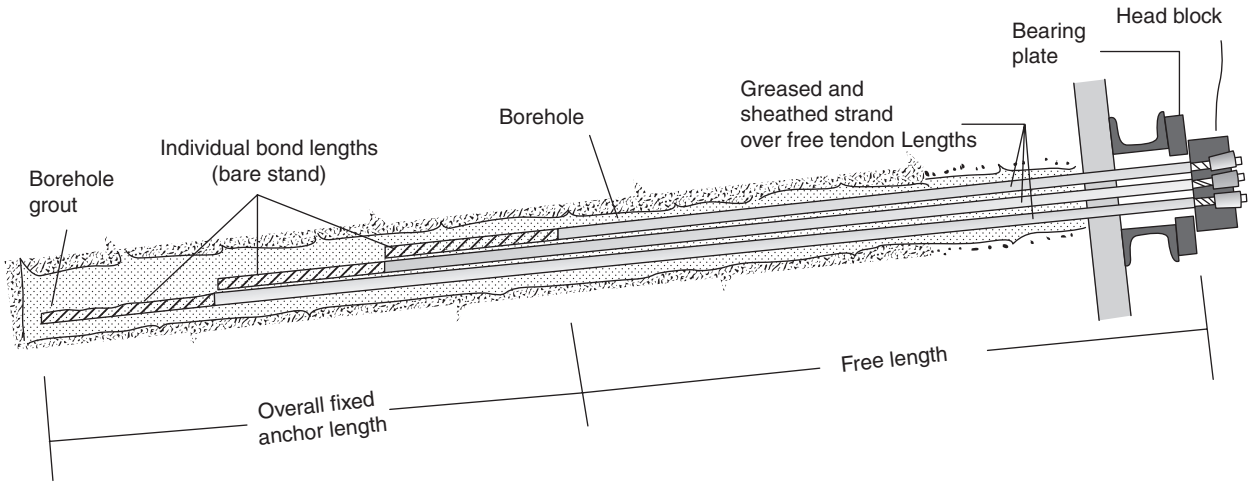


Figure 66.5 Characteristic features of a multi-stage ground anchor (temporary anchor shown)

stratum; d is the drillhole diameter; $\overline{\sigma'_v}$ is the average effective overburden pressure along the load transfer length; ϕ', is the angle of shearing resistance of the soil; A is a coefficient related to the contact pressure at the fixed anchor–soil interface and the average effective overburden pressure; k is a coefficient relating to the degree of enhancement of the diameter of the load transfer length from the grouting process and the penetration of the grout beyond the drillhole diameter; B is a bearing capacity factor; h is the depth of overburden to the top of the fixed anchor length; σ'_h is the effective overburden pressure at depth h.

This form of relationship is also described and discussed in BS8081 (Equation 3 of BS8081) and Littlejohn (1980).

Shaft friction

For BS8081 Type B anchors, values for k, the enhancement of the nominal drillhole diameter by penetration of the anchor grout into the surrounding ground, due to its fluidity and the pressure grouting process, have been determined by back analysis of pull-out tests. Derived values for k noted by Littlejohn (1980) were between 3 and 4 in coarse sands and gravels, perhaps 1.5 to 2 for medium dense sand and 1.2 to 1.5 for very dense sand.

Values for A depend to a large extent on the construction technique. Littlejohn (1980) derived values for A of 1.7 for compact sandy gravel ($\phi' = 40^o$) and $A = 1.4$ for compact dune sand ($\phi' = 35^o$). Other workers quoted in this reference derived similar values for A.

It can be seen that the product Ak derived from Littlejohn's work for compact sandy gravel is typically between 5.1 and 6.8. The same calculation for compact dune sand yields a range of between 1.7 and 2.1.

Littlejohn also quoted work in Sweden by Moller and Widing (1969), which reported on pull-out tests on ground anchors, where the authors had not tried to separate the varying influences of end-bearing and shaft friction (including enlargement or enhancement of the fixed anchor length). Moller and Widing's derived values for the product Ak (denoted in Littlejohn, 1980, as 'K_2') gave values varying between 4 and 9 in materials ranging from coarse silt and fine sand to sand and gravel, and with grout injection pressures of 3–6 bar. The lower values were for the finer-grained soils and the higher ones were obtained in coarser soils.

Back analysis by Turner (2010) of test anchors in coarse open gravels in Aberdeen produced similar values for K_2 of 9 or higher.

End bearing

With regard to the end-bearing component of equation (2), Littlejohn and his co-workers back-calculated values for the parameter B for compact sandy gravel as $B = 101$ and compact dune sand as $B = 31$. Further details can be obtained in the particular references cited above.

In practice, as noted earlier, the end-bearing component of equation (66.2) is usually ignored in design.

Summary

The above review suggests that load-carrying capacity in granular soils can typically be expressed by a relationship in the form of:

$$T_{ult} = K\pi dL\overline{\sigma'_y}\tan\phi' \qquad (66.3)$$

where 'K' is, effectively, the product of A (related to the increase of the radial effective stress around the fixed anchor length due to the grouting and installation process) and 'k' (a measure of the physical enhancement of the fixed anchor diameter by the pressure grouting process). It can be seen that this is very similar to the classic formula for calculating

the ultimate shaft friction of a pile in granular soils (see, for instance, Chapter 22 *Behaviour of single piles under vertical loads*).

In conventional pile design, typical values for K will vary between 0.7 and 0.9 for a bored pile, but there might also be a reduction in ϕ' to account for soil loosening. This range of K for bored piles can be compared with the much higher values obtained for ground anchors of between 4 and 9 by Moller and Wilding (1969) and 1.7 to 6.8 by Littlejohn (1980). (The higher Moller and Widing values might be explained because these results also include an element of the end-bearing component identified by Littlejohn.)

It can be envisaged therefore that the increase in K value exhibited by ground anchors in granular soils, compared with 'conventional' piles, can be attributed to:

(1) Scale differences between a typical pile diameter of perhaps 450 to 750 mm, compared with a ground anchor of 100 to 200 mm.

(2) Penetration of the fluid grout into the surrounding granular soil, so increasing the effective diameter of the fixed anchor length.

(3) The influence of grouting pressure on anchor capacity (by both permeation and by increasing the confining pressure around the fixed anchor length). The technique of pressure grouting includes, in order of increasing load capacity: grouting under hydrostatic head only (BS8081 Type A anchors), low-pressure grouting (less than hydro-fracture pressures) (BS8081 Type B anchors), or high-pressure post-grouted systems (using tube-a-manchette, or similar, techniques) (Type C anchors).

(4) The particle-size distribution of the granular strata: where coarser soils will, in general, permit a greater penetration of the anchor grout.

With respect to high-pressure post-grouted systems, empirical design curves for such techniques in sands and gravels were developed by Bustamante (1976) and Bustamante and Doix (1985) and are illustrated in **Figure 66.6**.

Fixed anchor length design in clayey soils

Straight-shafted anchors

A typical design relationship is expressed in the form of:

$$T_{ult} = \pi\, d\, L\, \alpha\, c_u \tag{66.4}$$

where c_u is the undrained shear strength; α is an adhesion factor; and d and L are as previously defined.

Again, it can be seen that this design approach is similar to that employed in bored-pile design practice.

As described in BS8081, design is almost always based upon undrained shear strength and total stress analysis rather than using the more fundamental concept of drained strength parameters and effective stress analysis. Drilling and grouting activities, usually using auger or water flush systems and fluid neat cement grouts, are difficult to model reliably using effective stress design. In addition, load-testing is of short duration

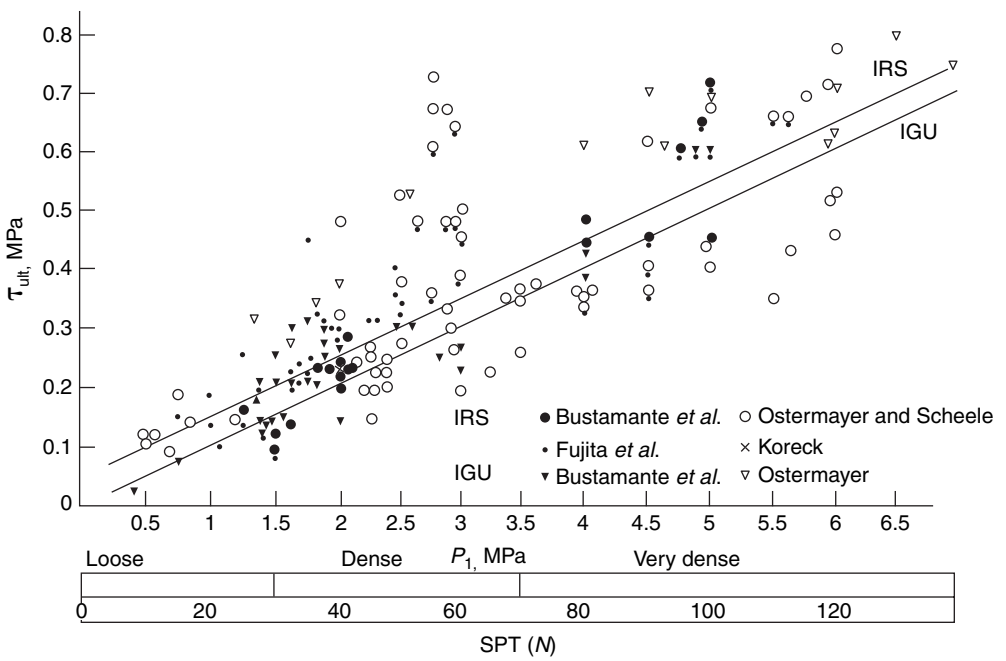


Figure 66.6 Empirical relationship for the determination of the ultimate skin friction for sand and gravel using IGU and IRP post-grouting techniques
Reproduced from Bustamante and Doix, 1985 (as published in FHWA, 1997)

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compared with the time for steady-state effective stress conditions to stabilise. Hence, the testing mode is more suited to a total stress approach.

It should be noted that increasingly in UK practice, anchor load testing involves not only investigating the ultimate 'short-term' load-holding capacity but also confirming acceptable creep values (see the Execution Standard EN 1537:2000 for more details).

For straight-shafted tremie grouted anchors with low or no pressure, quoted α values in BS8081 for stiff London Clay ($c_u > 90$ kPa) are typically around 0.3–0.35. For very stiff to hard Mercia Mudstone ($c_u = 287$ kPa) a design value of $\alpha = 0.45$ is suggested.

For multi-stage anchors, each strand is deemed to have its own short individual load transfer length within the overall fixed anchor length. Pull-out test data suggests this allows the use of markedly higher α values for design; values as high as 0.95 or 1.0 have been cited by Barley (1997).

Under-reamed anchors

Where anchors are constructed using multiple under-reaming techniques (BS8081 Type D anchors), as described in Chapter 89 *Ground anchors construction*, failure of the fixed anchor zone can be visualised as occurring through the outer edges of the bells within undisturbed material, together with the contribution from end-bearing and any non-under-reamed section of the shaft, as illustrated in **Figure 66.7**. A typical design relationship for such anchors would be of the form of:

$$T_{ult} = \pi\, DL\, c_u + \pi/4\, (D^2 - d^2)\, N_c\, c_{ub} + \pi\, dl\, c_a \qquad (66.5)$$

(side shear) + (end bearing) + (shaft resistance)

where D is the diameter of the under-ream (in metres); L is the length of the under-reamed section of the fixed anchor (in metres); c_u is the average undrained shear strength over the fixed anchor length (in kN/m^2); d is the diameter of the shaft (in metres); N_c is the bearing capacity factor (a value of 9 is commonly assumed); c_{ub} is the undrained shear strength at the proximal end of the fixed anchor (in kN m^2); l is the length of the shaft (in metres); c_a is the shaft adhesion (a value for c_a of 0.3 c_u to 0.35 c_u is commonly assumed, in kN/m^2).

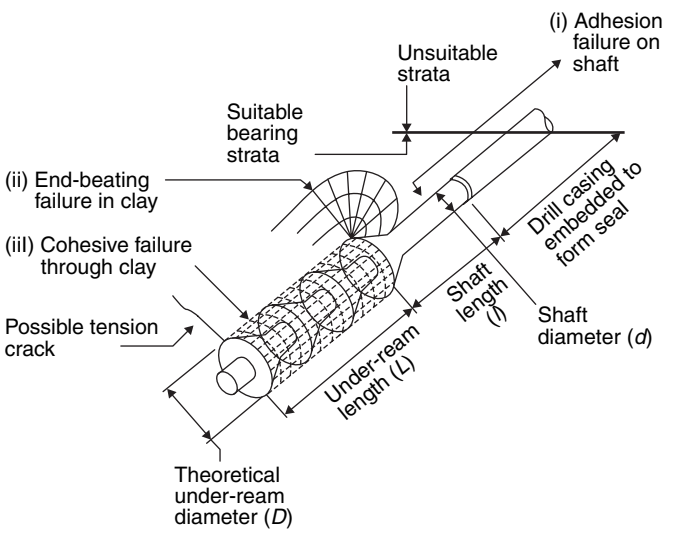


Figure 66.7 Diagram of multi-under-reamed ground anchor at ultimate capacity
Reproduced from Bassett, 1970 (as published in BS8081:1989)

It should be understood that l is the portion of the fixed anchor length at the proximal end of the under-reamed section. The contribution of this length is usually quite small, and is often ignored in practice.

Post-grouted anchors

For post-grouted (BS8081 Type C) anchors, additional grout is pumped under high, controlled pressures into the fixed anchor length using valved injection tubes, once the primary grout has achieved its initial set.

Load transfer design for such systems is based upon correlation with pull-out test data. This data effectively allows the estimation of an enhancement factor to the α value.

Design curves illustrating such relationships have been derived by Ostermayer (1974) as shown in **Figures 66.8** and **66.9** (from BS8081) and by Bustamante and Doix (1985) as in **Figure 66.10**.

Tests undertaken by Jones and Turner (1980) indicated an equivalent increase in the 'α' value of between 2 and 3 times the 'normal' design value of 0.3.

Fixed anchor length design in rock

A typical design relationship is expressed in the form of:

$$T_{ult} = \pi\, d\, L\, \tau_{ult} \qquad (66.6)$$

where τ_{ult} is a value for the ultimate (or 'failure') bond stress to be used for design, based upon previous pull-out test data or established empirical relationships.

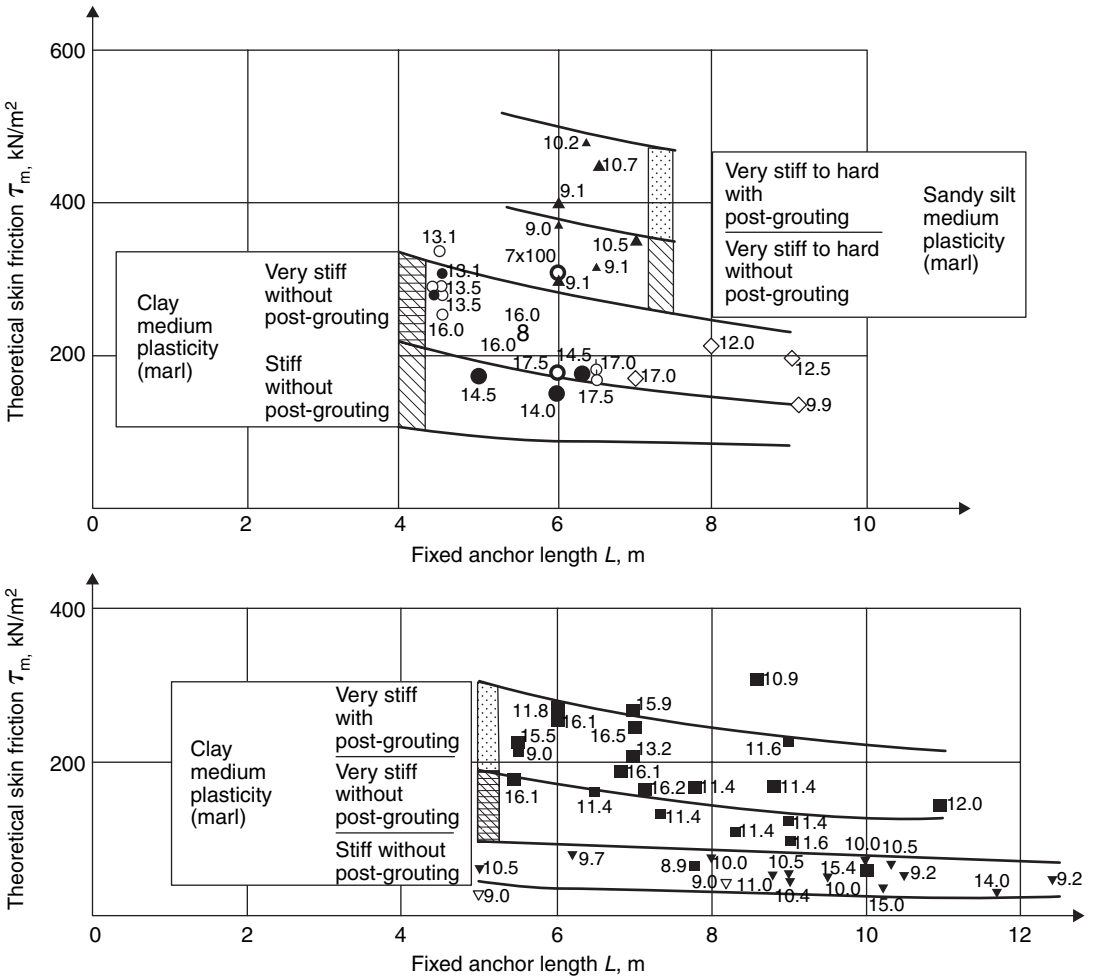
Established data may be site-specific or based upon tests in similar or related strata or conditions.

Most anchors installed in rock in the UK are straight-shafted, BS8081 Type A or B anchors (See Chapter 89 *Ground anchors construction*). Type C or D anchors have been less often used: mainly, as might be expected, in variable, low-strength or weathered rocks.

Almost all design bond-value data is empirical, based upon published case studies of field experience.

For strong intact rocks, an ultimate bond value equal to 10% of the unconfined compressive strength of the rock is often recommended. A maximum value of τ_{ult} of 4.0 N/mm^2 is usually recommended, based upon a typical design of the unconfined compressive strength for cement grout of 40 N/mm^2.

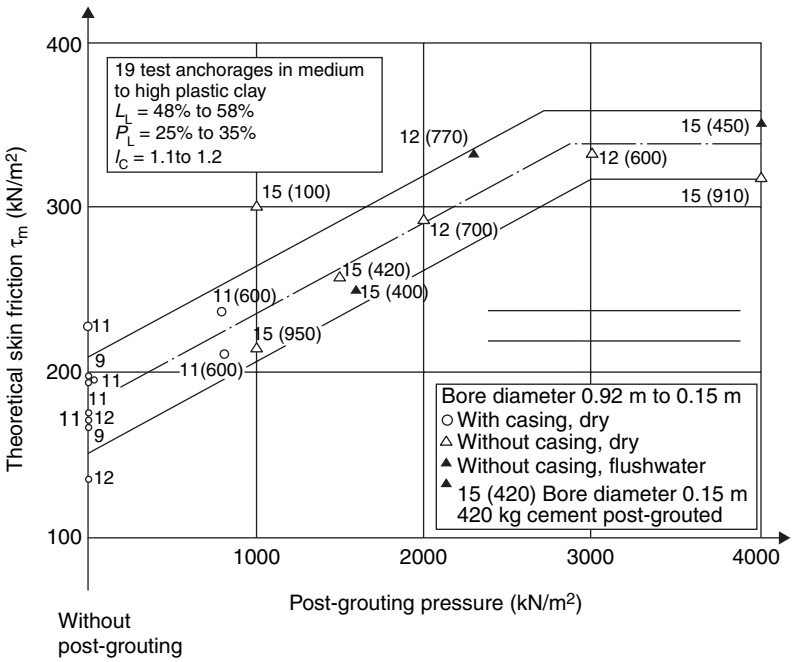
Tables 24 and 25 of BS8081 provide guidance on bond values, which have been utilised in design for a range of rock types and geological ages, mainly based upon field test data, or



Failure load was reached	Failure load was not reached	Post-grouting	Type of soil	L_L %	P_L %	I_c %
▲	△	Without	Silt, very sandy (marl) medium plasticity	~ 45	~ 22	~ 1.25
▲	△	With				
●	○	Without	Clay (marl) medium plasticity	32 to 45	14 to 25	1.03 to 1.14
○	◯	With				
●	○	Without		36 to 45	14 to 17	1.3 to 1.5
●	○	With				
	◇	Without	Silt medium plasticity	23 to 28	5 to 11	0.7 to 0.85
■		Without	Clay medium to high plasticity	48 to 58	23 to 35	1.1 to 1.2
■		With				
▼		Without		45 to 59	16 to 32	0.8 to 1.0

Figure 66.8 Ultimate skin friction in cohesive soils for various fixed anchor lengths, and with and without post-grouting
Reproduced from Ostermayer, 1974 (as published in BS8081:1989)

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NOTE. The theoretical skin friction is calculated from the borehole diameter and designed fixed anchor length

Figure 66.9 Influence of post-grouting pressure in cohesive soils
Reproduced from Ostermayer, 1974 (as published in BS8081:1989)

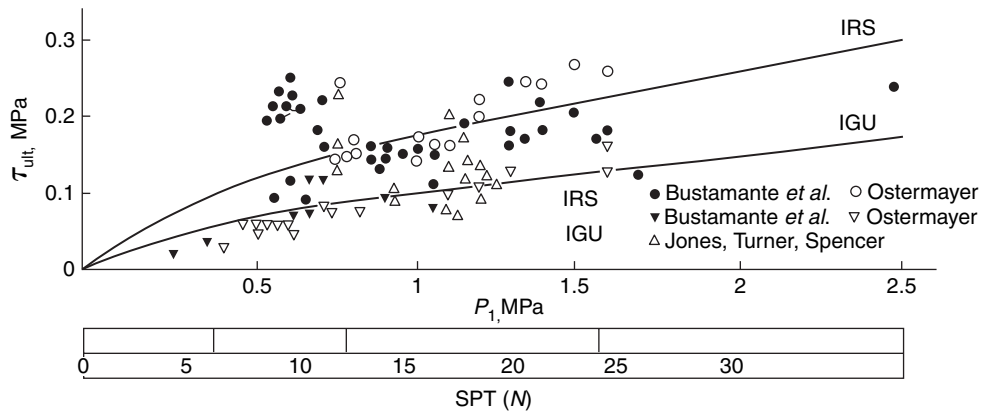


Figure 66.10 Empirical relationship for the determination of the ultimate skin friction for 'silty clay soils' using the IGU and IRP post-grouting techniques
Reproduced from Bustamante and Doix, 1985 (as published in FHWA, 1997)

satisfactory performance. These are expressed in both τ_{ult} and τ_{design} values as available, since there is often surprisingly little published data on bond-failure values.

For chalk and weak mudrock, as well as sandstone and 'other' rock types, Barley (1988) has produced a wide-ranging review of rock anchor load test data. Of particular assistance in design are correlations of SPT *N* values against bond stress for the weaker rock types. These correlations are based upon the relationships suggested by Cole and Stroud (1977) and Stroud (1989) for 'extrapolated' SPT *N* values and unconfined compression strength for rocks.

Important information and parameters required from a site-specific ground investigation were identified by Turner (2007) as:

- rock type, with a brief general description;
- geological formation: indicating age and possibly identifying typical characteristics of the strata;

- weathering grade: based upon the UK Geological Society Engineering Group (GSEG) grades, which are incorporated into BS 5930:1981, the Code of Practice for Site Investigations;
- unconfined (uniaxial) compressive strength (UCS): based upon UK GSEG grades, subsequently adopted by BS 5930;
- total core recovery (TCR);
- rock quality designation (RQD).

This paper also included data from proving tests in rock on both ground anchors and micro-piles, which included as much of the above data as was available from each particular site. Based upon this data and other published information, it was suggested the following general conclusions could be drawn, with particular reference to the weaker, more weathered, or more fractured rocks:

- There was a tendency that RQD had to be below 25% before it noticeably affected bond values.
- The ultimate bond value in chalk anchors tended to show a correlation with the SPT *N* value. The typical range of suggested values varied between an ultimate bond value of 10 to 30 times the SPT *N* value (kN/m^2), with a median value of around 15 to 20.
- Barley (1988) developed a useful guide relationship between ultimate bond and SPT *N* value for mudrocks, with a lower limit value of the ultimate bond value = 5 x (SPT *N* value). Such rocks are often difficult to sample adequately in standard site investigations.
- (UCS) / 10 is often used as a rough guide to ultimate bond stress. The test data generally supported such a rough correlation, but it should be used with great care.
- One would expect a correlation of increasing hole diameter with decreasing ultimate bond stress, for the same rock type, but there was not enough data to confirm this expectation.
- As would be expected, the degree of weathering affected the ultimate bond value, so that the more variable or the greater the degree of weathering, the more variable will be the performance of the anchor (or micro-pile).

It can be seen, therefore, that fixed anchor design in rocks is more empirical, and to a large extent is very much driven by both the quality of the ground investigation and the local knowledge of the anchor designer or installation contractor.

66.4.2.2 Encapsulation to anchorage grout bond

The design of the corrugated plastic sheath forming this interface is typically required to be such that a mechanical interlock will be formed between the external and internal grout on either side of the boundary. BS8081 recommends a maximum ultimate bond stress of 3.0 N/mm^2 at this interface where cement grouts are used, assuming that the bond is uniform over the surface. Higher values can be adopted if adequately proven. Bond tests have confirmed this is a reasonable but conservative low bound, and in most cases this is not a critical interface in designs.

It has to be remembered that, where two concentric sheaths are used, the inner sheath will be the critical interface.

(It should also be noted that EN1537 requires an increased sheath thickness compared with BS8081 recommendations.)

66.4.2.3 Tendon to encapsulation grout or anchorage grout bond

For temporary anchors, where the steel tendon is embedded directly in the anchorage grout, the tendon bond length is usually the same as the fixed anchor length. The length necessary for the grout–ground bond is usually more than adequate to ensure against failure of the tendon–ground bond. In the case of permanent anchors using strands, however, the encapsulation length is often shorter than the fixed anchor length (because a shorter encapsulation length is more convenient for handling the tendon at installation). In such a case the tendon–grout interface can become critical.

BS8081 suggests that the value for the ultimate bond stress at this interface when using cement grout should be limited to 2.0 N/mm^2 for a clean pre-stressing strand or deformed bar, rising to 3.0 N/mm^2 for strands that have been locally noded (where the outer wires of each strand are unravelled and then rewound around small collars placed onto the kingwire at intervals to form small nodes). Tests described by Turner (1980) suggested that a value of 2.0 N/mm^2 for a plain strand was reasonable, but that deformed bars could develop in excess of the 5.0 N/mm^2 ultimate bond stress. The validity of the 50% increase in the bond value for noded strands to 3.0 N/mm^2 was considered to be more questionable, since tests indicated that such nodes would have to be at close centres to achieve the stated 50% increase in the bond value.

66.4.3 Tendon design

The great majority of anchors in the UK are constructed using tendons composed of one of the following materials:

- *Low-relaxation pre-stressing strands* A 15.2 mm diameter strand with a characteristic strength of 300 kN is the most common. Multiple strands are used for higher loads.
- *High-tensile steel bars of pre-stressing quality* Most often fully threaded bars are used, with diameters of between 15 and 75 mm, and characteristic strengths up to 4 500 kN. Larger, higher capacity bars are also available.
- *High-yield steel bars* Again, these are most often fully threaded, with diameters between 16 mm and 63.5 mm and characteristic strengths up to 2 200 kN.
- *High-yield steel hollow bars* These are usually fully threaded, with external diameters between 25 and 130 mm and characteristic strengths up to 2 310 kN.

When designing to BS8081 guidance, it is usual to use global factors of safety as in **Table 66.2** for both high-tensile (pre-stressing) and high-yield steels. When designing to EN1997 Eurocode-7 and BS EN1537 both these publications refer the

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designer to BS EN1992 Eurocode-2 The Design of Concrete Structures (BSI, 2004b).

66.4.4 The location of the fixed anchor zone (overall stability)

As noted in section 66.2 and **Table 66.1**, EN1537 implies that the responsibility for the specification of a minimum distance from the structure to the middle of the fixed length to ensure the stability of the structure lies with the wall designer. This is only part of the design. It has to be the responsibility of the anchor designer, in addition, to ensure that the anchors are deeply enough embedded beyond the zone identified by the wall designer to avoid an overall pull-out failure of the body of soil or rock surrounding the anchors.

The methodology generally adopted for such a design is to determine the location of a notional failure surface behind the wall that has an acceptable stability against general failure. The fixed anchor lengths of the ground anchors should then be located beyond this failure surface. In European practice it is not uncommon that the mid-point of the fixed anchor length is located on this surface. In UK practice it is more usual to locate the whole of the fixed anchor beyond this surface (e.g. Littlejohn *et al*., 1971).

Appendix D of BS8081 reviews several design methods for ensuring the overall stability of an anchored retaining wall, and an introduction to these is given below.

More general guidance on limit equilibrium methods of stability analysis are given in Chapter 23 *Slope stability* and Chapter 64 *Geotechnical design of retaining walls*.

66.4.4.1 For c′=0 'granular' soils

The design methods used to determine the location of such an acceptably stable surface within granular soils vary from 'planar' wedge or 'sliding block' analysis techniques to conventional circular or non-circular stability analyses, as described below and, in detail, in Chapter 23 *Slope stability*. The methods are also reviewed in more detail in BS8081 and in Littlejohn (1970, 1972).

The planar wedge method is illustrated in **Figure 66.11**. It has been used for design in both granular and clayey soils, and for multiple rows of anchors. Values for the parameters 'β' and 'x' in this figure, which have been used in practice, are recorded (BS8081) as: varying between 27° and 45° to the vertical for β, with 'x' varying between zero and 6.0 m. However, it is commonly held (e.g. Hanna, 1982) that justification of the method is difficult because the mode of failure is unlikely to be planar.

A typical 'sliding block' analysis is illustrated in **Figure 66.12**. It is suitable for analysing a single row of anchors. The method illustrated is based upon that developed by Kranz (1953), and modified by Locher (1969) and Littlejohn (1970, 1977). Other methods are also described in BS8081.

As described by Littlejohn *et al.* (1971), the lateral earth pressure, P_n, on the vertical surface *b–c* drawn through the proximal end of the fixed anchor length is calculated using a 'nominal' friction angle ϕ'_n, which is less than the effective angle of internal friction of the soil such that

$$\tan \phi'_n = (\tan \phi'_d) / F \qquad (66.7)$$

where F is a safety factor, and ϕ'_d is the design value of the angle of internal friction of the soil.

The resultant R_n on the inclined sliding plane *c–d* will also be inclined at ϕ'_n to the normal to *c–d*.

By iteration, when the disturbing and restoring forces are in equilibrium, by revising the geometry of the soil block *a–b–c–d*, then the desired safety factor has been achieved. The additional stabilising force due to the passive resistance at the toe of the wall is ignored in the calculation, which is a conservative assumption.

The dashed lines in **Figure 66.12** represent the typical shape of the failure curve, derived from experimental tests on plate or 'deadman' anchors. The polygon *a–b–c–d* is a simplification of this shape for calculation purposes.

It should be noted that both the above examples show the minimum distance to the proximal end of the fixed anchor length from the face of the wall as being at the boundary of the calculated 'failure' surface. As noted earlier, this is at variance with EN1537, which does suggest that the designer should specify the minimum distance to the mid-point of the fixed anchor length, as in **Table 66.1**. Hence, care should be taken to ensure that the wall designer and the anchor designer are in concert with their design approaches. As a general approach, it is recommended that the anchor designer locates the whole of the designed fixed anchor length beyond the design failure surface unless both designers agree that the EN1537 approach is appropriate.

It should also be noted that the anchors must also have a sufficient depth of vertical cover above the fixed anchor length to avoid localised passive failure of the soil, similar to the failure condition associated with shallow 'deadman' anchors. A typical rule of thumb cited by Littlejohn (1972), and based upon early experience of ground anchors in UK alluvial sands and gravels, suggests that a minimum depth of cover of 5 to 6 m is normally considered sufficient to guarantee a deep-seated failure condition at pull-out. It can be seen, also, that equation (66.3) for the calculation of the load capacity of an anchor is also sensitive to the overburden pressure around the fixed anchor length. Hence, by implication, it would be expected that the smaller the overburden pressure above a given fixed anchor length, the lower the capacity of the ground anchor. If there is any doubt about the performance of a near-surface anchor, its performance should be confirmed either by investigation or suitability testing to EN1537.

For multi-tied walls a circular-type of stability analysis is usually recommended to determine the location of the acceptably stable failure surface. This is because the shape of the sliding surface is not clear experimentally with multiple rows of anchors. Littlejohn *et al*. (1971) recommend a logarithmic

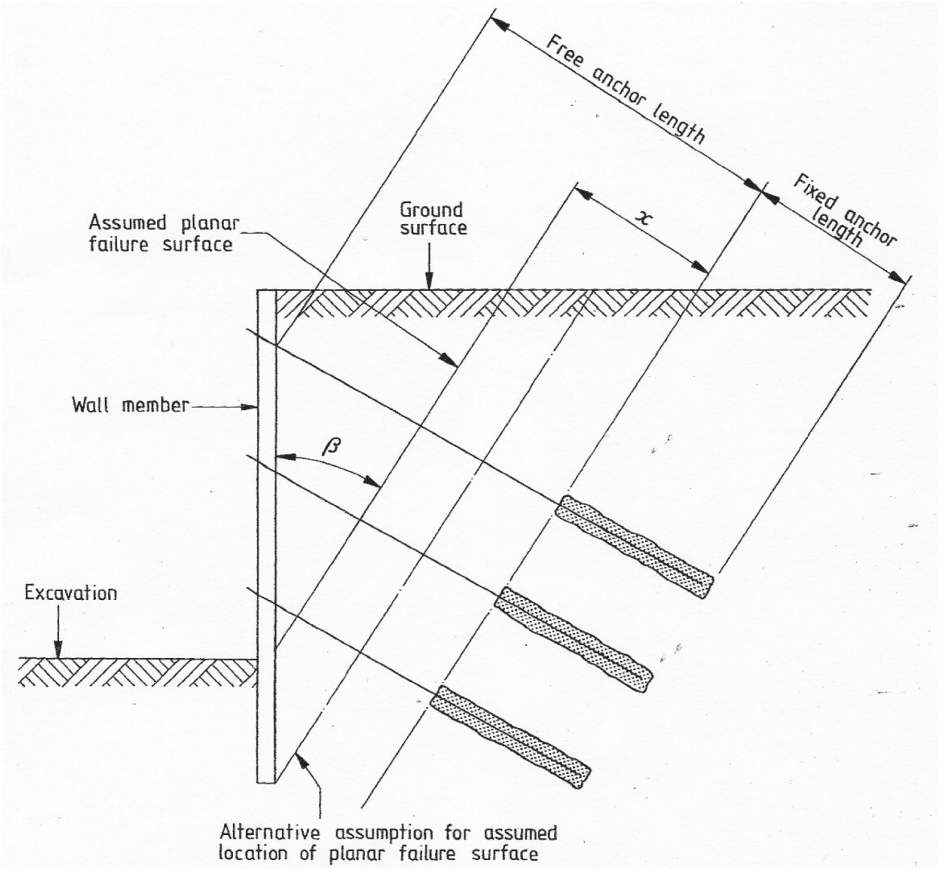
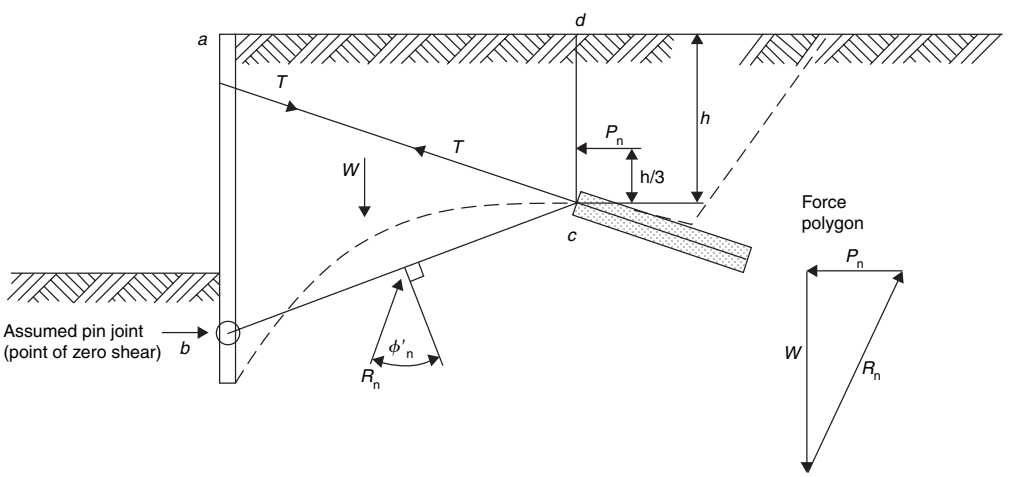


Figure 66.11 Typical planar wedge method of analysis
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Factor of safety F is given by:

$$F = \frac{\tan \phi'_d}{\tan \phi'_n} \geq 1.5$$

where

ϕ'_n is nominal angle of shearing resistance in terms of effective stress (in degrees).

NOTE: If ϕ'_n has been correctly assumed, the weight W and the forces R_n, and P_n are in equilibrium. If this is not the case ϕ'_n has to be altered.

Figure 66.12 Typical sliding block method of analysis
Reproduced from Littlejohn (1972)

spiral method for defining this surface, as illustrated in **Figure 66.13**. The reason for this is that a log spiral has the property that the radius from the spiral centre to any point on the curve forms a constant angle ϕ_s with the line drawn normal to the curve. Hence, if a nominal friction angle, ϕ'_n, for the soil is selected for ϕ_s, then the line of action of the resulting forces on each part of the sliding surface will pass through the centre point of the spiral. Hence, none of the forces along the failure surface will create a moment around the centre point and they can be neglected when equating moments about this point.

In a similar manner to the sliding block method, the value for F is satisfied when the value for ϕ'_n is such that the moments of the remaining forces sum to zero, as shown in **Figure 66.13**.

It is again usual to ignore the stabilising effect of the passive resistance of the soil beyond the toe, so that when the moments W_s and W_o are equal, a conservative value is obtained of $F = \tan \phi'_d / \tan \phi'_n$.

The definition $F = \tan \phi'_d / \tan \phi'_n$ can be considered as a partial factor of safety against failure of the soil body. The chosen value of F used in practice can vary between 1.2 and 1.5 for a sliding block design, and ≥1 for a spiral-shaped sliding surface design. The higher value would be selected for more critical applications, or where conditions are less well defined.

All the analysis techniques illustrated in **Figures 66.11** to **66.13** make an underlying assumption that the pre-stress exerted by the post-tensioned anchors increases the shear strength of the soil sufficiently to ensure the potential failure plane is beyond the proximal end or the mid-point of the fixed anchor length, depending on the design philosophy noted above.

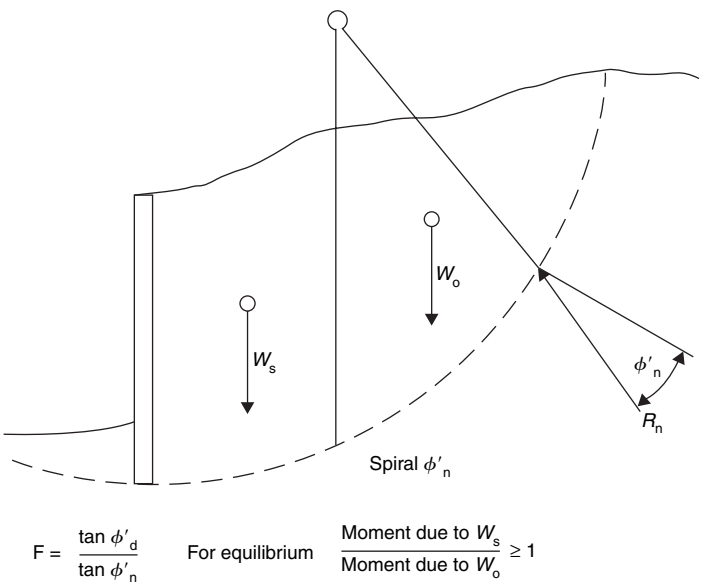


Figure 66.13 Typical stability analysis using a log spiral shaped sliding surface
Reproduced from Littlejohn, 1972

66.4.4.2 For clayey soils

For clayey soils a circular form of failure surface would normally be preferentially analysed, and, again, as a general recommendation, the whole of the fixed anchor length should be located beyond the failure surface with the required value for the factor of safety, unless the designers agree otherwise.

Reference has been made to the use of a planar wedge design method described above for granular soils, being used also for clayey soils, but this method has to be used with caution, and, also perhaps previous experience. It has been used successfully in the past on such projects as the Neasden Underpass in London (Sills *et al*., 1977).

Compared with the design assumptions for granular soils, it can be appreciated that anchor pre-stress will only slowly increase the shear stress of a clayey soil over time. It is usual for such soils, therefore, that a conventional analysis of the overall stability of the wall should be undertaken, ignoring the effect of the proposed ground anchors, and identifying the location of the failure surface that has an acceptable factor of safety, in a similar manner to that described above for granular soils. The fixed anchor lengths of the anchors should then be located beyond this calculated surface.

For mixed soil and rock conditions, where, typically, anchors might be located in a rock stratum beneath the overlying soils, as illustrated in **Figure 66.14**, it should be appreciated that rock anchors generally develop higher unit bond strengths than soil anchors. In some circumstances there could, therefore, be a risk of pull-out failure of a conical or wedge-shaped mass of rock by the tensile force imposed by the wall if the anchor is not embedded sufficiently deeply into the rock stratum. This possibility needs to be checked by the anchor designer.

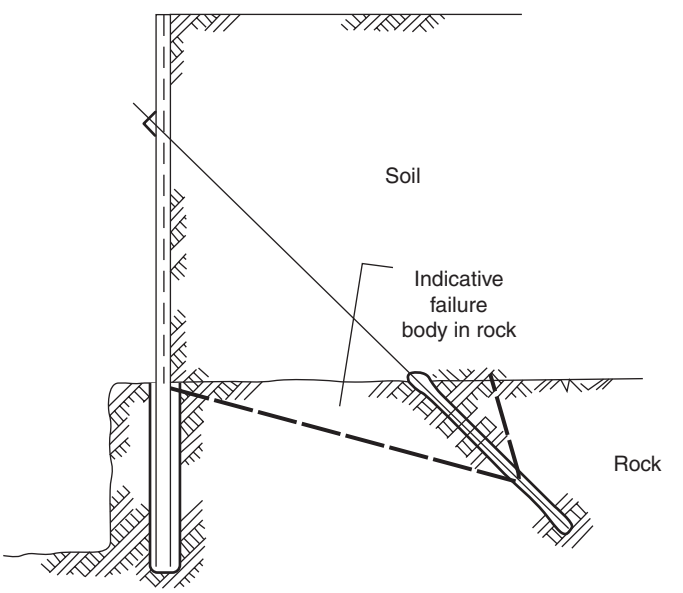


Figure 66.14 Indicative pull-out failure body within rock stratum

66.4.5 Load transfer into the structure

The head assembly for a ground anchor typically consists of a stressing head or anchor block onto (or into) which the tendon is anchored, and a bearing plate, which transfers the anchor force onto the structure. In the case of an embedded retaining wall the bearing plate can often bear onto steel or concrete waling beams to distribute the imposed anchor forces more evenly into the wall.

An important factor in such load transfer and distribution systems is that the vertical component of the anchor force has to be accommodated within the design.

In steel sheet piles and king-post walls, with steel waling beams, some form of angled bracket assembly must be provided. Typical solutions to this problem are illustrated in **Figure 66.15**.

In contiguous or secant piled walls, very often reinforced concrete walings or load transfer assemblies would typically be employed, as in **Figure 66.16**.

For diaphragm walls, a suitable angled pocket is usually cast into the wall panel, together with a guide tube through the wall section to locate the drill string clear of the reinforcement.

With particular reference to secant and diaphragm walls, it may be necessary to make provision for the anchor head system to withstand hydrostatic pressures. This is often more difficult to achieve than might be expected, and needs close attention to design, installation and construction procedures.

EN1537 (Section 6.3) outlines the design and tolerance requirements of the anchor head. One requirement is that the anchor head should be designed to tolerate an angular deviation of up to 3° from the axial direction of the tendon. Whilst this is a normal requirement for post-tensioning systems, it can cause difficulties with the protective systems for permanent ground anchors if attention is not paid to this detail. Specific requirements should be highlighted, discussed and clarified with the suppliers of such systems.

BS8081 makes specific reference to the design requirements of ground anchors in terms of the facility for the anchor to be re-stressed or even detensioned during its service life. It defines the requirements of 'normal', 're-stressable' and 'detensionable' anchor heads. These cases are not recognised by EN1537, which, effectively, leaves the decision to the knowledgeable designer. For reference, BS8081 identifies the following cases:

- A normal anchor head is designed to allow tensioning and overload testing of the anchor tendon during the acceptance test phase. Once the anchor has been accepted, the excess stressing tendon may be cut off and no further measurement or adjustment is possible.
- A re-stressable anchor head has all the properties of a normal head and, in addition, allows the load in the tendon to be checked during the life of the anchor and allows small losses of up to 10% of the working load to be recovered, normally by shimming or thread turning.

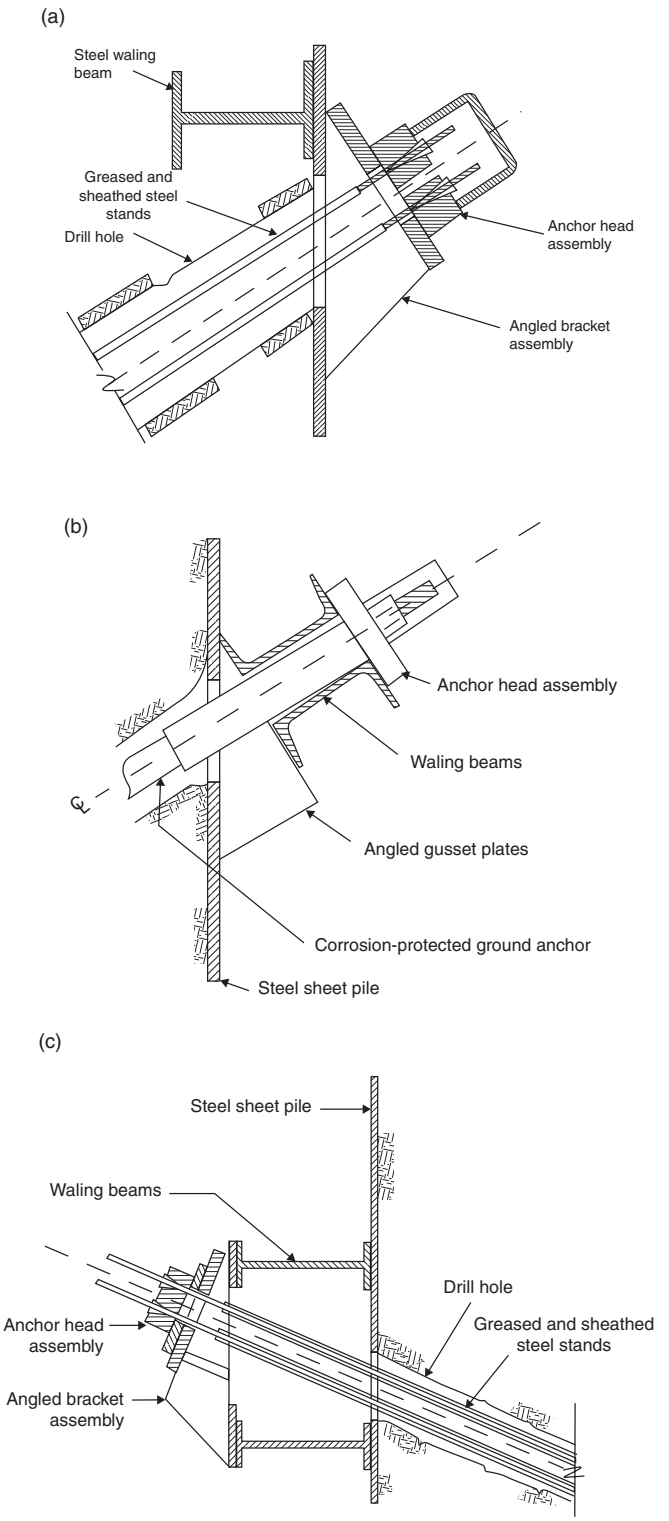


Figure 66.15 Typical use of ground anchors with steel walings. (a) Internal waling, with external angled bracket assembly; (b) external angled waling; (c) external waling with angle bracket assembly

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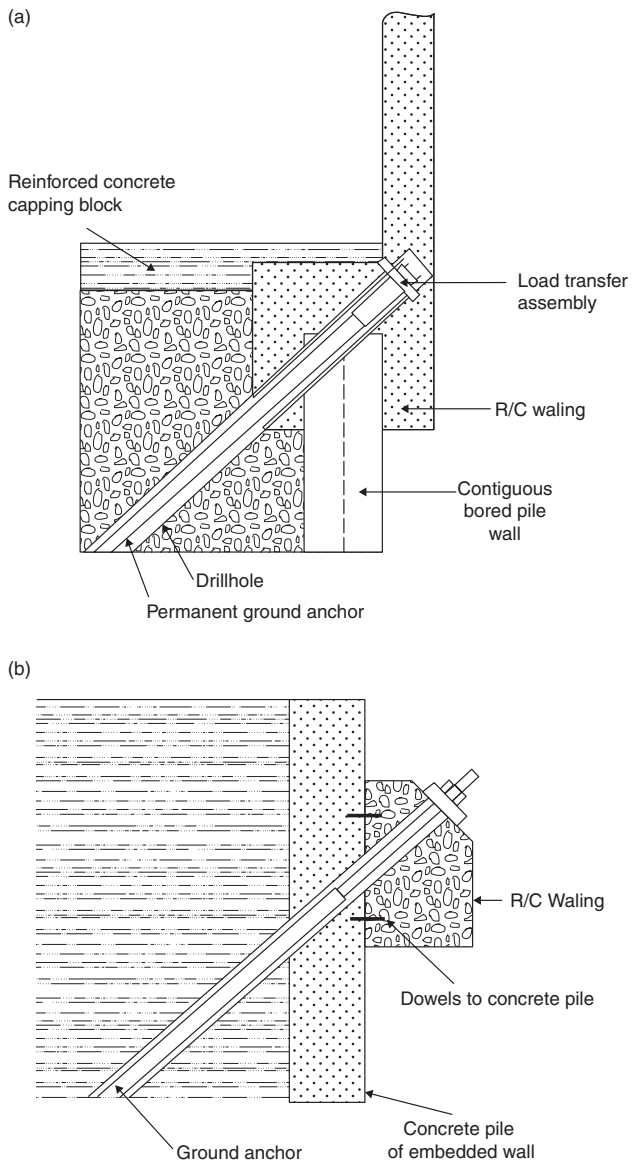


Figure 66.16 Typical use of ground anchors with reinforced concrete walings or load transfer assemblies

- A detensionable anchor head has all the properties of a re-stressable head and, in addition, allows the tendon to be detensioned in a controlled way at any time during the life of the structure.

It is important that the wall designer, as well as the anchor designer, is aware of these different facilities and is clear what level of 'adjustability' is required for their particular case.

BS8081 also suggests (at Clause 7.4.6) that a degree of redundancy should be allowed within the anchored system to a retaining wall. A structure should be designed so that, in the limit, any one anchorage can fail without the load on the other anchors exceeding the proof load to which they were initially tested.

66.4.6 Testing

The ground anchors supporting retaining walls should be subjected to the regime of investigation, suitability and acceptance testing recommended by BSEN1537, as outlined in Chapter 89 *Ground anchors construction*.

As part of the routine monitoring of the supported excavation, it is wise to consider monitoring loads in selected anchors, especially for deep, multi-tied excavations. It is important to confirm that the performance of a wall is maintained within its design envelope, and monitoring anchor loads is one part of this regime. Such load-checking can be undertaken using load cells or by routine check-lifting operations.

It is also not unknown for anchor loads to fall or fluctuate during the life of the structure, especially for sheet-piled walls where new fill has been placed behind the wall, in the construction of new river walls, for instance. Turner and Richards (2007) discuss load changes and lateral movements recorded in an anchored sheet pile wall constructed along the River Thames in 1979, where the site was redeveloped some twenty years later. Horizontal movements of the piled wall, supported by a single row of inclined ground anchors, were found to vary in a roughly cyclic manner between 25–30 mm over a year.

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It is recommended this chapter is read in conjunction with

- Chapter 89 *Ground anchors construction*
- Chapter 94 *Principles of geotechnical monitoring*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

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Chapter 67

Retaining walls as part of complete underground structure

Peter Ingram Arup, London, UK

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Retaining walls usually form part of an overall structure. They are often used both to create basement space and to act as part of the overall foundation system. As a geotechnical designer it is important not to lose sight of the fact that the function of the entire structure is the priority for the client, rather than the function of the walls and foundations alone.

This chapter is intended to highlight how, as part of designing an overall underground structure, it is necessary for the retaining wall designer to consider not only how the various elements forming the underground structure will interact and react as a whole structure, but also how the geotechnical design must be consistent with the work of other disciplines.

The following discussions provide the reader with a basic understanding of the issues relating to the holistic design of the whole basement/underground structure and interaction with other disciplines.

67.1 Introduction

This chapter provides a description of some of the wider issues which need consideration for holistic basement design in addition to basic retaining wall design calculations. The whole structure should be considered by designers before and during the design of the wall, as described in here and in Chapters 64 *Geotechnical design of retaining walls* to 66 *Geotechnical design of ground anchors*. Successful application of the concepts contained in this chapter can require a high degree of interaction between the various engineering disciplines involved in basement design.

67.2 Interfaces with structural design and other disciplines

Underground structures require input from a variety of disciplines, often with conflicting requirements. In order to safely design for the whole structure, it is important that the various engineering disciplines, and the architect, understand clearly the pinch points in each other's designs. For example, floor slab (e.g. propping) levels may be constrained by stair heights, and may therefore not be relocated to optimum levels to suit the geotechnical design. For small projects it is relatively easy to get a consensus as, typically, a small number of individuals are involved. For larger projects, this is most likely to be successfully achieved by:

- having the team in one location;
- regular multi-disciplinary design reviews;
- production of formal design guidance, setting out clearly how the different disciplines will design their respective elements of the structure, and how their design will interact with others.

The following sections deal principally with common design interfaces with the structural engineer.

67.2.1 Design life and durability

BS EN 1990: 2002 requires the designer to specify the design working life for the structure, and outlines indicative time periods for typical structures. *Building structures and other common structures* are to be designed for a suggested working life of 50 years, with *monumental building structures, bridges, and other civil engineering structures* having a suggested design working life of 100 years.

Durability in concrete structures is typically provided by three means:

(i) Having an appropriate concrete mix to take account of the method of placement, and exposure to aggressive ground conditions – determined with reference to BRE (2005).

(ii) Appropriate reinforcement detailing to limit cracking, usually incorporating a continuous layer of reinforcement in each face, for walls and slabs in contact with the ground.

(iii) Appropriate concrete cover to reinforcement in accordance with the requirements of BS EN 1992–1-1:2004.

67.2.2 Design philosophy for buried structures (crack width control, construction joints, watertightness, etc.)

Understanding the client's requirements for end use, and hence water tightness, is key to determining appropriate wall details.

BS 8102 (2009) defines grades of watertightness and methods of achieving them. **Table 67.1** is reproduced from the standard.

The previous edition of BS 8102 (1990) referred to grade 4 (or special) environments. The 2009 edition has not retained this grade, stating that the only difference between grades 3 and 4 is the performance level related to ventilation, dehumidification or air conditioning, and suggests referring to BS 5454

(2000) for recommendations for the storage and exhibition of archival documents. BS 8102 notes that the structural form for grade 4 could be the same or similar to grade 3.

The *Specification for Piling and Embedded Retaining Walls* (ICE, 2007) provides guidance on levels of acceptable wall seepage for the grades defined in BS 8102.

- *For grade 1 structures* Beading and limited damp patches are tolerable, but no weeping.
- *For grades 2, 3 (and 4) structures* Beading and limited damp patches are tolerable, but no weeping. Other components will be needed in addition to the retaining wall component to achieve the required watertightness of the whole system.

BS 8102 states that resistance to water ingress is usually provided by one or a combination of the following methods:

- *Type A (barrier) protection* Protection against water ingress which is dependent on a separate barrier system applied to the structure.
- *Type B (structurally integral) protection* Protection against water ingress which is provided by the structure.
- *Type C (drained) protection* Protection against water ingress into usable spaces which is provided by the incorporation of an appropriate internal water management (drainage) system.

Watertightness provided by the structure (types A and B) is affected by cracking, construction joints, differential movements, openings for services, etc., so design for an appropriate watertightness requires an understanding of how to control water ingress at these points.

BS EN 1992–1-1:2004 (BSI, 2004) provides requirements for crack control in reinforced concrete structures. Gaba *et al*. (2003) states that 'Cost savings are possible if a pragmatic approach is taken to crack width control'. Consideration of appropriate limits of cracking for buried retaining wall structures can lead to a significant reduction in the reinforcement required to control cracks. Useful guidance on this is given in IStructE (2004) and Gaba *et al*. (2003).

Grade	Example of use of structure	Performance level
1	Car parking; plant rooms (excluding electrical equipment); workshops	Some seepage and damp areas tolerable, dependent on the intended use
2	Plant rooms and workshops requiring a drier environment (than Grade 1); storage areas	No water penetration acceptable. Damp areas tolerable; ventilation might be required
3	Ventilated residential and commercial areas, including offices, restaurants, etc.; leisure centres	No water penetration acceptable. Ventilation, dehumidification or air conditioning necessary, appropriate to the intended use

Table 67.1 Grades of watertightness
Data taken from BS 8102:2009 (BSI, 2009)

In addition to consideration of appropriate limits on flexural cracking in the retaining wall section, and the cracking of wall structural elements, the following are the areas where particular care is required:

- Interfaces between structural wall elements and capping beams or slabs. These require careful detailing if water flow paths are to be restricted.
- Points of gross change in construction (e.g. where new and existing construction join) result in higher risk of entry for water.
- Discrete movement joints. These should be avoided where possible in the outer envelope of basements as they are a frequent point of weakness and leakage. Where such movement joints cannot be avoided they should be protected against water ingress.
- Construction joint spacing in water-resistant concrete should be determined in accordance with requirements to limit shrinkage and early age cracking.
- Applied barrier system waterproofing must be able to accommodate the predicted lateral wall movements following installation of the system.

Further discussion on watertightness of basement structures, and guidance for clients can be found in ICE (2009) *Reducing the Risk of Leaking Substructure – A Clients' Guide*.

67.2.3 Design codes and standards

Chapter 10 *Codes and standards and their relevance* provides a comprehensive list of the main codes and standards to be used in the UK for design. That chapter draws attention to the first point-of-call design guidance for underground structures in the UK, although these are likely to be broadly applicable to other geographic locations. The primary sources of geotechnical and structural design guidance for underground structures in the UK are shown in **Figure 67.1**.

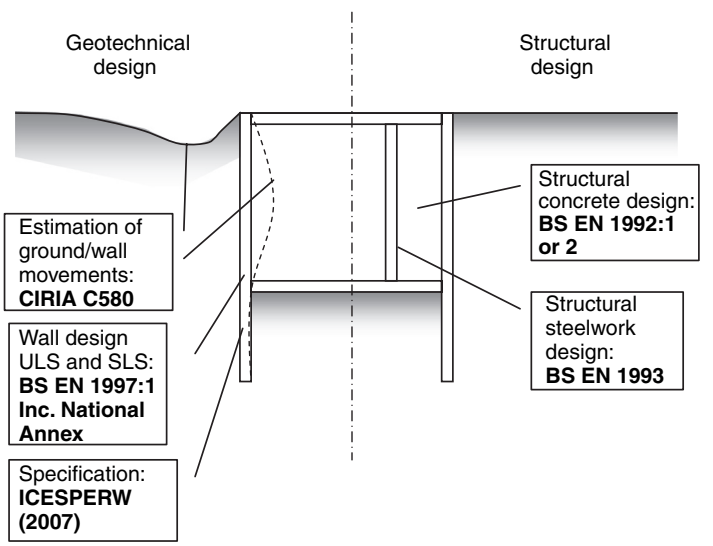


Figure 67.1 UK design guidance

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In addition to the guidance highlighted in **Figure 67.1**, a series of execution standards are available to complement the Eurocodes. Most relevant to geotechnical construction of basements are:

- BS EN 1536: 2000 *Execution of Special Geotechnical Works – Bored Piles*;
- BS EN 1537: 2000 *Execution of Special Geotechnical Works – Ground Anchors*;
- BS EN 12063: 1999 *Execution of Special Geotechnical Works – Sheet Pile Walls*.

67.2.4 Load combinations (lateral and vertical)

BS EN 1997–1 Eurocode EC7 (2004) defines the following load factors for use in retaining wall design. Design is to be carried out using Design Approach 1 in the UK, using both Combination 1 and Combination 2 factors (**Figure 67.2**). Both of these load combinations are ultimate limit state (ULS) conditions. Serviceability must be considered separately.

67.3 Resistance to lateral actions

As part of the whole underground structure, the retaining wall needs to fulfil a variety of functions, and must:

- be able to resist lateral forces in bending or in overall horizontal stability, to provide an adequate factor of safety;
- be able to support the applied vertical loading at all stages of construction, and in the permanent condition, including the need to maintain a satisfactory factor of safety against flotation and soil heave;
- provide hydraulic cut-off and/or maintain satisfactory basal stability;
- be of sufficient stiffness to restrain ground movements outside of the structure to within tolerable limits.

67.3.1 Determination of retaining wall toe level for lateral stability and associated load effects

Chapter 64 *Geotechnical design of retaining walls* provides design guidance for considering retaining wall lateral stability and loading, highlighting the need for compatibility with the requirement for the wall to support vertical loading, and the importance of designing for both.

The design of underground structures requires a holistic and consistent approach to the design of the retaining walls, internal piles and base slabs. The following is a step-by-step approach to considering the interaction between the designs of the various elements:

(i) Determine magnitude of wall frictions for vertical/horizontal compatibility of wall movements.

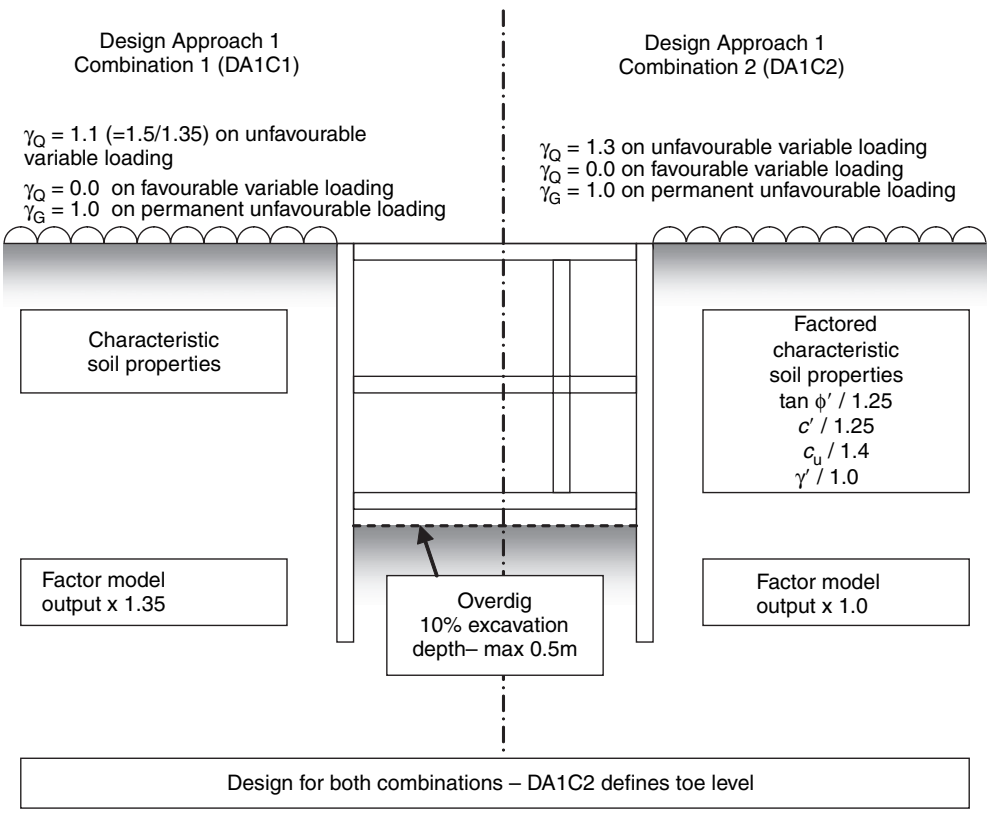


Figure 67.2 Design factors to BS EN 1997–1:2004, Eurocode EC7 (2004) . γ_Q partial factor for a variable action; γ_G partial factor for a permanent action; ϕ' effective angle of shearing resistance; c′ effective cohesion intercept; cu undrained shear strength; γ' effective unit weight

(ii) Analyse walls to determine minimum toe depth for lateral stability; see Chapter 64 *Geotechnical design of retaining walls.*

(iii) Consider the ability of walls to carry the vertical loads during excavation, and increase toe depth if necessary; see Chapter 64 *Geotechnical design of retaining walls.*

(iv) Determine additional foundation dimensions to carry vertical loading, e.g. pile foundations.

(v) Adopt design wall toe level and review the direction and magnitude of assumed wall friction and adhesion at soil/wall interface. Re-do lateral analyses, if appropriate, to confirm wall design toe level.

(vi) Consider vertical uplift forces (these can be reduced where deflection of the structure is acceptable).

(vii) Consider any heave void or under-slab drainage.

(viii) Increase foundation element/wall depths to ensure sufficient factor of safety against uplift, where necessary.

(ix) Consider the design of the base slab to accommodate any net uplift forces.

(x) Consider the base slab/piles/walls acting as an overall foundation system to resist vertical loads.

67.3.2 Accidental loading

Accidental loss of a temporary prop The designer should allow for the accidental loss of a temporary prop during construction which could happen for a variety of reasons; for instance, if something were dropped from a crane. The designer must ensure an adequate load factor, and deformations must remain within repairable limits. If the design is particularly sensitive to the loss of a temporary prop, the designer may opt to specify redundancy within the propping system to account for such an eventuality.

Loss of a permanent prop Most basement walls are unlikely to be able to survive this scenario without serious damage, and would most likely totally collapse. If this is recognised by the designer as a viable load case (for instance, in the design of metros which may be vulnerable to terrorist activity), then the designer needs to ensure that the slab/prop is structurally able to tolerate the foreseeable forces to which it may be subjected in such a scenario.

Flooding The designer should consider whether flooding is likely, to what extent, and for what duration. Flooding can increase loads on the structure in two main ways: (i) it may increase lateral water pressures behind the walls; and (ii) it may apply an additional surcharge on the slabs and the soil retained by the walls.

67.3.3 Out-of-balance forces

Underground structures are often subjected to out-of-balance lateral forces for a variety of reasons. It is important to consider the interaction of out-of-balance forces in design, rather than just designing the wall with the higher lateral forces and not considering how this will affect (or be affected by) the opposing wall.

Typical causes of permanent asymmetric loads are:

- ground or groundwater conditions, e.g. a sloping site, heavily variable stratigraphy, or a basement adjacent to water on one side;
- surcharges adjacent to the basement, e.g. from nearby structures, roads or railways;
- superstructure: varying building heights over the basement causing an imbalance in vertical loading of walls;
- substructure stiffness, e.g. different response of opposing walls formed in different manners;
- adjacent tunnels or other underground infrastructure;
- pressures from adjacent ground treatment works.

In addition, it is important that the designer considers temporary imbalances, for example:

- loading from adjacent construction plant;
- construction storage or stockpiles;
- asymmetrical construction sequences.

When the design is being carried out using finite element modelling (FEM), catering for these imbalances once they have been established is relatively straightforward. For underground structures subject to significant out-of-balance forces, FEM is likely to be the most expedient way of understanding and designing for these forces. It is common to design retaining walls using limit equilibrium or pseudo-finite element (PFE) analyses which consider a single wall only. For simpler force imbalances, it is possible to iterate between two separate PFE models, each representing opposite sides of the excavation, with the prop pre-stress forces adjusted at each iteration until deflection and prop force are balanced across the excavation.

67.4 Resistance to vertical actions

For a basement structure it is likely (and usually economical) that the retaining wall will be used to resist vertical loading. Vertical loading on the wall is likely to be different in the temporary and permanent condition. A typical basement structure carried out using a bottom-up methodology may be subject to the following variations in loading over its construction and use (**Figure 67.3**):

(i) Excavation of basement, using temporary props – walls lightly loaded.

(ii) Completion of substructure. Vertical downwards loads – increased from (i) due to the slabs – now supported by combination of walls and base slab bearing pressure. Uplift or heave forces on the structure to be resisted by a combination of structure dead weight and wall frictional resistance (or controlled by a heave void/drainage layer).

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(iii) Completion of superstructure. Increased resistance to uplift/heave forces provided by superstructure weight. This also increases vertical downwards forces into walls/base slab.

(iv) Redevelopment of superstructure whilst re-using basement structure – similar to condition (ii).

The magnitude and direction of the vertical loading affects the direction of the friction forces acting on the wall, and hence the lateral pressure applied to the wall by the surrounding ground. The design of embedded retaining walls in the UK is often carried out using limit equilibrium or PFE computer programs such as STAWAL, FREW or WALLAP, where the designer must select the magnitude and direction of wall friction when determining active and passive earth pressure coefficients. It is therefore important to consider the variation of vertical loading in the temporary and permanent cases when determining appropriate wall frictions for use in lateral design.

67.4.1 Design of retaining walls to support/resist vertical loading

The design of embedded retaining walls in the UK is described in the previous paragraph. Typical directions of wall friction adopted for basement wall design are shown in **Figure 67.4**, which has been reproduced from CIRIA C580 (Gaba *et al*., 2003). These assumptions of wall friction are appropriate for excavations where vertical loading of the retaining wall is minor, such as in **Figure 67.5(a)**.

Diaphragm or piled retaining walls for deep excavations, particularly in the top-down method of construction or anchored walls, may be subject to significant vertical compressive loading during construction, in particular just prior to the casting of the base slab (**Figure 67.5(b)**). The designer must consider whether the magnitude of applied vertical loading is such that the assumed applied sense of the wall frictions reverses, as this will influence the applied horizontal pressures (see e.g. Ingram *et al*., 2009).

If the design is carried out using appropriate finite element analysis, then compatibility of friction assumptions can be modelled directly.

For excavations below the water table or in heaving soils, the retaining wall may become a tension element resisting the upward pressures acting on the underside of the base slab. Under these conditions, the friction directions shown in **Figure 67.6** may be applied.

The change in wall frictions caused by uplift conditions will reduce the passive resistance of the wall, increasing the

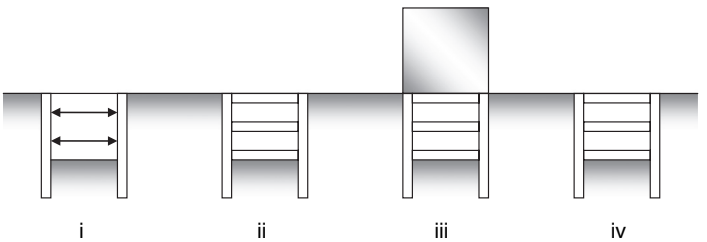


Figure 67.4 Typical construction conditions for basement wall loading

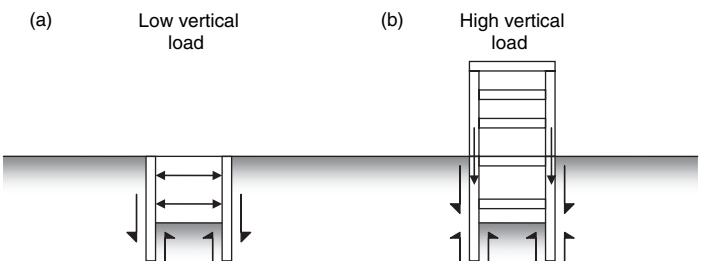


Figure 67.5 Possible wall frictions around excavations with low and high vertical loads

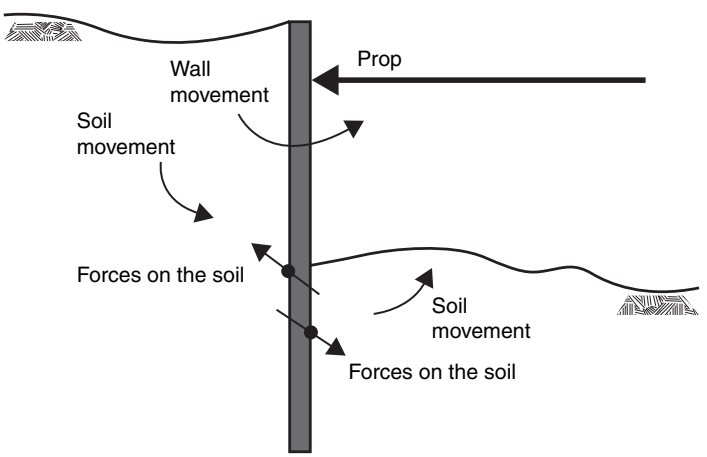


Figure 67.3 Effect of wall friction
Reproduced with permission from CIRIA C580, Gaba *et al*. (2003), www.ciria.org

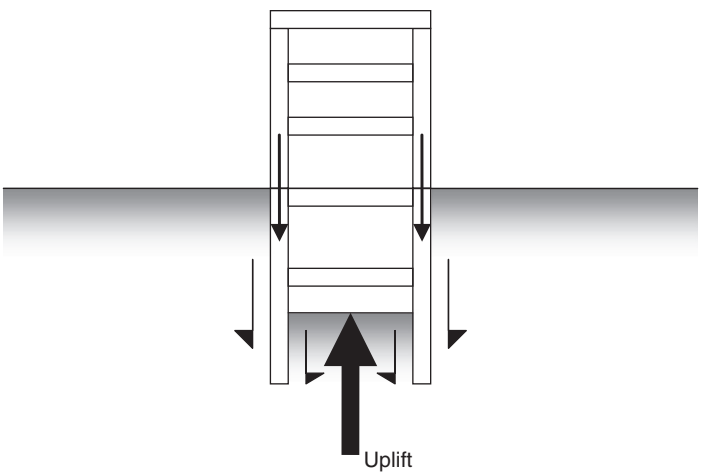


Figure 67.6 Possible wall frictions around excavations subject to long-term uplift

horizontal load in the slabs or props and modifying the bending moment and shear forces in the retaining wall. This should be considered as part of the wall design, but is often overlooked.

67.4.2 Flotation and heave

Uplift actions on underground structures can be caused by pore water pressure acting under the base slab and, in overconsolidated fine-grained soils, by soil heave pressure. The approach to dealing with the uplift pressures associated with these two phenomena should be considered early on in the design process. For both processes, it is possible to either resist or dissipate the forces on the substructure.

Flotation can be accommodated in design by either dissipating under-slab pore water pressures by under-slab drainage, or by resisting the imposed forces by a combination of the building weight and soil friction acting on the walls and any tension piles. Underdrainage can introduce other factors such as increased maintenance and operation costs, and may result in widespread settlement as a result of consolidation and/or loss of fines which could affect both the basement structure and adjacent third parties.

Possible methods of providing under-slab drainage are:

- a no-fines concrete drainage blanket beneath the base slab blinding;
- a series of shallow *French drains* beneath the base slab blinding;
- a number of passive wells, discharged by artesian pressure into the base slab drainage system;
- a number of active wells, discharged by pumping.

Soil heave pressure dissipates as upward movement is allowed (see **Figure 67.7**) and can therefore be accommodated in design by allowing the soil to move upwards. This can be achieved either by specifying a heave void or by allowing the structure to rise upward (which may not be appropriate from a serviceability perspective). If required, all or some of the heave pressure can be resisted using a combination of the building weight and soil friction acting on the walls and any tension piles.

It should be noted that where the designer opts to dissipate the uplift/heave forces using a drainage blanket or heave void, the effective stress under the base slab can be reduced, resulting in reduced vertical load capacity of the walls and piles.

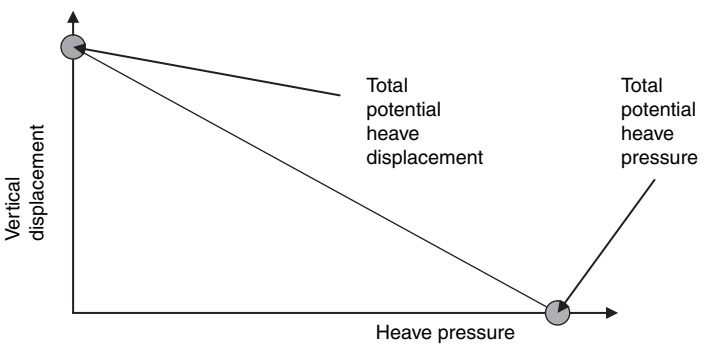


Figure 67.7 Dissipation of soil heave pressure with vertical displacement. Note: assumes linear elastic behaviour for simplicity

67.4.3 Design of ground-bearing base slab (for vertical downward loading and to resist heave and water pressures)

Aside from the geotechnical design of the base slab as a foundation, it is important to consider how it functions as part of the overall structure. The designer should consider the robustness of the whole structure, including connections. The base slab connects the walls to the ground below, and also to any additional piled foundations. Interaction of these various elements must be considered in design.

The base slab must be strong enough to resist the upwards flotation and heave forces, restrained by the self-weight of the box structure together with any tension forces mobilised through shaft friction in the piles and walls. It must also be able to distribute any concentrated applied loads to the supporting ground.

Provided that the appropriate overall ULS and serviceability limit state (SLS) considerations are satisfied, an economic foundation which just satisfies global resistance to uplift (or downwards) loading may well have individual pile or wall elements with a lower factor of safety.

The base slab must be strong enough to safely redistribute the loading in the scenario where the individual pile or wall elements only just achieve (or less obviously, achieve more than) their theoretical working load resistance. For example, piles that perform significantly in excess of predictions may settle less under applied loads, and therefore act as stiff points, potentially attracting further load. A robust consideration of the potential relative stiffnesses of the foundation system can be used to envelope the possible responses.

In addition to the strength of the base slab, the connection between the slab and the walls must be sufficiently strong to transfer all of the possible loads between the walls and the foundation structure.

67.5 Design of bored piles and barrettes to support/resist vertical loading beneath base slab

Where piles or barrettes are used as a foundation solution within a basement, their design has additional considerations compared to the design of piles with a cut-off level at or close to the ground surface. Excavation of the basement causes two phenomena which must be allowed for in pile design: (i) a reduction in overburden stress and hence ground strength; and (ii) heave-induced ground movements.

The change in the horizontal stress state ($\Delta\sigma'_h$) in the ground following a reduction in overburden stress can be calculated by considering an equation of the type:

$$\Delta\sigma'_h = \left(\frac{\nu}{1-\nu}\right)\Delta\sigma'_v$$

where ν is Poisson's ratio, and $\Delta\sigma'_v$ is the change in vertical effective stress due to excavation.

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Piles beneath deep basements should be designed using effective stress parameters, as total stress design does not account for strength changes (unless, for example, c_u was measured after basement excavation). A detailed discussion of the effective stress approach to pile behaviour and design is given in Chapter 22 *Behaviour of single piles under vertical loads*.

Where piles are installed within a deep basement, pile reinforcement typically should extend full depth to resist heave forces. Consideration of the relative displacement and capacity of the pile shaft can be used to reduce the length of cages where the piles are not required to provide additional tensile capacity.

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67.6.1 Further reading

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67.6.2 Useful websites

CIRIA; www.ciria.org

Eurocodes; www.eurocodes.co.uk and www.bsigroup.com/templates/FourTabContent.aspx?id=147778&epslanguage=EN

ICE Specification for Piling and Embedded Retaining Walls; www.thomastelford.com/books/bookshop_main.asp?ISBN=9780727733580

It is recommended this chapter is read in conjunction with

- Chapter 2 *Foundations and other geotechnical elements in context – their role*
- Chapter 26 *Building response to ground movements*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

Section 7: Design of earthworks, slopes and pavements

Section editor: **Paul A. Nowak**

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Chapter 68

Introduction to Section 7

Paul A. Nowak Atkins Ltd, Epsom, UK

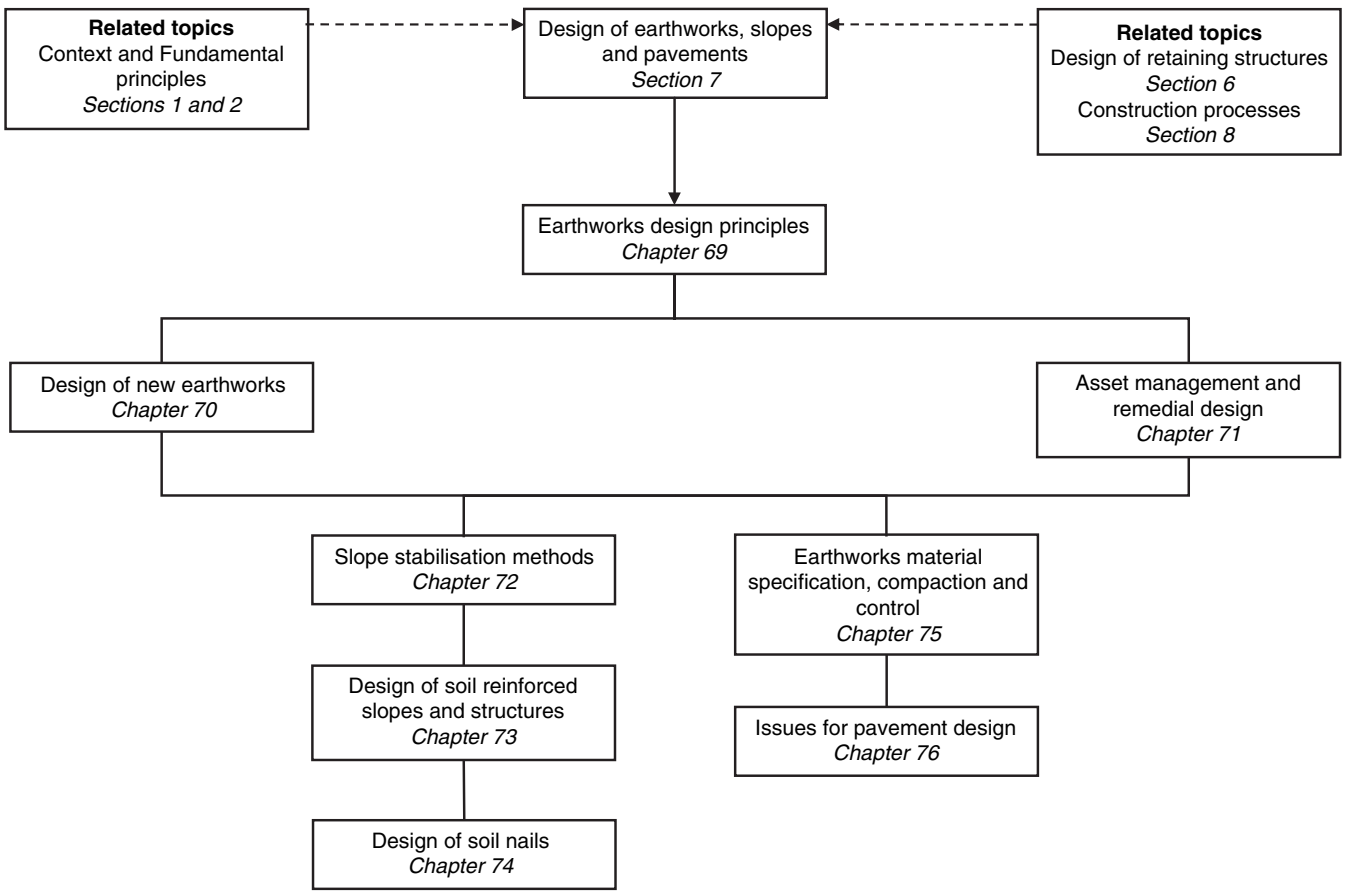


Figure 61.1 Relationships between the chapters in Section 7

Man has constructed earthworks in Europe since the Stone Age, but a major expansion of activity occurred in the nineteenth century with the development of the railways, in order to meet operational maximum gradient requirements.

The development of major road networks in the twentieth century continued this expansion and heralded the extensive use of earthworks plant and formal methods of compaction.

In both cases the earthworks were constructed to provide a balance between excavated and deposited material and to provide a stable platform on which the infrastructure can operate.

Current earthworks engineering can be considered to have three major streams:

- design and construction of new build routes;
- alteration or widening of existing earthworks;
- assessment and maintenance of existing assets.

The design of earthworks, although predominantly engineering, has always demanded a feel for material that is essentially heterogeneous and of varying properties, and the balance has changed over time. Much of earthworks construction occurred before the formal advent of soil mechanics theory in relation to earthworks engineering. Its development in the UK through the 1950s to the 1970s forms the basis of the concepts used in modern-day design. It grew in great part due to failures of constructed slopes on the railway network and slopes under construction on the trunk road network, and the need to understand failure mechanisms to prevent further earthworks failures.

This development was in parallel with that in the USA, which was driven mainly by failures of embankment dams such as Teton Dam.

Although the failure mechanisms were generally different in the UK (the presence of relict slip features in new build failures and pore water pressures in existing earthworks slopes) compared with the USA (internal erosion of embankment dam shoulders and core) the same theories were established which has allowed the development of more rigorous analysis as soil mechanics has evolved. It is interesting to note that, in both cases, the presence of water generally had a great influence on the instability.

Traditional earthworks design, where input assumptions have been tested against a varying final factor of safety against failure, have been replaced over the last 10 years by the design approach of Eurocode 7, the use of which was written into EU law in April 2010. This borrows the concept of partial factors on input parameters from structural engineering and represents a change in thinking, where the choice of input parameters demands more focus, although it does not detract from previous practice. Its introduction has provoked extensive use of this new approach but has not fully replaced the use of the final factor of safety approach.

Overall earthworks design usually considers ultimate limit state failure but, increasingly, in the assessment of existing assets, serviceability limit state behaviour is an important consideration, as 'failure' can be defined as loss of service through unacceptable movement of the earthworks formation.

Chapter 23 *Slope stability* of this manual provides an introduction to slope stability design. **Figure 68.1** outlines the layout and contents of Section 7 *Design of Earthworks, Slopes and Pavement.* This section covers the design of both unreinforced and reinforced earthworks, the assessment of existing assets, the specification of earthworks materials and consideration of pavement design.

The future holds an interesting mix of potential earthworks problems. New build schemes will demand a potentially differing approach from that of the maintenance and operation of existing, ageing assets and their potential replacement. The effects of climate change are likely to produce changes in rainfall pattern and a rise in sea levels. These effects are likely to generate the need for earthworks engineering in the provision of flood defence and sea erosion protection measures.

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Chapter 69
Earthworks design principles

Paul A. Nowak Atkins Ltd, Epsom, UK

Earthworks, in the form of cuttings and embankments, have been constructed for some 4000 years. This chapter presents a brief history of development of analysis methods and the development of factors of safety to the current use of Eurocode 7.

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69.1 Historical perspective

The construction of earthworks in the UK can be traced back some 4000 years when earthworks took the form of buried mounds and mounded features such as Silbury Hill in Wiltshire (Charles, 2008).

The tradition was carried on by the Romans whose roads tended to follow natural contours but contributed with earthworks in the form of barriers like Hadrian's Wall to keep out invading tribes from the Empire.

The bulk movement and placing of earthworks coincided with the Industrial Revolution in the mid 18th century, initially in the construction of the canal network. This comprised the excavation of channels with gradients being overcome by the construction of locks – the most spectacular of which is probably the system on the Leeds–Liverpool Canal to take it over the Pennines.

The advent of the railways from the 1830s required the construction of cuttings and embankments as locomotives could not negotiate gradients of greater than 1 in 50. Initially, all bulk earthmoving was carried out using horse-drawn wagons with the materials being end tipped into embankments, then cut and placed at their angle of natural repose. This form of technology did not detract from large volumes of construction and Skempton (1995) reports the movement of two to three million cubic yards of material per year over the period 1834 to 1841. In the late 19th century, steam driven equipment was introduced but the method of placing of the fill was largely the same.

Embankment settlements could be large both during and subsequent to construction and were controlled by reballasting of the track and the imposition of speed restrictions. McGinnity and Russell (1995) describe London Clay embankment construction on London Underground in the late 19th and early 20th centuries, where settlement during construction was topped up with London Clay and steam locomotive ash.

The development of the roads network in the UK from the 1920s required earthworks gradients of less than 1 in 10 and coincided with large scale mechanisation of earthmoving plant in the United States. The development of mechanised plant allowed the control of fill quantity and the compaction of soil in layers for embankment construction.

69.2 Fundamental requirements of earthworks

The fundamental requirement of an earthwork is to form a stable foundation for the proposed end use over its design life. This usually represents minimal settlement for a road pavement or a railway track bed.

With respect to cuttings, the fundamental requirement is the stability of the cutting side slopes which, if they fail within the design life of the earthwork, may cause the deposition of debris at the toe of the cutting or failure of material at its base, causing damage to the road surface or railway track.

With respect to embankments, failure of side slopes is similarly important as loss of slope could result in undercutting of the asset that the embankment supports. Additionally, the settlement of the embankment over its design life, whilst not necessarily contributing to catastrophic failure, may result in the loss of service of the asset due to unacceptable differential movement. This is particularly the case with respect to railways, where differential settlement between the running rails is usually set at less than 5 mm. Acceptable differential settlement may also vary along the earthwork. The most critical area is usually where an earthwork adjoins a structure. The structure abutment is designed to minimise settlement of the overall structure which will be less than the likely settlement of the embankment adjoining the abutment. This can lead to eventual cracking of the road pavement or loss of horizontal rail alignment. In the short term, it can lead to a reduction in ride quality.

In planning earthworks for a linear transportation scheme, it is usual, where practicable, to endeavour to set the vertical alignment such that the scheme is 'in balance'. This means that the volume of material excavated from cuttings is nearly the same as that required to form embankments. The calculation of overall volume will take account of the amount of cut material that is unsuitable for use as embankment fill. This material is likely to have to be removed from the site. Common practice is now, however, to incorporate unacceptable material into landscape features, or to render it acceptable for use in the works by conditioning. This is dealt with in more detail in Chapter 75 *Earthworks material specification, compaction and control*. In calculating the overall volume of earthworks, bulking of the material from cutting to placing into embankment is sometimes considered. For most earthworks materials, the volume placed in embankments is up to 5% less than the

volume won from cuttings. This is due to the specification of formal compaction and the fact that mechanical placement and compaction in an embankment does not fully replicate the geological process under which the cut material was first deposited. The notable exception to this rule is unweathered chalk which shows up to 5% shrinkage in volume from cut to fill. This is due to the fact that the material occurs naturally with a high voids ratio and open jointing – which are destroyed by the compaction process.

It is common practice to ignore the volume increase due to bulking from cut to fill when calculating earthworks quantities, or to include it, but balance it with the volume of earthwork embankment lost due to settlement of the foundation material.

Overall guidance to the excavation of materials and their compaction into earthworks' embankments for highways schemes in the UK is given in document HA44/91(1995) produced by the Highways Agency.

69.3 Development of analysis methods

It should be noted that most of the earthworks built in the 19th and early 20th centuries were constructed when soil mechanics was in its infancy. Collin (1846) produced a slip circle stability analysis, but it was not until the work of Terzaghi (1925) on the influence of pore water pressure on the deformation and strength of soils, that stability and prediction of failure started to be understood.

In the 1930s the principle of effective stress for earthworks materials began to be understood and a move from empirical to numerate determination of slope stability began to be developed. Over time, a number of methods have been developed in order to ascertain the factor of safety of new and existing earthworks slopes. The rigour of the analysis was, however, was tied to the development of analysis technology and hence in the 1950s and 1960s (before the development of computer analysis programs) more traditional methods of mathematical computation were used. The main methods of analysis can be summarised as:

- infinite slope method;
- stability charts;
- limit equilibrium;
- numerical modelling;
- observation method.

The infinite slope method is a quick computational method developed for failures that occur at shallow depth in cuttings and embankments where the failure surface is assumed to be a plane parallel to the slope surface. The method is described in Skempton and DeLory (1952) and in Trenter (2001).

Stability charts like those by Taylor (1948) and Bishop and Morgenstern (1960) were developed from more rigorous analysis to produce a series of charts and tables where knowledge of shear strength, angle of friction and water conditions allowed a factor of safety to be computed for varying slope angle without further rigorous computation. These charts have been updated by Bond and Harris (2008) for use with Eurocode 7.

The limit equilibrium method was developed as the more rigorous analysis of a series of slices through the proposed slope where interaction between the slices was computed as the analysis method developed. Bishop (1955) developed the method for analysing circular potential failure surfaces. The method was developed by Morgenstern and Price (1965) and Janbu (1972) for analysing non-circular modes of failure.

Computational limitations in the 1950s and 1960s restricted the number of potential failure surfaces that could be practically analysed, and work carried out by Skempton (1964) and Chandler (1972), among others, tended to concentrate on back analysis of slope failures where the form of failure surface was already known. The advent of computer analysis in the 1980s has allowed these analysis methods to be applied to everyday earthworks analysis, with many failure surfaces analysed in the same computational run.

The development of finite element and finite difference analysis techniques in the 1990s has allowed more rigorous analysis of earthworks slopes to be computed from the limit equilibrium methods described above. These techniques model elements bounded by nodes and allow both ultimate failure conditions and movement of slopes, during their design life, to be determined.

The observational method of earthworks slope analysis, summarised by Nicholson, Tse and Penny (1999), represents an approach where design assumptions are reviewed during the construction and design life of an earthwork by use of instrumentation. The principles of the observational methods can be summarised as:

- Establishment of acceptable limits of behaviour.
- Assessment of the range of possible behaviour showing that there is an acceptable probability that the actual behaviour will be within acceptable limits. A plan of monitoring is devised which will determine whether actual behaviour lies within acceptable limits.
- The response time of the instrumentation and proposals for analysing the results shall be sufficiently rapid in relation to the possible evolution of a failure mechanism.
- Adoption of a plan of contingency actions which may be enacted if the monitoring reveals behaviour outside acceptable limits.

More information on the analysis and stability of natural and man-made earthworks can be found in Chapter 23 *Slope stability*.

69.4 Factors of safety and limit states

69.4.1 Factors of safety

Traditionally, the factor of safety against failure of an earthworks slope has been considered as a target 'lump' factor which should represent the minimum value that an analysis of failure can achieve.

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Trenter (2001) comments that the chosen minimum factor of safety will depend on two sets of circumstances:

- technical assessment of the geotechnical data collected for a potential slip;
- judgement on the safety, environmental and economic costs of any failure.

BS6031 (1981) gives minimum factors of safety of 1.3–1.4 for first time failures, and 1.2 for failures involving ancient shear surfaces.

Trenter, considering environmental, economic and safety risks, suggests the minimum factors of safety for first time and reactivation failures of cuttings and embankments; these are summarised in **Table 69.1**.

The factors of safety for new dam construction are shown in **Table 69.2** (Johnston *et al*., 1999).

Other studies have considered a variable factor of safety for earthworks stability considering design soil parameters and groundwater/loading conditions. Egan (2005) considered the acceptable factors of safety for Network Rail earthworks shown in **Table 69.3**.

Perry *et al*. (2003) considered the variable minimum factors of safety for infrastructure embankments (see **Table 69.4**).

As can be seen from this table, the choice of minimum factor of safety lies with the designer and generally reflects the choice of design soil parameters, the application of loading and groundwater conditions, and the influence of failure on the infrastructure asset. These aspects are discussed in greater depth in Chapter 70 *Design of new earthworks*.

The implementation of BS EN 1997–1, Eurocode 7 (2004) introduces the concept of partial safety factors to input parameters, rather than the use of a final target 'lump' factor for the design of earthworks slopes.

The required condition to be met by the analysis is:

$$E_d \leq R_d$$

where E_d is the design value of the effect of actions, and R_d the design value of the resistance to an action.

The National Annex to BS EN 1997–1 for the UK (2006) adopts Design Approach 1 which requires the computation of two load combinations:

(i) Combination 1 = A1 + M1 + R1

(ii) Combination 2 = A2 + M2 + R1

where A = actions, M = materials, and R = resistance. The partial safety factors to be applied are given in **Table 69.5**.

Analysis to Eurocode 7 is usually reported as satisfying the condition above, i.e. $E_d \leq R_d$.

Bond and Harris (2008) introduce the concept of a degree of utilisation where the analysis result is reported as a percentage of the design value of the resistance to an action, rather than a factor of safety.

69.4.2 Limit states

The calculation of a satisfactory factor of safety against failure, or satisfactory achievement of design value of resistance to Eurocode 7, should result in a stability state where overall failure of the slope is unlikely to occur. This can be considered to be the ultimate limit state failure of the slope.

Conditions can, however, occur where the earthwork does not reach its ultimate failure state – movement occurs which affects its performance. This can be considered to be a serviceability limit state condition and is controlled by the

	Factor of safety (first time failure)	Factor of safety (reactivation failure)
Cuttings		
Permanent	1.30–1.50	1.10–1.30
Temporary	1.10–1.30	>1.0–1.20
Embankments		
Permanent	1.40–1.60	1.30–1.50
Temporary	1.20–1.40	1.10–1.30

Table 69.1 Typical factors of safety for cuttings and embankments
Data taken from Trenter (2001)

Loading Condition	Typical minimum acceptable factor of safety
End of construction	1.3–1.5
Steady seepage with reservoir full	1.5
Rapid drawdown	1.2

Table 69.2 Typical factors of safety for new dam construction
Data taken from Johnson *et al*. (1999)

Failure class	Factor of safety: Moderately conservative peak parameters	Factor of safety: Moderately conservative residual soil parameters
1. Affecting track and lineside services	1.3	1.1
2. Affecting earthwork	1.2	1.1
3. Deeper failures	Not less than preworks condition	1.1

Table 69.3 Typical factors of safety for Network Rail embankments
Data taken from Egan (2005)

	Worst credible	Moderately conservative
Shallow failure	1.05	1.15
Deep failure	1.10	1.30

Table 69.4 Typical factors of safety for infrastructure embankments
Data taken from Perry *et al*. (2003)

Action	Symbol	Set A1	Set A2
Permanent – unfavourable	Y_G	1.35	1.0
Permanent – favourable	$Y_{G,fav}$	1.0	1.0
Variable – unfavourable	Y_Q	1.5	1.3
Variable – favourable	$Y_{Q,fav}$	0	0
Soil parameter		M1	M2
Angle of shearing resistance (tan φ)	Y_φ	1.0	1.25
Effective cohesion (c')	$Y_{c'}$	1.0	1.25
Undrained shear strength (c_u)	Y_{qu}	1.0	1.4
Unconfined strength (q_u)	Y_{qu}	1.0	1.4
Weight density (Y)	Y_Y	1.0	1.0
Resistance		R1	R2
Earth resistance	Y_{Re}	1.0	1.0

Table 69.5 Partial safety factors for analysis to Eurocode 7

performance of the infrastructure that the earthwork supports. Common examples of this condition are:

- creep of cutting slopes, resulting in unacceptable movement of services, e.g. drainage, electricity or communications cables;
- settlement of embankment foundations or creep of embankment slopes, resulting in unacceptable deflections of railway tracks or highway surfacing/services.

BS6031 (2009) recommends that when considering deformation of earthworks, it is appreciated that they can undergo large deformations without detriment to their own serviceability, although the effect of deformations on shear strength may be sufficient to cause failures at ultimate limit state as shear surfaces develop. It is important to consider the effect of deformation of structures adjacent to or supported by earthworks, as these may control the overall earthworks design.

Deformation of earthworks over their design life can be determined by numerical methods of finite element or finite difference modelling outlined in section 69.2 above.

69.5 References

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It is recommended this chapter is read in conjunction with

- Chapter 70 *Design of new earthworks*
- Chapter 71 *Earthworks asset management and remedial design*
- Chapter 94 *Principles of geotechnical monitoring*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 70
Design of new earthworks

Paul A. Nowak Atkins Ltd, Epsom, UK

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The design of earthworks needs to consider the interaction of a number of factors to understand their stability and performance over time. These include soil design parameters, groundwater conditions, external loading and the effects of vegetation. Additionally for embankment design the nature of fill material and the competence of the bearing stratum needs to be considered. This chapter introduces these concepts in the framework of design of cutting slopes and embankments.

An introduction to the mechanics of natural and earthworks slopes is presented in Chapter 23 *Slope stability* and an introduction to earthworks slopes can be found in Chapter 69 *Earthworks design principles*.

70.1 Failure modes

70.1.1 Introduction

The design of earthworks is predominantly driven by the possibility of failure of the completed earthworks slope. The failure of earthworks can generally be considered to fall into two categories:

- failure during or soon after construction;
- failure during operation through the design life of the earthwork.

Although a new earthwork would be analysed in the same way to take account of both types of failure the mechanisms of failure are fundamentally different as they are the result of a different set of circumstances.

Most modern earthworks are constructed to fulfil a 60-year design life (*design life = period without significant maintenance rather than failure after the period of the design life*) but throughout the UK infrastructure earthworks have been in place for up to 150 years with no significant signs of distress.

70.1.2 Types of slope failure

The main types of potential slope failure are summarised in **Table 70.1**. The failure mechanisms are shown in **Figure 70.1**.

The design against failure of a new earthworks slope is generally by limit equilibrium methods described in Chapter 69 *Earthworks design principles* and tends to concentrate on rotational circular failure, after Bishop, or rotational noncircular/translational failure, after Morgenstern and Price, as these are the analysis methods commonly available in computer analysis programs.

Compound failure, flow slides and slab slides are more difficult to predict as a failure mechanism for new earthworks. The analysis of compound failures tends to be the subject of back analysis of failed or partially failed slopes. The factor of safety against failure by flow slides and slab slides can, however, be evaluated by the infinite slope method or by limit equilibrium methods where shallow circular failures close to the slope surface are similar in shape to shallow flow slides.

Progressive failure is discussed further in section 70.1.4. It is difficult to analyse using limit equilibrium as the mechanism is strain-dependent and empirical adjustments are required to the measured or estimated material strength. It is more suited to finite element analysis methods which can be used to predict the behaviour of strain softening soils. It should be noted that calculation of progressive collapse has not predominantly been developed as a predictive failure tool but as a method of explaining the failure mechanism of failed slopes.

70.1.3 Failure during construction

The failure of earthworks slopes during, and within a short period after construction, are rare in modern earthworks but when they occur they can be attributed to a number of factors.

The failure of cutting slopes can usually be attributed to heterogeneity of the material in which the cutting has been formed such as:

- the presence of geological structures, bedding/laminations/fissures;
- the presence of zones of contrasting permeability, e.g. inter-bedded sands and clays;
- the presence of historical slip surfaces.

The presence of geological structures is more common in rock slopes but where they occur in soil slopes in the form of

Failure mode	Definition	Comments
Rotational slide	Rotation of mass of soil along curved surface	Not usually differentiated between both mechanisms
Circular slide	Rotational slide on a slip surface that is approximately circular	
Rotational non-circular	Rotational slide on a slip surface that is not wholly circular	
Translational	Movement of a shallow mass of soil in a plane roughly parallel to the slope due to a weakness on a plane	
Compound	Movement of soil mass that combines the characteristics of a rotational and translational slide	
Mud flow	Translational slide in saturated soil, caused by a sudden increase in pore pressure, in which the soil flows as a viscous liquid	
Debris slide	Translational slide of debris, forming a mantle on a slope or the disturbed material at the toe of a rotational slide when rainfall or surface water causes downward movement of debris	
Slab slide	Translational slide in which the sliding mass remains more or less intact	Usually occurs in weathered surface of a slope
Progressive failure	Brittle failure of a soil mass by development of a rupture surface which migrates causing failure before the surface fully develops	Not specifically covered by BS EN1997-1 (2004)

Table 70.1 Main types of failure of earthworks slopes

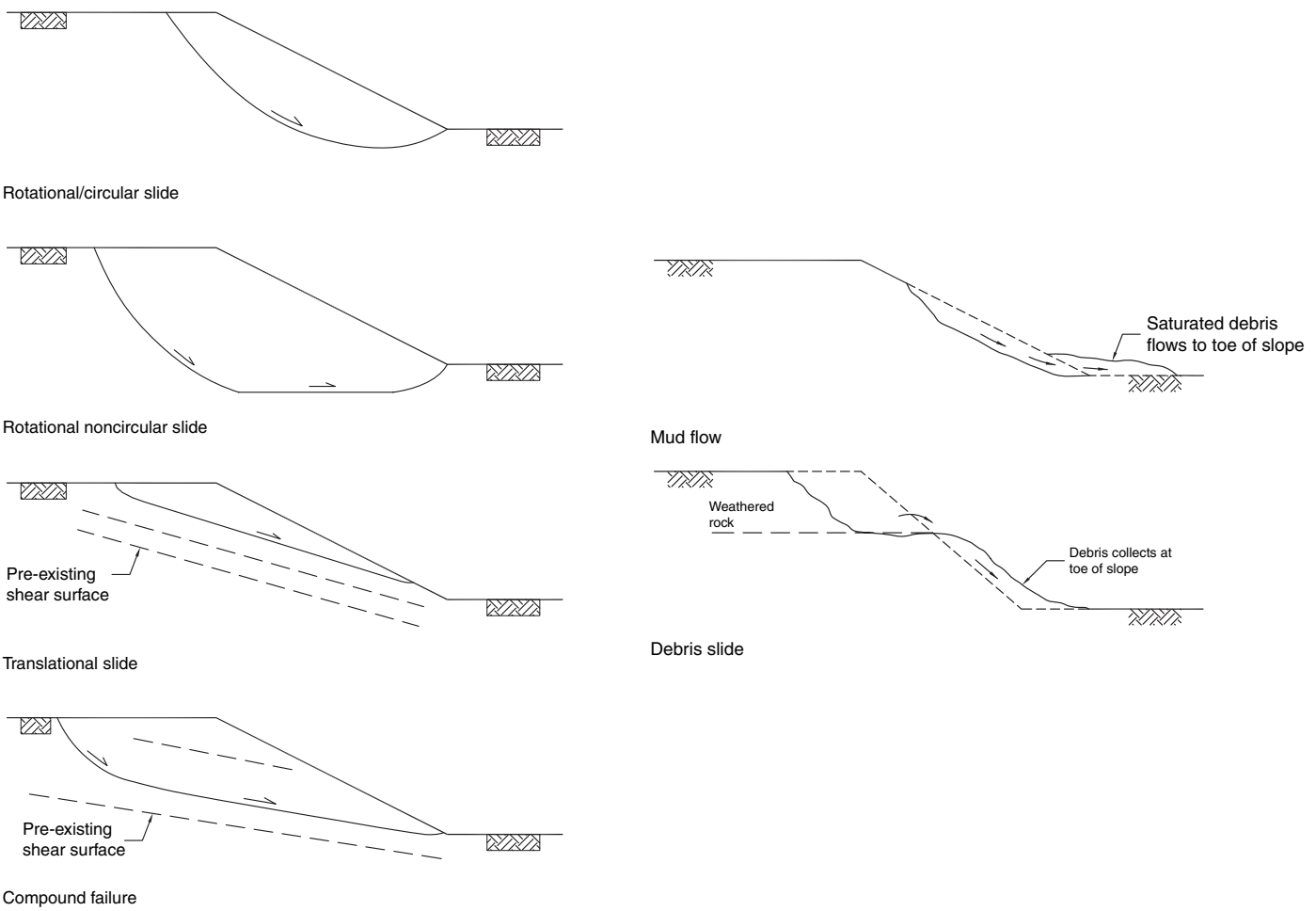


Figure 70.1 Types of failure of earthwork slopes

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bedding or laminations these can form planes of preferential sliding if these surfaces are inclined detrimental to the inclination of the cut face.

The presence of fissures in cohesive materials can reduce material strength over time particularly in overconsolidated clays when pore pressures within a saturated soil mass readjust to new stress conditions.

The presence of zones of contrasting permeability can lead to large movements and slope failures on construction where the groundwater table is high in relation to depth of cutting. Where granular materials are inter-bedded with cohesive layers drainage is impaired and flow of groundwater is possible only through the day-lighting of the granular layers. The presence of groundwater in these layers considerably reduces their strength and flow can occur undermining the surrounding cohesive soils and causing instability of the slope.

The M3 motorway at junction 3 is underlain by Eocene Bracklesham Beds material comprising inter-bedded silty fine sands and clays. The final construction earthworks slopes were designed as 1V:3H. During construction in the 1970s failure of the slope occurred due to flow of groundwater and instability of the sand layers. Stable slopes were eventually formed at 1V:10H following the employment of extensive drainage measures.

The presence of historical slip surfaces is common in areas where cohesive strata are present which have been subject to peri-glacial activity and freeze–thaw action has caused the historical movement of material on shallow slope angles. The presence of the slip surfaces considerably reduces the material strength as described in section 70.2.2.

Failure of cutting slopes during construction occurred in Weald Clay on the A21 Sevenoaks bypass in the 1960s reported by Symons and Booth (1971) and in Gault Clay on the M25 motorway at Godstone in the 1970s. In both cases original cutting slopes were designed without taking reduced strength of relict slip surfaces into account. On the M25 motorway west of London shear surfaces were recognised in London Clay and Reading Beds material (Spink, 1991), and cutting slopes of 1V:6H were designed to prevent failure.

Hughes in Vasilikos (2009) reports a failure of a slope at Dromore on the M1 Belfast to Dublin motorway where a pre-existing shear zone in the Glacial Till caused failure of a 1V:2H cutting slope during a period of heavy rainfall.

The failure of embankments during construction can usually be attributed to:

- slope height, geometry and angle;
- foundation inadequacy;
- the presence of pre-existing shear surfaces in the embankment foundation material;
- change in the nature of embankment construction materials.

Failure due to slope geometry and angle are rare in current construction as adequate tools are available for the design of safe slope angles.

Similarly the adequacy of foundations can be controlled in order to prevent failure of the overall embankment slope and these are discussed further in section 70.2.6.

Historical failures of this type can be identified by lower slopes of low gradient, representing failures during construction. The 12-metre-high Roding Valley embankment on the London Underground Central Line was built in 1903 and comprises 4 metres of London Clay at the base at side slopes of 1V:6H overlain by 8 metres of ash. Contemporary accounts of its construction describe failure of the lower levels of embankment construction, sometimes overnight, due to failure of the underlying alluvial clay foundation.

Change in material type for embankment construction can present similar problems to those encountered in cuttings formed in inter-bedded materials described above but usually manifest themselves as shallow translational slides close to the surface of the embankment.

Construction of the M25 motorway between Egham and the M3 in Surrey in the 1970s comprised embankments up to 10 metres high built of Eocene Bagshot Beds silty fine sand. Some layers of the embankment material were laid at high moisture content which on further embankment loading caused build-up of pore pressure and subsequent loss of strength in these layers leading to sloughing of the embankment slopes with resulting regarding required. Similar failures were experienced during construction of the M65 motorway in Lancashire in the 1990s where an embankment 19 metres high was constructed of layered Glacial Till and fine-grained Glacial Sand.

70.1.4 Failure during operation

The failure of an earthworks slope during operation can be generally sub-divided into two modes:

- shallow failure by translation or shallow circular failure;
- deep failure, usually circular in nature by progressive collapse.

Shallow failures, particularly with respect to the UK highways network, have been studied over the last 20 years in an attempt to understand the mechanisms causing failure in order to minimise maintenance requirements over the design life of the earthwork.

Perry (1989) surveyed 570 km of the then constructed 2700 km of UK motorways. His study relates the angle of constructed slope for cuttings and embankments with the incidence of failure in particular geological strata. It presents a probabilistic and risk-management approach in which varying slope angles in the same material were examined for percentage of failure over the total lengths of earthwork. Stable slope angles were proposed for the various materials in cutting and embankment where incidence of failure was

recorded as less than 5% of total earthwork length. Perry concludes that the failure surface rarely exceeded a depth of 1.5 m with a minimum depth of 0.2 m and a maximum depth of 2.5 m.

Crabb and Atkinson (1991) studied failures in road embankments and cuttings in the south of England and recorded failures to 1–2 m below the slope surface. Reid and Clark (2000) observed similar depths of failure on slopes generally greater than 4.0 m in height. This concurs with the work of Perry where shallow failures were more critical in slopes greater than 5.0 m in height.

Investigation of these failures indicates that the most likely mechanism for failure is one of moisture content or pore pressure increase in the near surface of the embankment slope together with a potential reduction in strength properties of the near surface material. This increase is usually due to infiltration by surface water during and after precipitation and could generally be attributed to the lack or non-performance of slope drainage.

Regular inspection of earthworks of the UK major highway network over the last 15 years has revealed not only shallow failures as described by Perry but also signs of creep and tension cracking on existing slopes. Whilst these mechanisms may not pose an immediate danger to infrastructure users, any lack of maintenance or remediation may cause prolongation of the potential failures in surfaces with more significant volumes of failed material in the long term.

Perry *et al*. (2003b) record that operational serviceability failure of railway embankments is generally slow and insidious, associated with excessive movement rather than overall instability failure. Ultimate limit failure is less frequent but develops if serviceability failure is not addressed. Coppin and Richards (2007) report shallow embankment failures generally do not exceed 2.0 m in depth and are commonly translational although some shallow circular slips can occur.

Operational failure of embankment slopes can usually be attributed to:

- shrink–swell induced due to seasonal moisture changes and vegetation;
- deterioration of drainage;
- presence of water in the embankment over time (discussed further in section 70.2.3).

Work by Skempton after the Second World War on failures to London Underground cuttings, predominantly in London Clay, considered failure some time after construction. The failure mechanism and type was different to that described above, involving deep-seated circular failures which caused a significant hazard to the operation of the railway. Work by Potts (1997, 2000) has postulated a theory of 'progressive collapse' for these types of failure where the fissured, overconsolidated cohesive soil acts in a brittle failure mode with an increase in pore pressure along a failure surface propagating from the toe of the slope. Failure occurs when the strength of the soil on the remainder of the surface above which increased pore pressure is acting is less than the overall weight of material causing it to fail as shown in **Figure 70.2**.

Ellis and O'Brien (2007) have attempted to extend this work as a predictive tool for safe slope angle and slope height where progressive collapse will not occur. Their findings show that progressive failure is unlikely for cutting slopes in London Clay shallower than 1V:3H and less than 8 m in height. For steeper slopes or those greater than 8 m in height progressive collapse may occur up to 125 years after construction.

70.2 Typical design parameters

70.2.1 Derivation of design parameters

Traditionally, for slope stability analysis to a target factor of safety 'best estimate' or 'worst credible' parameters are usually chosen from a data set of laboratory or *in situ* test results or from back analysis of failed slopes.

Selection may be based on statistical analysis but is usually the result of engineering judgement. A typical example is shown in **Figure 70.3**.

The selection of best estimate or worst credible parameters will lead to a different target factor of safety being chosen as an acceptable minimum output from the analysis. From the discussion in Chapter 69 *Earthworks design principles* a value in the range 1.3–1.4 is usually considered acceptable where best estimate parameters are used. Where worst credible parameters are used a minimum value in the range 1.05–1.15 would be acceptable. Worst credible parameters can also be derived from the back analysis of failed slopes as these parameters equate to a factor of safety of just less than 1.0 immediately prior to the failure of the slope.

In deriving design values for slope analysis, the variation in parameters in relation to displacement should be considered as shown in **Figure 70.4**.

Values of c'_{pk}, φ'_{pk} reduce with greater displacement to c'_{cv}, φ'_{cv} and, in the case of cohesive soil where the plasticity index is greater than 25%, to c'_r, φ'_r when movement is sufficient to develop shear surfaces.

Best estimate parameters tend to equate to value between c'_{pk}, φ'_{pk} and c'_{cv}, φ'_{cv} whereas worst credible parameters tend to equate to c'_{cv}, φ'_{cv} or, in the case of cohesive soils with existing shear surfaces, c'_r, φ'_r.

The adoption of *Eurocode 7* (2004) introduces the concept of a set of characteristic values for analysis which are then factored prior to analysis as described in Chapter 69 *Earthworks design principles*.

Clause 2.4.5.2 (P) of BS EN1997-1 defines the characteristic value as 'a cautious estimate of the value affecting the occurrence of the limit state'. BS 6031 (2009) suggests

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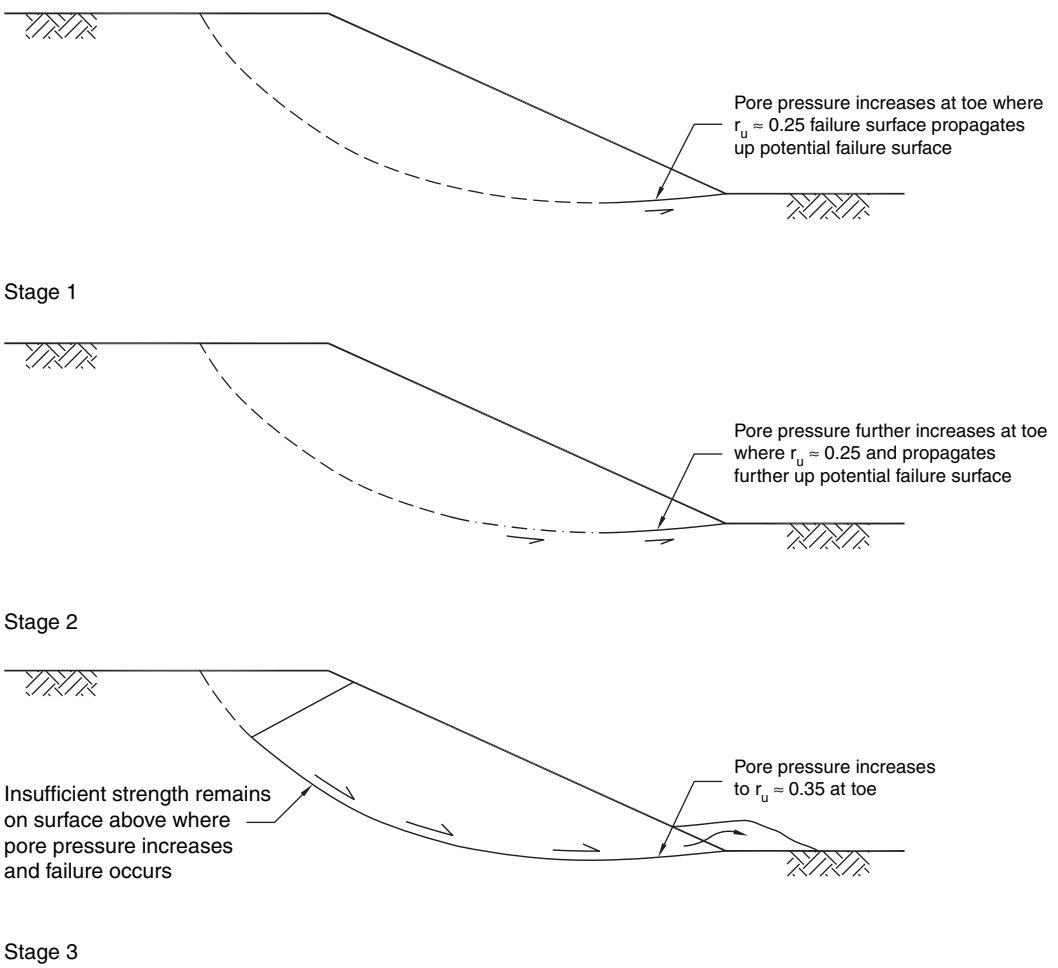


Figure 70.2 Simplified mechanism of progressive collapse

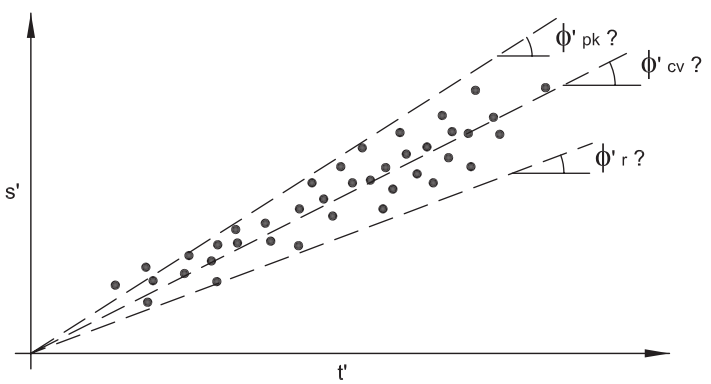


Figure 70.3 Possible selection of design parameters

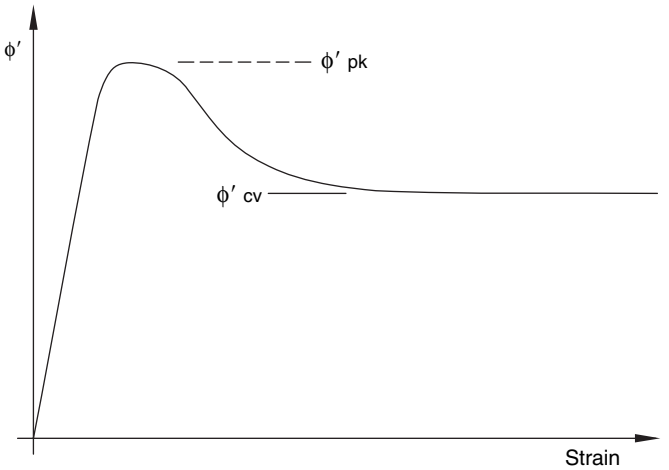
the factors affecting the choice of a characteristic design value are:

- geological, historical and other background data;
- the amount of measured data relating to the parameters under consideration;
- variability and degree of confidence in the measured data;
- extent of the zone of ground governing the limit state under consideration;
- ability of ground to transfer load from weak to strong zones;
- the consequences of failure at the limit state.

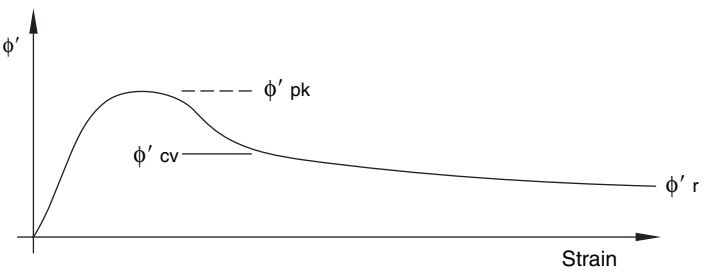
The characteristic value is, therefore, a choice through engineering judgement and although this is likely to equate to a value close to the best estimate value it may tend to worst credible if the consequences of failure are deemed to be exceptionally serious.

The choice of a value tending to worst credible must be viewed in the light of the likely failure mechanism and the fact that design values will have been assigned a subconscious partial safety factor prior to those applied through *Eurocode 7*.

It may be viewed as conservative to generally use lower design parameters for long-term stability of earthworks slopes when, as described in section 70.2.1, likely failure surfaces are shallow with no immediate effect on the overall failure of the earthwork immediately subsequent to failure.



(a) Grangular soils and cohesive soils for which Ip < 25%



(b) Cohesive soils for which Ip ≥ 25%

Figure 70.4 Variations of φ′ with displacement
Modified with permission from BS 6031 © British Standards Institution 2009

Bond and Harris (2008) equate characteristic design values to the 'moderately conservative' value used for design of retaining walls (Gaba *et al*. 2003). This equates to a low cautious average of the design data set. They state that 'The difference between CIRIA 104's moderately conservative value and Eurocode 7's cautious estimate is … merely one of semantics' (Bond and Harris, 2008: 139). This should be treated with caution in light of the discussion above.

Bond and Harris also introduce the theory of an upper and lower characteristic value from a data set. It is considered that the derivation of a characteristic value is unique to a set of ground conditions and the impact of failure of an earthwork as defined in BS 6031 and that a single set of values only should be derived.

70.2.2 Typical design parameters

Guidance on design values where no existing data are available, or where comparison of a data set with existing data and relationships is desirable, is available from a large number of sources some of which are described below.

For cohesive soils, BS 8002 (1994) presents a relationship between φ'_{cv} and plasticity index (Ip). Terzaghi *et al*. (1990) present a similar relationship between φ'_{pk} and Ip. These are presented in **Table 70.2**.

Skempton (1977) reports the following design values from testing and analysis of cutting slopes in London Clay.

$\varphi'_{pk} = 20°$, $c'_{pk} = 14$ kN/m^2, 38 mm samples;
$\varphi'_{pk} = 20°$, $c'_{pk} = 7$ kN/m^2, 250 mm samples;
$\varphi'_{cv} = 20°$, $c'_{cv} = 1$ kN/m^2, back analysis critical state;
$\varphi'_{r} = 13°$, $c'_{r} = 1$ kN/m^2.

It can be seen that φ'_{pk} and φ'_{cv} remain the same and that the difference in design value results from a reduction in cohesion (c'). Trenter (2001) recommends a similar variation in c' relationship with maximum c' of 2 kN/m^2 for fissured and structured overconsolidated clays and maximum c' of 10 kN/m^2 for non-structured clay.

Crabb and Atkinson (1991) report φ'_{cv} values from a study of shallow failures of embankments and cuttings in south-east England where failure was recorded at a depth of 1–2 m below slope surface. These are summarised in **Table 70.3**.

It should be noted that the results reported by Crabb and Atkinson do not entirely correlate with the results reported by Skempton or the Ip versus φ'_{cv} relationship in BS 8002. This indicates a scatter of results which is reported by Terzaghi *et al*. (1990). By way of illustration, **Table 70.4** presents a range of values reported for London Clay.

Soil design parameters, derived from previous analysis and testing, are available for selected strata in the literature listed in **Table 70.5**.

Undisturbed samples of granular soils are more difficult to obtain from ground investigation and design parameters tend to be taken from *in situ* testing, predominantly the standard penetration test (SPT) or cone penetration testing. Guidance on derivation of design parameters from *in situ* test results is

Ip (%)	15	30	50	80
φ′cv (°)	30	25	20	15
φ′pk (°)	34	28	25	22

Table 70.2 Comparison of the relationship between Ip and $\varphi'_{cv}/\varphi'_{pk}$ for cohesive soils

Stratum	Ip (%)	φ′cv (°)
Gault Clay	21–22	23
Oxford Clay	32	25
London Clay	26	25
Reading Beds	34	19
Kimmeridge Clay	27	22
Weald Clay	26	24

Table 70.3 Comparison of Ip and φ'_{cv}
Data taken from Crabb and Atkinson (1991)

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Source	c′ (kN/m²)	φ′ (°)
Cripps and Taylor (1981)	12–18	17–23
(weathered) (unweathered)	31–252	20–29
Potts *et al.*	0–20	20
LUL Standard 1-054	2	21
BAA Terminal 5, Heathrow	0–8	23
BAA M11/A120 Junction	0–5	25

Table 70.4 Comparison of c′/ φ′ for London Clay

Reference	Stratum
Cripps and Taylor (1981)	Mudrocks
Davis and Chandler (1973)	Mercia Mudstone
Chandler and Forster (2001)	Mercia Mudstone
Chandler (1972)	Lias Clay
Hight *et al.* (2004)	Lambeth Group
Lord *et al.* (2002)	Chalk

Table 70.5 Useful literature sources for design values of cohesive strata

provided in the literature from sources such as Clayton (1995) and Lunne *et al*. (2002). Design values can be derived from shear box testing (see **Figure 70.6**), but only if the *in situ* density of the material is replicated by the test.

The relationship between φ′ and SPT from commonly used relationships like Peck *et al*. (1974) will generally give φ′ values in the range 28° to 43°. These are usually acceptable for derivation of design values for cutting stability.

It should be noted, however, as described in section 70.1.3 above that the density of granular materials will be influenced by the presence of groundwater. This is particularly the case for silty fine-grained granular soils where removal of overburden below the groundwater table will effect considerable loosening of the soil structure. During the construction of the M25 motorway Bagshot Beds strata, comprising silty fine sand with an *in situ* SPT N-value of greater than 50, were seen to 'flow' when excavated below the water table. This is particular concern for the short-term stability of cutting faces in fine-grained granular soils before the groundwater table has reached its new equilibrium level subsequent to the formation of the cutting face and may require dewatering measures to be installed during construction to prevent short-term failure.

Where embankment construction comprises predominantly granular materials it is not normally common practice to test the density of the compacted material *in situ* and normally a conservative estimate of the φ' design value is taken. The London Underground Standard 1-054 gives a design value φ′ of 35° for granular embankments composed of well-graded sandy gravel of Alluvial Terrace gravel. The implementation by the Highways Agency in the UK of BA 42/96 (Highways Agency, 1996) for the design of backfill behind integral bridge abutments has focused the minds of materials engineers to provide a low φ′ material. Recent experience of shear box testing on well-graded granular materials for this purpose indicates that, when compacted to the required density, design φ'_{peak} is in the range 40° to 45°. The approach of BA42/96 has now been largely superseded by British Standard Document PD6694-1:2011 (BSI, 2011), which advocates the use of φ'_{max}, after Bolton (1986).

70.3 Pore pressures and groundwater

Pore pressures within an earthwork will vary with time and can influence the factor of safety against failure. This is shown in **Figure 70.5**. At a point A beneath an embankment foundation pore pressure increases initially on loading and then reduces as dissipation occurs over time increasing the factor of safety against failure over the design life of the earthwork. This is particularly marked in embankment foundations comprising cohesive materials. At points B and C, in the slope of an earthwork, pore pressure suctions act initially and dissipate with time resulting in a decrease in the factor of safety.

The consideration of groundwater in slope design is usually modelled as:

- adoption of a pore pressure ratio, r_u;
- modelling of a defined groundwater table.

The concept of r_u was developed in conjunction with the investigation of failures of existing earthworks slopes in order to simulate pore pressure conditions on a known failure surface by back analysis. Early soil mechanics, prior to the development of the computer, investigated conditions on a single or a small number of potential failure surfaces generally by hand calculation.

It must be borne in mind that the adoption of an r_u value is unique to a given failure surface and its use in computer analysis programs will generate a variable groundwater table with each potential failure surface generated. This can lead to conservative modelling of groundwater particularly when a free-draining material overlies a material of low permeability.

Exploratory hole data will usually allow the modelling of a defined groundwater table. The recorded groundwater level will, however, be influenced by the permeability of the soil. Whereas higher permeability sands and gravels are likely to indicate equilibrium groundwater level as flow into an exploratory hole during its construction the natural groundwater level in cohesive soils may not equilibrate for some time and accurate prediction will only be achieved from readings taken from piezometer installations.

In the modelling of the groundwater profile for long-term stability of earthworks slopes the effect of the earthworks construction should be considered with respect to long-term groundwater conditions.

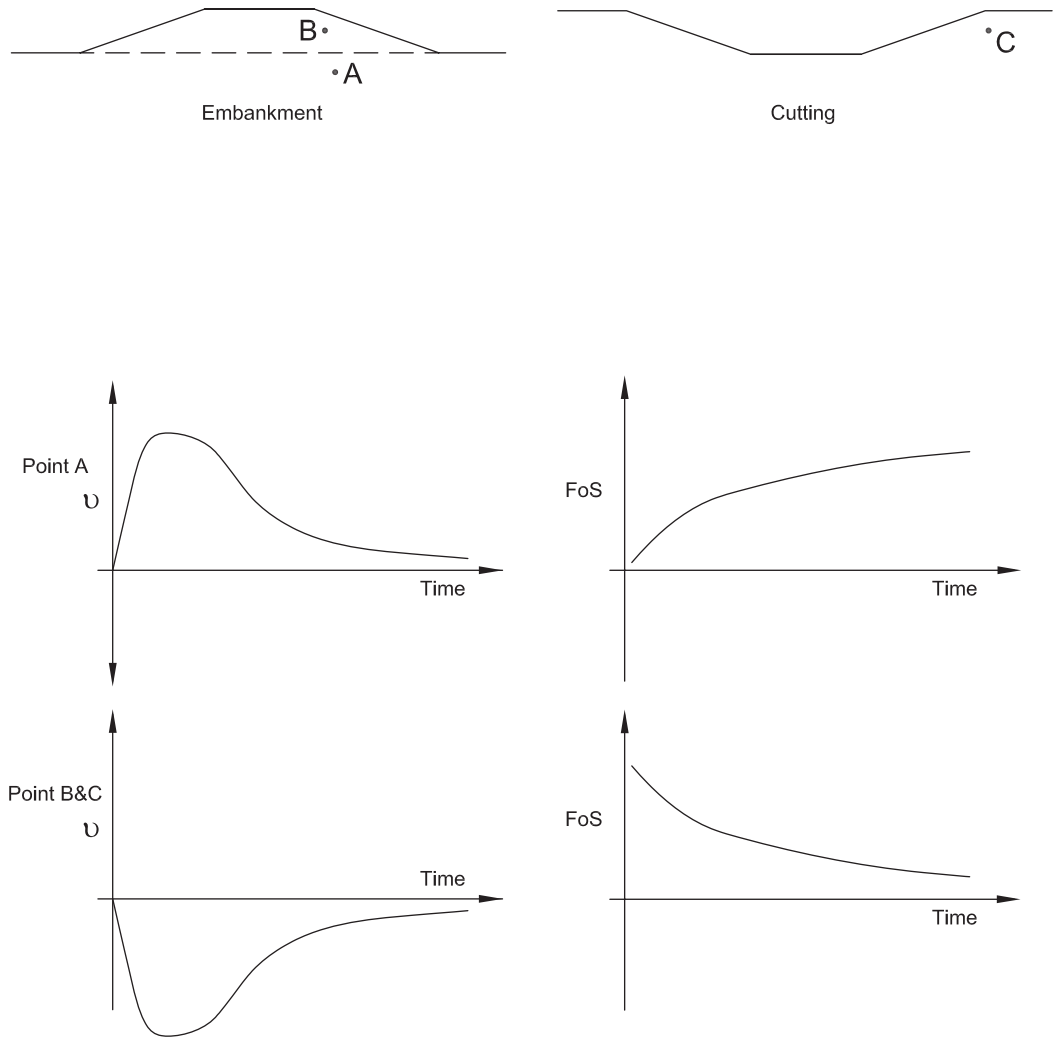


Figure 70.5 Change in pore pressure with time

Where cuttings are formed in materials where the original groundwater profile is above final finished road level this will be influenced by drainage installed at the base of the cutting to develop a revised profile over time depending on material permeability. This is illustrated in **Figure 70.6**.

Where the groundwater level is below the base of the cutting it is common to allow for a presence of groundwater in the cutting slope above the groundwater table. Farrar (1978) suggests a value of r_u in the range 0.1–0.3 should be used for slopes in cohesive materials. London Underground (2000) also suggest a similar approach for London Clay slopes with a base value of 0.25 within a range 0.15–0.35. The lower value is employed where slopes are underdrained and the higher value where no slope vegetation is present.

Groundwater conditions for embankment design differ from those in cuttings as the groundwater table will generally occur in natural ground below the embankment construction. It is usually accepted that some percolation of precipitation onto the embankment will occur over time and a nominal r_u value in the range 0.05–0.1 is adopted throughout the embankment material.

In adopting any groundwater conditions within the embankment the nature of the constructed embankment material and the likely infiltration of precipitation should be considered.

Where the embankment construction comprises granular material the permeability of the material is high and any percolating water will drain relatively easily. Where the embankment construction comprises cohesive material percolation will be slower but once saturated the embankment material will take much longer to drain.

When considering infiltration into an embankment its use should be taken into account. A highway embankment will be capped by an impermeable surfacing and possibly verge drainage which will preclude the infiltration of precipitation apart from on the shoulders of the embankment. A railway embankment is normally capped with permeable ballast to support the

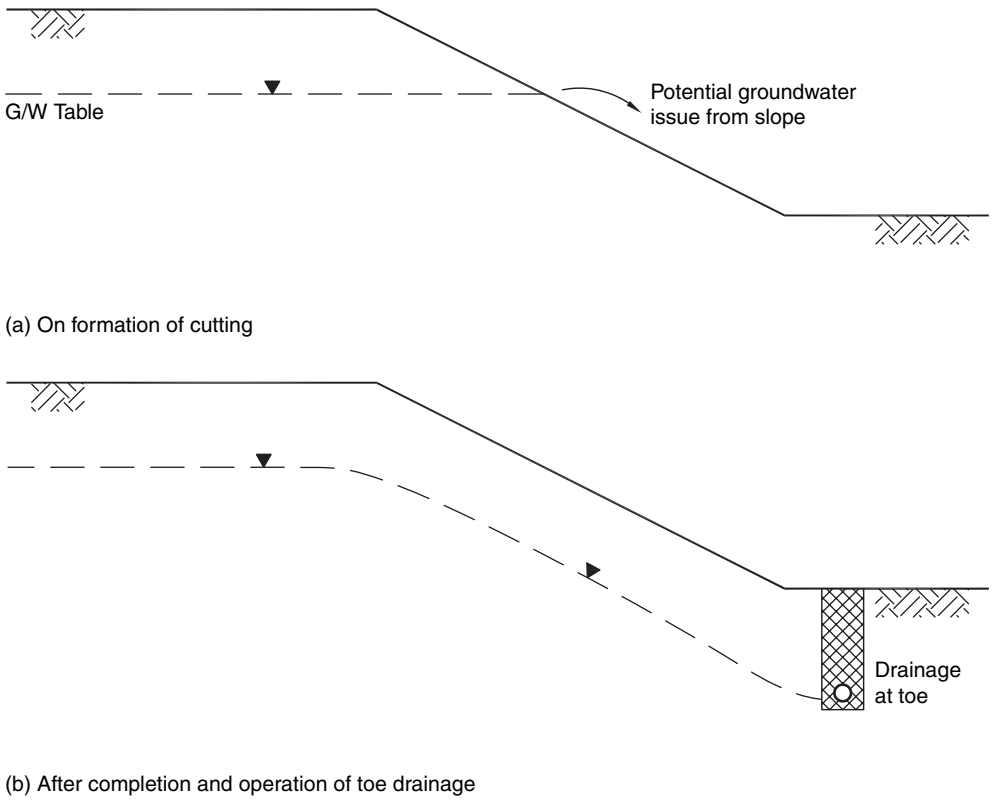


Figure 70.6 Influence of drainage of long-term groundwater level in cutting slope

tracks and, hence, infiltration can occur across the whole area of the embankment. This is shown in **Figure 70.7**.

It is common to simulate the increase of moisture content close to the slope surface, described in section 70.1.4 by an increase in r_u value in the upper 1.5–2.0 metres of the slope surface. A typical value of 0.2 is suggested by Jewell (1996) which is similar to the range 0.1–0.3 suggested by Farrar (1978).

Design of earthworks should consider the provision of drainage both to prevent the flow of surface water onto or below the newly formed earthwork and also to control the flow of groundwater.

The prevention of surface water flow is usually critical to cutting slopes. It takes the form of pre-earthworks drainage as it is usually constructed prior to the main earthwork slope and normally comprises the construction of a cut-off ditch at the crest of the cutting. This both collects run-off water from adjacent areas and can also collect any field drainage that may be intercepted as a result of the new cutting.

Pre-earthworks drainage in areas of embankment construction usually comprises measures to deal with existing watercourses and ditches in order that the earthwork can be constructed without detriment to the existing groundwater regime. This normally takes the form of new collector drains or culvert construction.

Interception of groundwater in a cutting face and its subsequent lowering may be critical to the short-term, construction, life of the earthwork or its long-term stability. Construction of permanent dewatering usually takes the form of gravel-filled trench drains in parallel or herring-bone pattern. Positive pumping from well points or deep wells is commonly employed for dewatering of temporary earthworks slopes and excavations. Long-term adoption of these measures is, however, normally impractical as a permanent measure due to the cost of operation and maintenance.

More details on the aspects of drainage and dewatering design are given in BS 6031 (2009), Hutchinson (1977), Preene *et al.* (2000) and Sommerville (1986).

Any drainage design should consider the discharge of collected water. This can be into existing watercourses or pipework, possibly through ponds or attenuation facilities. Alternatively a sustainable drainage system (SUDS) can be designed. Details of SUDS systems can be found in CIRIA Report C697 (Woods and Kellagher, 2007). In all cases, discharge should be agreed with the relevant authorities, for example the Environment Agency and water companies.

70.4 Loadings

Two types of loading should be considered in the design of earthworks slopes, namely:

- permanent loading;
- transient loading.

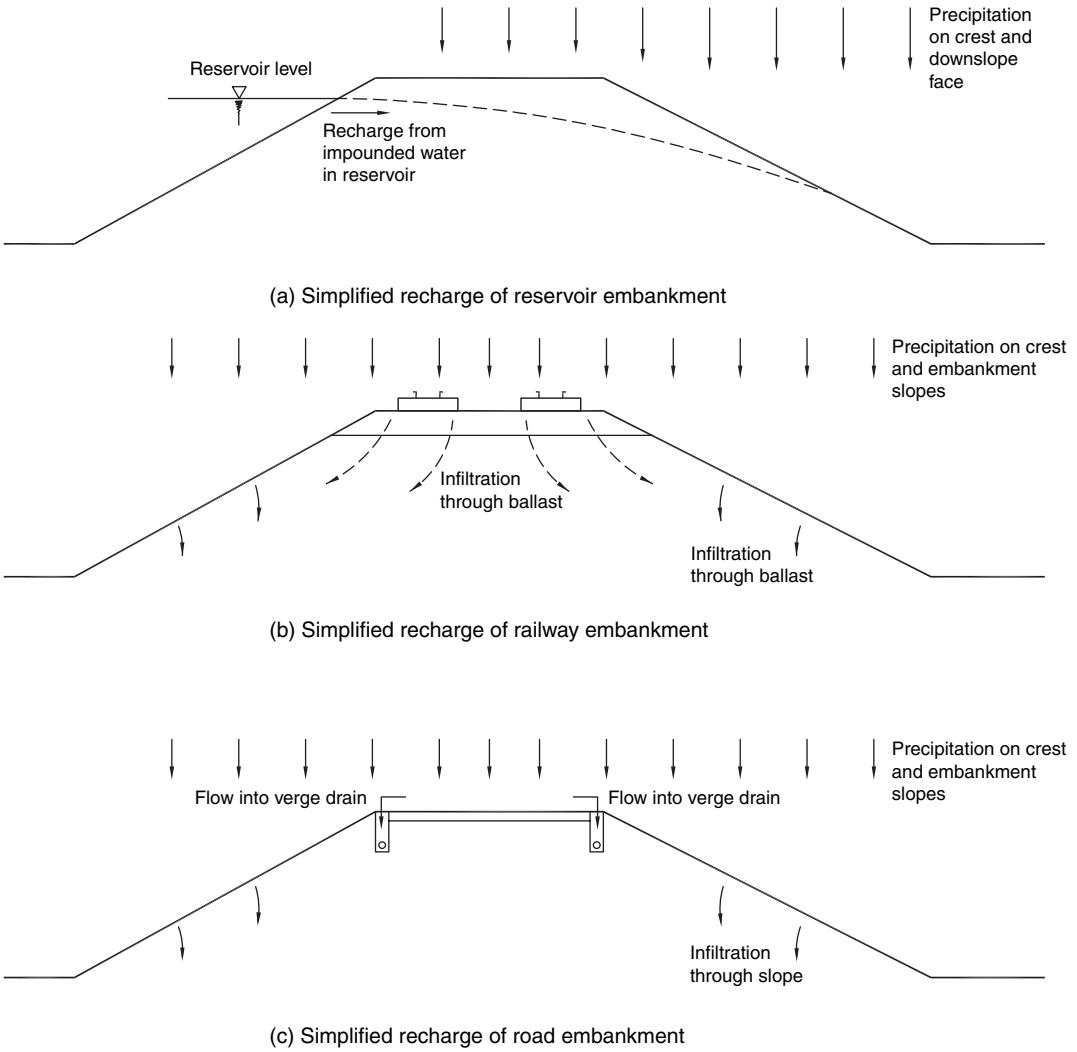


Figure 70.7 Recharge into different types of embankment

Permanent loading will be applied from buildings adjacent to cutting slopes and abutments of railway and highway structures that are constructed mid-height on the cutting slope.

Transient loads can be applied to earthworks in the form of:

- construction loads;
- stored materials/maintenance loads;
- road/rail loading on embankments.

It is usual to apply transient loads only where they contribute to the potential failure of an earthworks slope. Where transient loads may be of benefit to the stability of the slope they should be ignored.

Construction loads will influence the stability of a slope in the short term and are generally applied as a uniformly distributed load (UDL) at the top of cuttings and embankments. It is common practice to set loading from construction plant at 20 kN/m^2 for normal loading conditions. Where large items of construction plant are working at the top of an earthwork individual calculation of the applied load should be based on the weight of the plant from manufacturers' data. It should be borne in mind that large items of plant will generally operate on a construction mat comprised of granular material which will distribute the applied plant load as a UDL at the base of the mat but will also impart load in the short term.

Where materials may be stored or maintenance carried out to the earthwork over time it is usual to apply a 10 kN/m^2 UDL to take account of this eventuality. As stated above, this load would only be applied in areas where it may contribute to the failure of the slope, e.g. at the top of a cutting slope. The application of this transient load is not normally applied to cutting or embankment slope faces as significant maintenance activities or material storage is unlikely to occur at these locations.

Design of infrastructure earthworks require the application of traffic or rail loading to the top of embankment construction

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Standard load	UDL (kN/m²)	Application
Highways Agency HA	10	Normal roads
Highways Agency HB	20	Trunk roads and motorways
Highways Agency Exceptional	37.5	Routes with exceptional traffic
Rail RL	30	London Underground and light rail
Rail RU	50	UK railway standard

Table 70.6 Design loads according to BS 5400 Part 2
Data taken from BS 5400 (2002), British Standards Institution

as detailed in **Table 70.6**. Information on road and rail loading is also contained in Eurocode 1 (2008, 20010).

In applying these loads for the calculation of stability of new earthworks the following should be noted:

- The loads are derived from British Standard BS 5400, Part 2 (2002), which is the design standard for steel, concrete and composite bridges, and from Eurocode 1. In the application of the loads to structures they are in direct contact with the structure and are applied to ensure that there is sufficient redundancy in the structure design.
- CIRIA Report C592 (Perry *et al.*, 2003b) comments that loading should be applied only to the worst credible analysis case and not to an analysis using moderately conservative values. The loadings above are transient and a cohesive soil is acting in its undrained condition during loading so applying load with drained strength parameters is only suitable for the worst credible case.
- BS 6031 (2009) comments that traffic loads are normally applied to the surface of an earthwork and usually contribute only a small proportion of the total earthwork load, thus may generally be ignored in settlement assessments unless there is a particular reason take them into account.

70.5 Vegetation

The effect of vegetation on long-term stability is not extensively considered in the design of new earthworks slopes. Perry *et al.* (2003a) detailed the main benefits of vegetated slopes as:

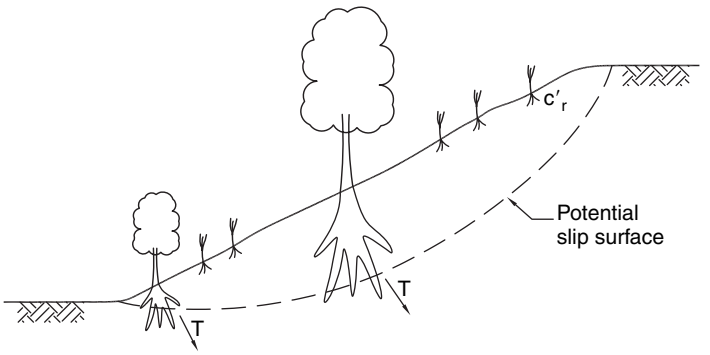
- control of erosion by shielding against the impact of rain, wind and water flow;
- enhancing shear strength through reinforcement by plant roots;
- removal of water by transpiration.

These are indicated in **Figure 70.8**.

Perry *et al.* indicate that most root reinforcement extends generally to 50 mm depth below slope surface with tree roots penetrating up to 3.0 m depth.

Norris *et al.* (2008) builds on the work of Greenwood (2004) and Norris and Greenwood (2006) in developing the concept of increase in soil cohesion due to root growth (c'_R). Typical values of c'_r from historical data after Norris *et al.* are summarised in **Table 70.7**.

A similar approach is developed by Coppin and Richards (2007) who indicate that an enhanced c′ value of up to 20 kN/m² can be taken in the upper surface of a slope where root structures penetrate.



C′r - Enhanced cohesion due to fine roots

T - Tensile force of roots acting on slip surface

Figure 70.8 Forces exerted on a slope by vegetation
Data taken from Norris et al. (2008)

	c'_r (kN/m²)	
Vegetation Type	Maximum	General range
Grasses and shrubs	60.0	2.0–6.0
Deciduous trees	63.0	3.0–10.0
Coniferous trees	94.3	3.0–6.0

Table 70.7 Shear strength contribution from vegetation
Data taken from Norris *et al.* (2008)

A practical observation of enhanced c′ due to plant growth is provided by the Longham Wood Cutting Trial in the Gault Clay on the A20 reported by Greenwood *et al.* (2001).

Norris *et al.* (2008) do not attribute any specific effect of vegetation on the groundwater table but note that pore water suctions can develop close to the surface of vegetated slopes. These can be evaluated by piezometers such as those developed by Ridley (2002).

The benefits of this suction are difficult to quantify on a definitive basis as they will vary with vegetation pattern and geology. They are likely to be beneficial to the stability of cutting slopes and the London Underground Standard for Earth Structures Assessment (2000) allows a reduction in design r_u of 0.05 where cutting slopes are vegetated. Vaughan *et al.* (2004) from work on London Underground embankment slopes report, however, that pore water suctions generate a seasonal pore water gradient through the embankment construction which can lead to seasonal shrink–swell movement in the form of creep. This movement can eventually lead to unacceptable movement of line-side services and London Underground have instituted a managed vegetation policy over the last few years to handle this problem. The difference between rail and highway embankments in the regime of potential recharge,

described in section 70.3 above should be noted, this problem being more likely on rail embankments.

70.6 Embankment construction

Embankment construction to the early twentieth century was carried out with no formal compaction with slopes generally formed at the angle of repose of the deposited material.

The mechanisation of earthmoving equipment, particularly in the United States in the 1920s and 1930s, created the need for control of earthworks compaction by a rapid means of determination. The work of Proctor (1933) developed the concept of 'optimum moisture content' and 'maximum dry density' for compacted material with laboratory testing carried out prior to placement being controlled on site by the use of moisture content determination. The use of optimum moisture content for control of compacted embankment fill is described more fully in Chapter 75 *Earthworks material specification, compaction and control*.

The Casagrande system of soil classification published in 1942 drew on the concept of Plasticity Index for cohesive soil. These classifications were developed in the UK for control of compacted earthworks material by the Road Research Laboratory leading to the development of the Department of Transport's *Specification for Highway Works* (Highways Agency, 2009). As the classifications were predominantly developed for quality control they do not translate directly into design values for earthworks materials but design values can be derived when the ranges of acceptability set are considered.

The control of moisture content for granular materials based on Optimum Moisture Content or cohesive materials based on plastic limit achieves a compacted density of greater than 90% of optimum and a measured density similar to that of the naturally occurring material from which they were derived. Hence, design values derived material from samples of natural material (as described in section 70.2 above) will be similar when compacted into earthworks embankments.

For most granular materials an acceptable moisture content of −3% to +2% of optimum achieves a compacted density of greater than 90% of optimum but this should be tested for all proposed materials. Care must be taken when specifying the upper limit for fine-grained granular materials particularly with a high silt content. These materials develop high pore pressures on compaction at moisture contents above optimum with a resulting loss of strength and limiting acceptable moisture content to optimum is sometimes necessary.

For cohesive materials the limiting of moisture content to limits around the plastic limit was not only to produce an acceptable compacted density but also to control the range of undrained shear strength of the placed material. Whyte (1982) showed that the shear strength of remoulded compacted clay is at 1.5 kN/m^2 at its liquid limit and between 100 and 130 kN/m^2 at its plastic limit (PL). The lower bound of moisture content for acceptability was set at PL -4% by the UK *Specification for Highway Works* and this reflects a maximum undrained shear strength of approximately 200 kN/m^2 at which normal compaction plant could achieve greater than 90% of maximum dry density.

Clayton (1979) notes that the purpose of the upper limit of moisture content was set to ensure embankment stability in the short term, keep self-settlement within acceptable limits and avoid problems due to loss of traffickability of earthmoving plant on the freshly placed fill. The latter is particularly important to the economics of earthmoving and Arrowsmith (1978) quoted minimum undrained shear strength of 35 kN/m^2 for caterpillar tractors and scrapers and 50 kN/m^2 for large rubber-tyred scrapers.

Dennehy (1978) relates remoulded undrained shear strength to tyre pressures of earthworks plant where he indicates a limiting rut depth of 275 mm for efficient plant working. This equates to a minimum shear strength in the range 40–60 kN m^2 for low tyre pressure plant and 60–80 kN/m^2 for high tyre pressure plant.

The Transport and Road Research Laboratory (Anon., 1975) record that for scrapers with a 16–18 m^3 struck capacity rut depth increased from 30 mm to 110 mm from a moisture content at plastic limit to one at 1.1× plastic limit. This reduced average speed from 15 km/h to 5 km/h and resulted in a 75% increase in earthworks costs. The depth of rutting of recently compacted material by plant tipping the layers above increases with moisture content and at undrained shear strength of less than 50 kN/m^2 it is likely that rutting could penetrate the previously compacted layer where compacted layer thickness is normally of the order of 250–300 mm. Further information is also provided by Farrar and Darley (1975).

The range of multiplier on plastic limit for upper bound acceptable moisture content varied with material, and usually Plasticity Index, but accepted limits were in the range PL × 1.1 and PL × 1.3. Arrowsmith (1978) presents a range between PL × 1.1 and PL × 1.2 for Glacial Clays in the north-west of England from which a value of PL × 1.2 was chosen to produce a minimum compacted remoulded shear strength of 70 kN/m^2.

The use of moisture content, with optimum moisture content or plasticity limit, as a compliance tool does, however, present a problem with respect to the reporting of results particularly for cohesive materials. To comply with BS 1377, samples have to be dried for 24 hours before a moisture content can be determined. In the case of cohesive samples the dried sample is used to determine plastic limit and a further 24 hours is required to determine the corresponding moisture content at plastic limit. This means that in excess of 48 hours could pass between sampling and determination of compliance or non-compliance over which time further earthworks layers will have been placed and remedial measures are difficult to effect if non-compliance is determined.

In order to overcome this time lag between sampling and confirmation of compliance Parsons (1976; Parsons and Boden, 1979) developed the moisture condition test. This uses

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site-based laboratory compaction equipment to determine an immediate value which can be compared with a predetermined acceptability range from pre-earthworks testing. The pre-earthworks testing is accompanied by the determination of undrained shear strength, a correlation curve of the moisture condition value (MCV) against undrained shear strength can be plotted and MCV limits set based on the minimum and maximum compacted shear strength measured. This method was developed further by Parsons and Darley (1982) in relation to the operation of earthmoving plant.

It is usual to set the minimum MCV value at 8 which generally corresponds to a minimum undrained shear strength of 50 kN/m^2 for most cohesive materials. Correlation testing should, however, be undertaken on all materials prior to incorporation in embankment earthworks as the relationship between MCV and remoulded undrained shear strength is unique. On the M65 Blackburn bypass Glacial Till material achieved a minimum 50 kN/m^2 at MCV as low as 6.5.

The upper MCV limit is usually set at 16 but for most materials the equivalent remoulded shear strength is in excess of 300 kN/m^2 at this value. As stated above, minimum moisture content in terms of plastic limit was set to achieve a maximum remoulded shear strength of some 200 kN/m^2. Earthmoving plant has developed since this criterion was set and a remoulded shear strength of 250 kN/m^2 will still allow greater than 90% of maximum dry density to be achieved on compaction. This is likely to equate to an MCV value in the range 13–14 for most cohesive earthworks materials.

70.7 Embankment settlement and foundation treatment

70.7.1 Failure of embankment foundation

In considering the performance of new build embankments other factors apart from the stability of the embankment slopes should also be taken into account. These are:

- failure of the embankment foundation;
- settlement of the foundation material;
- self-settlement of the embankment fill.

The potential failure of embankment foundation by failure surfaces passing below the level of the embankment fill should be determined as part of the overall assessment of the stability of the embankment slopes. This failure mechanism is unlikely to occur where the embankment is underlain by granular material or overconsolidated clay. It is, however, a potential failure mechanism where the embankment is underlain by lightly and normally consolidated silts and clays or organic rich materials such as peat. These materials have lower strength properties in short-term undrained conditions rather than long-term drained conditions and failures have, historically, occurred during construction as mentioned in Chapter 69 *Earthworks design principles*. Analysis of the critical failure case should, therefore, consider undrained strength of these materials in slope stability analysis.

It is common where these materials are shallow, up to 2.0 m thickness, that they are removed and replaced with acceptable embankment fill, usually granular material, prior to the main embankment construction. Excavation to a greater depth can be considered if economically viable and there are no stability problems with short-term stability of the excavated slopes. This was the case on the N8 Fermoy byass in Ireland where 6.0 metres of soft material was removed prior to the construction of a 9.0-m-high embankment and replaced with Old Red Sandstone rockfill.

Where soft material cannot be wholly removed below embankment construction and the stability of the overall embankment slope is an issue a number of options are available to improve stability. The main options are:

- geogrids or geocells at the base of embankment construction;
- vibro stone columns or mix in place columns below embankment construction;
- vibro concrete columns or driven pre-cast piles.

The use of geogrids or geocells provides additional tensile strength along a potential failure plane in a manner similar to their use in steepened soil slopes. Design of this type of basal reinforcement is discussed in Chapter 73 *Design of soil reinforced slopes and structures*.

The use of vibro stone columns or soil mix columns produces a block of soil of greater strength than the existing ground and they are usually installed on a grid pattern over the footprint of the embankment. They perform, however, in different ways. Vibro stone columns introduce gravel size material as a column, usually up to 600 mm diameter, and displace the surrounding soft soil effecting a degree of compaction. The process produces a composite soil of bearing capacity up to 150 kN/m^2 but is generally limited to a depth of 10 m. Vibro stone columns are not very efficient in cohesive soil of undrained shear strength of less than 25 kN/m^2 as the untreated soil provides insufficient lateral restraint.

Soil mix columns introduce lime and cement into weak soils by initially mixing the column of soft soil then mixing in lime and/or cement by a dry mix process to produce a column of greater strength. The soil mix columns, normally up to 600 mm diameter, are usually installed as a series of rows. The use of soil mix columns is not as cost effective in soils of high organic content as the required cement content increases significantly.

Outside the UK jet grouting is more popular for treating weak soils. Columns of diameter up to 1.5 m can be formed to produce an overlapping grid of stiffened columns by a wet mix process.

A different mechanism of load transfer is employed if vibro concrete columns or driven pre-cast piles are used. These methods transfer the embankment load onto a stiffer stratum through end-bearing on the column or pile.

Vibro concrete columns are installed by the same process as that to install vibro stone columns, with the same depth

restrictions, but the vibrated material is a wet concrete mix. Most of the vibro concrete columns installed in the UK have been constructed to transfer load to either granular strata, weak rock, e.g. Chalk or Mercia Mudstone, or more competent bedrock.

Driven pre-cast piling is used particularly in treating ground on the approach to structures, the process for which is described in section 70.7.2 below. It was extensively used as a technique on the Channel Tunnel Rail Link in areas of soft sediments in the Thames Estuary. The same contract also experimented with screw piles, a product used extensively in Belgium (NCE; Anon., 2002), for the same purpose.

Typical load transfer mechanisms are shown in **Figure 70.9**.

70.7.2 Embankment foundation settlement

In addition to overall embankment failure settlement of the founding material should also be considered as part of new embankment design.

Settlement below embankment construction, like that below structural ground-bearing foundations, can be considered to comprise three phases:

- immediate elastic settlement on loading;
- primary consolidation settlement;
- secondary consolidation settlement.

Settlement in granular materials below embankment construction with a permeability of greater than 1×10^{-5} m/s can be considered and calculated using elastic theory such as:

$$\Delta h = \Delta p \times I \times \frac{H}{E'} \qquad (70.1)$$

where: Δp = applied pressure (kN/m^2); I = influence factor taking account of depth below embankment foundation; H = thickness of layer in metres; E′ = long-term Young's modulus of soil (kN/m^2).

For cohesive soils settlement will occur on applied load as partly immediate elastic and partly primary consolidation. This is usually calculated using:

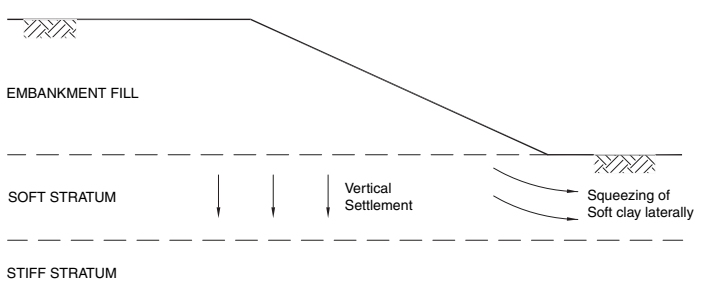
$$\Delta h = \delta p \times h \times m_v \text{ for over-consolidated soils} \qquad (70.2)$$

where: δp = applied pressure (kN/m^2); h = thickness of layer in metres; m_v = coefficient of compressibility (m^2/MN)
and for normally consolidated soils

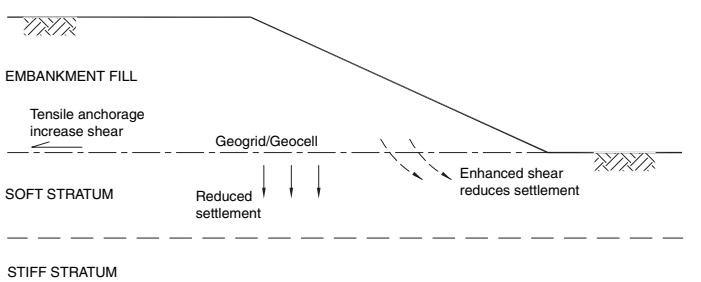
$$\Delta h = \frac{Cc}{1+e_o} \times H \times \log \frac{p_o + \Delta p}{p_c} \qquad (70.3)$$

where: Cc = compression index; e_o = initial void ratio; H = thickness of layer in metres; p_o = initial overburden pressure at centre of layer (kN/m^2); p_c = pre-consolidation pressure (kN/m^2); Δp = applied pressure (kN/m^2).

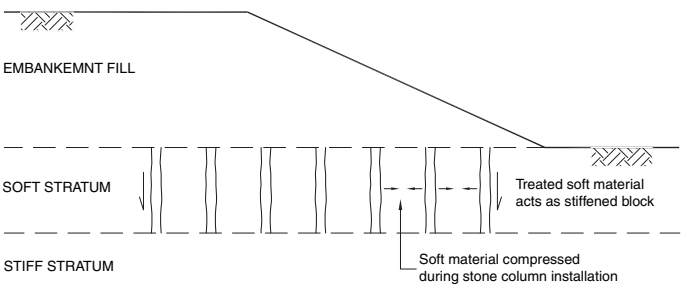
The parameters m_v, Cc, e_o and p_c can be usually obtained from the results of laboratory oedometer testing. m_v should be taken



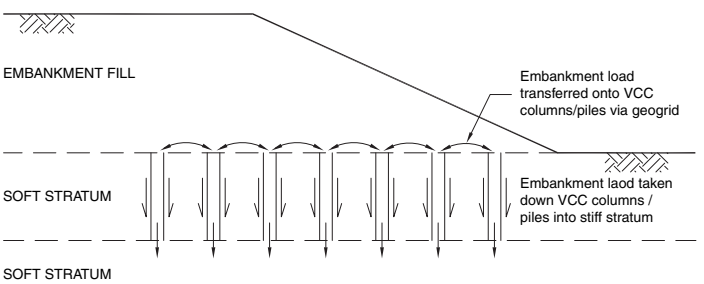
(a) Behaviour of soft stratum without foundation treatment



(b) Foundation treatment with Geogrid/Geocell



(c) Foundation treatment with vibro stone columns



(d) Foundation treatment with vibro concrete column/driven pre-case piles

Figure 70.9 Embankment foundation treatment

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in the range $p_o + \Delta p$ to obtain a realistic value with regard to embankment height. Cc is taken as the gradient of the straight portion of the consolidation curve and p_c taken from construction from the change of gradient of the consolidation curve.

The reduction in applied pressure in the soil beneath the embankment construction can be determined using elastic theory such as Boussinesq or Newmark (1942). The most appropriate is possibly that developed by Oesterberg and presented by Leroueil (1990).

It should be noted that the equations above calculate both immediate elastic and primary consolidation settlement. The proportion of each component will depend on the stress history of the cohesive soil. For an overconsolidated clay the two components are usually equal but for normally consolidated materials the proportion of primary consolidation is greater. Further guidance is provided by Padfield and Sharrock (1983).

The rate of primary consolidation settlement is governed by the Coefficient of Consolidation, C_v, which can be obtained from oedometer testing and is expressed as:

$$C_v = \frac{K}{m_v} \times \gamma_w \qquad (70.4)$$

where: k = coefficient of permeability (m/s); γ_w = unit weight of water.
C_v values from oedometer testing should be taken over a representative pressure range as for m_v.

The time over which primary consolidation occurs subsequent to embankment construction is expressed as:

$$t = T_v \times \frac{d^2}{C_v} \qquad (70.5)$$

where: Tv = time factor; d = length of drainage path (m).

The length of the drainage path is critical to the time required for primary consolidation to occur. If permeable materials such as granular strata both overlay and underlie the cohesive layer two-way vertical drainage can be considered and the length of the drainage path is halved with a subsequent reduction in the time over which primary consolidation is achieved.

In layered soils horizontal drainage should also be considered as although the drainage path to the sides of the embankment may be longer C_h may be considerably greater than C_v than the difference in the vertical and horizontal drainage paths.

In some cases, large consolidation settlements may be theoretically calculated but the length of the drainage path is such that these will be realised over a long period of time. This was the case for a 12-metre-high embankment construction for Heathrow Airport Terminal 5 spur which was underlain by up to 30 m of London Clay. Although 400 mm of total settlement was calculated the time to realise primary consolidation was over 200 years.

In soft compressible soils, where the pre-consolidation pressure is usually exceeded by the applied pressure from embankment loading, settlements will be large and normally unacceptable if the time for primary consolidation exceeds the construction period. A number of solutions are available to mitigate this situation, namely:

- use of lightweight fill;
- pre-loading or surcharging;
- drainage;
- structural elements.

The use of lightweight fill such as polystyrene, PFA or lightweight aggregate will reduce the initial applied load from embankment construction and, hence, the amount of settlement but will not influence the rate of consolidation.

Pre-loading, with embankment construction at the start of and prior to main construction works, will lengthen the period over which primary consolidation can occur. As the rate of consolidation settlement with time is hyperbolic this additional time may allow sufficient primary consolidation to occur such that any remaining subsequent to completion of construction will have minimal impact on the finished works.

The use of surcharge, where greater embankment height and, hence, load is applied, results in primary consolidation occurring in response to the greater applied load and a smaller percentage of total consolidation settlement is required over time to achieve an acceptable percentage under the original embankment loading.

Drainage usually comprises the construction of vertical drains either as vertical plastic strips or vertical sand drains. These are installed at close centres (1.0–2.5 m) in order to significantly reduce the drainage path and produce two-way drainage. A similar effect is produced by the use of vibro stone columns described in section 70.7.1 above.

Structural elements generally comprise driven pre-cast piles or vibro concrete columns and act in the same way as that described in section 70.7.1 by carrying the applied embankment load through the soft compressible stratum to a competent gearing stratum below.

The design of embankments on soft compressible soil next to structures should also take account of the influence of embankment loading on the structure abutments which are likely to be on piled foundations. Leroueil (1990) reports for an applied load greater than pre-consolidation pressure that horizontal movement is 20% of vertical deflection under load. This lateral squeezing of the soil will generate additional shear and bending moments which will have to be taken into account in the pile reinforcement in addition to the requirement from structural loading. An approach to calculate additional load on structural abutment foundations is presented by Springman and Bolton (1990).

One approach to reduce any additional lateral load is by reducing applied vertical load through use of lightweight aggregate. Another alternative is to support the earthworks

approach to the abutment on a structural platform using piles, vibro stone columns, vibro concrete columns or soil mix columns. This has the advantage of not only limiting settlement but also reducing lateral stress transferred from the soil below embankment construction onto abutment foundations. Driven pre-cast piles have been used extensively in this application and guidance on their use is given by Reid and Buchanan (1984). Structural support to the embankment construction behind a structure abutment also minimises the differential settlement between the earthwork and the structure which without treatment creates a hard spot requiring ongoing maintenance.

Embankment construction on soft compressible soils will also result in an increase in pore pressure if the drainage path is not sufficiently short that it can dissipate in a reasonable period subsequent to application of load.

In these conditions, where the drainage path is not reduced by the installation of vertical drainage described above, the increase in pore pressure can be such that failure of the soft material below the embankment can occur as the pore pressure is greater than the strength of the material.

In order to prevent failure it is usual to limit the construction rate in order that pore pressure can dissipate during embankment lifts. The build-up of pore pressure and its dissipation would usually be observed with a piezometer installed in the soft material below the embankment construction. Additionally, staged construction may be applied where hold periods are introduced subsequent to lifts of embankment construction. This allows pore pressure to dissipate prior to construction of the next lift. Examples of staged embankment construction are presented in Leroueil (1990).

The advent of PPP (public–private partnership) contracts with longer maintenance periods has demanded that the designer consider more closely the effects of secondary consolidation with time subsequent to embankment construction. Historically, secondary consolidation has been considered mainly in peat and organic soils where further breakdown and deterioration of the organic structure occurs subsequent to realisation of primary consolidation.

Secondary consolidation, which is load independent, can also occur in cohesive materials particularly normally and lightly overconsolidated clays where applied load is greater than the pre-consolidation pressure in the form of creep. For overconsolidated clays the pre-consolidation pressure is not usually exceeded and loading is applied on the shallow gradient of the consolidation curve with the result of very small settlements.

Secondary consolidation can be determined as:

$$p'_C = C_\alpha \times H \times \log\left(\frac{t_1}{t_0}\right) \qquad (70.6)$$

where: C_α = coefficient of secondary consolidation; H = thickness of layer (m); t_o = time for 95% primary consolidation; t_1 = time after start of embankment construction.

C_α can be derived from natural moisture content after the approach of Mesri (1973). It can also be derived as a ratio with Cc, compression index. Mesri and Feng (1991) and Mesri and Godlewski (1977) report Cc/C_α relationships in the range 0.024–0.055 and 0.025–0.085 respectively, the higher values being reported in peats and organic soils.

Andersen *et al.* (2004) report C_α values in the range 0.019–0.026 for alluvial silt and 0.055–0.108 for alluvial peat from back analysis.

The time to reach primary consolidation is important in the calculation of secondary settlement which may increase in the medium term if measures are taken during construction to accelerate primary consolidation. The use of vertical drainage will reduce the time to realisation of 95% primary consolidation settlement to, say, six months rather than five years without drainage. Although solving construction problems the onset of secondary consolidation occurs at an earlier stage in the maintenance period after construction rather than at a later stage and is of greater magnitude.

A method for reducing the amount, and onset, of secondary consolidation is the use of surcharge kept in place subsequent to achievement of the required amount of primary consolidation settlement within the construction period. This method is described by Mesri and Feng (1991) and Lambrechts *et al.* (2004).

70.7.3 Embankment self-settlement

The self-settlement of embankment fill, like that of the founding material, is dependent on fill type. Trenter (2001) reports the total self-weight settlement of fill can be calculated from:

$$p_i = 0.5 \times \frac{\gamma H}{D^*} \qquad (70.7)$$

where: γ = bulk density of fill material (kN/m^3); H = total height of embankment fill (m); D* = equivalent constrained modulus.

Charles (1993) gives values of D* for a 10 metre high embankment of 50 for well-compacted sandy gravel and 6 for low plasticity clay.

Charles (2008) reports settlements of less than 0.5% embankment height for well-graded granular embankments and less than 1% for clay embankments.

70.8 Instrumentation

Instrumentation is commonly used to confirm earthworks design assumptions and to monitor the performance of earthworks during construction. **Table 70.8** shows commonly used types of instrumentation.

Instrumentation can be installed during a ground investigation but its location should be considered as it may be destroyed by earthmoving plant during construction.

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Instrumentation	Earthwork	Purpose
Piezometer	Cutting	Confirmation of *in situ* pore water pressure
	Embankment	Increase in pore pressure of founding material during construction
Inclinometer	Cutting	Movement of cutting slope
	Embankment	Movement of founding material beyond embankment toe
Settlement loop	Embankment	Settlement of founding material
Rod and plate	Embankment	Settlement of founding material
Survey monitoring	Both	Movement of earthwork during and after construction

Table 70.8 Commonly used instrumentation

The installation of embankment monitoring equipment is usually carried out as a pre-construction activity and, in the case of piezometers and rod and plate settlement gauges the reading tubes are extended as embankment construction proceeds. Survey monitoring has the disadvantage that survey stations cannot usually be installed until earthworks are complete and movement during construction may have been missed.

The use of piezometers should take cognizance of the time over which readings are required and the permeability of the stratum into which they are installed. Whereas a rapid response can be expected in granular materials the low permeability of cohesive materials is such that a period of time is required for the groundwater level in normal standpipe piezometers to stabilise. If readings are required rapidly after installation pneumatic or vibrating wire piezometers are preferable.

The observational method (Nicholson *et al*., 1999) is now commonly used to monitor existing assets, particularly in the railway environment, where instrumentation monitors the medium- and long-term performance of an earthwork, rather than employing immediate remediation, where initial analysis of stability indicates that the current factor of safety against failure is lower than would be normally acceptable.

The observational method is also used in earthworks construction. In this case a suitable contingency measure needs to be determined prior to construction to consider the situation where the installed instrumentation is showing the onset of potential failure. The contingency measure needs to be commensurate with the nature of the earthwork and the time for its instigation. An example is the temporary stockpiling of soil to act as a rapid toe-weight in the case of earthworks cutting failure.

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It is recommended this chapter is read in conjunction with

- Chapter 69 *Earthworks design principles*
- Chapter 94 *Principles of geotechnical monitoring*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 71

Earthworks asset management and remedial design

Brian T. McGinnity London Underground, London, UK
Nader Saffari London Underground, London, UK

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Earthworks are the most common product of civil engineering activities and perform a crucial function in the efficient operation of civil infrastructure.

This chapter provides guidance on the asset management, stability, condition appraisal and stabilisation of existing earthworks.

71.1 Introduction

Asset management is a specialist field in its own right, and this section does not attempt to cover it in detail other than to comment on some of the principal considerations of asset management where they relate to earthworks and to guide the reader to other references where a wider or more detailed appreciation of the subject is required (Hooper *et al*., 2009).

The key texts on earthworks asset management to which reference should be made are Perry *et al*. (2003a) and Perry *et al*. (2003b). The recommendations of these CIRIA documents are relevant to all earthworks asset managers in the UK.

71.1.1 History

Embankments and cuttings form civil engineering structures known as earthworks or earth structures. They are the most common product of civil engineering activities. Little can be constructed without some excavation and transfer of soil or rock. The total length of embankments in the United Kingdom is considerably longer than that of bridges.

It was not until relatively recently that earthworks were designed. Earthworks for roads, canals and railways in the 18th and 19th centuries were built at slope angles that were based on experience and essentially by trial and error. If failure occurred, which it often did, the slope was slackened or other remedial works implemented (Skempton, 1996). These earthworks are generally operating at lower safety factors and steeper slopes than would be considered prudent today.

Before the 1930s nearly all embankments were constructed of relatively uncompacted material, as the process of compaction was poorly understood, and due to the lack of modern construction plant. Large settlement commonly occurred soon after construction. The legacy of these construction methods is reflected in the performance of these historic embankments and, hence, in the degree of current maintenance.

Since the 1930s the development of construction plant and a greater understanding of the discipline of geotechnical engineering have resulted in modern earthworks being better compacted and built at generally less steep slopes than historic earthworks. As a result modern earthworks in the UK suffer less from settlement due to their better compaction. However, slope failures still occur (Perry, 1989).

The deep-seated delayed failure of UK cutting slopes in stiff clays has been the subject of extensive study with a significant body of published research, e.g. Skempton (1964) and Potts *et al*. (1997).

71.1.2 Asset management systems

All earthworks should be managed by an appropriate asset management system to ensure that acceptable performance is achieved and that the earthworks do not present a risk to users.

Effective asset management allocates sufficient maintenance resources, within budgetary constraints, for efficient performance. Until fairly recently in the UK, a reactive approach to earthwork management has frequently prevailed but this is disruptive, inefficient and uneconomic, and inconsistent with long-term asset management objectives. Engineering performance has consequently suffered as a result. More recently there has been increasing trend to adopt a more proactive asset management approach. This requires the implementation of a reliable system of condition appraisal, maintenance and repair or renewal so that existing earthworks can be kept in good condition, avoiding service loss and minimising expensive unplanned works.

The nature of the asset management system will vary to reflect the use of the earthworks and the risks they pose. An asset management system should as a minimum include:

- an asset catalogue: an inventory and summary of condition;
- performance requirements, service level and required duty;
- an asset strategy and plan including consideration of whole-life costs and deterioration models;
- a risk register.

Typical performance requirements for earthworks include:

- safety and reliability;
- satisfying level of service and cost of service requirements;

- operational efficiency;
- satisfying statutory and regulatory obligations;
- value for money and business improvement;
- minimising environmental impact;
- maximising environmental value.

Figure 71.1 illustrates a typical earthworks asset management cycle, which follows a continuous process of condition appraisal, maintenance and improvement or repair, leading to a constant awareness of and improvement in asset condition and performance. An integral part of this process is the management of asset data and the provision of information links between condition appraisal and intervention planning.

An earthworks asset management system (EAMS) comprises a framework that provides asset life-cycle management from conception and design, construction, the work management required during the asset's operation and the planning for renewal, repair and replacement. An EAMS can support all condition appraisal and maintenance work management processes from the creation of work orders, through issuing of work, to the recording of work done. It can be a powerful support tool assisting asset managers to ensure the following:

- An earthwork's condition is improved to reduce the risk to safety and service loss.
- An earthwork achieves a life expectancy that meets the performance requirements.
- Asset management decisions are made on an economic whole-life basis.
- The knowledge and understanding of an earthwork is constantly improved.

The level of complexity and sophistication required of such a system will depend on the size and character of the earthwork asset base. Historically, earthworks management has relied upon written records, including card-indexing systems and files containing paper-based information. Although such systems may still be adequate for the management of very small numbers of assets, the demands of managing large numbers of earthworks and dealing with an ever increasing quantity of information mean that computer-based management systems, relying on electronic information storage and retrieval, offer definite advantages.

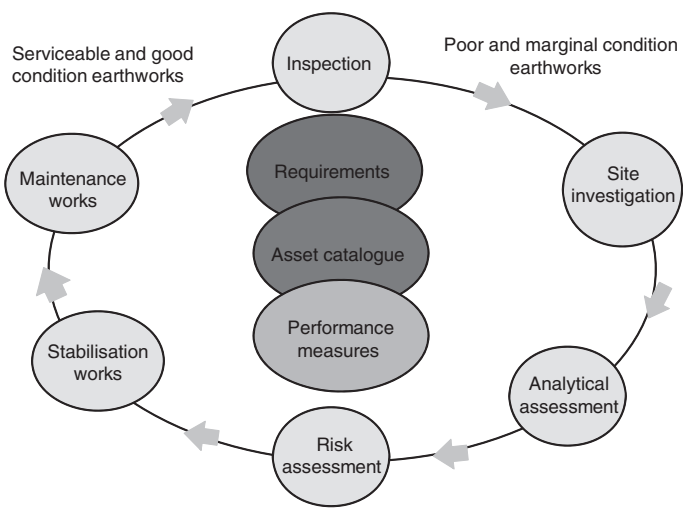


Figure 71.1 Earthwork asset management cycle

Like any other structure, earthworks will have a finite life before significant renewal or replacement works are required. This lifespan can be extended by systematic, continued maintenance and repair.

It is very common for earthwork assets to be in the operate-and-maintain phase of their life cycle, with no likelihood of closure or decommissioning. In the United Kingdom, much of the earthwork asset base has already been in service for longer than the design life of equivalent modern assets.

71.1.3 Whole-life asset management

Whole-life asset management balances maintenance, repair, refurbishment, renewal, replacement and upgrade activities to optimise the long-term value of an asset. The concept of whole-life costing is described by Perry *et al*. in CIRIA Report C592 (2003a). It can be a useful tool when comparing the capital cost of renewal schemes against longer term maintenance costs. Although whole-life asset management is a potentially useful tool it does have its limitations, particularly for existing earthworks that are expected to have very long service lives. These limitations should be understood and the process used with care to ensure sensible results. In practice it is difficult to set up a reliable model for the management of existing earthworks because the performance requirements, the availability of expenditure and an appropriate discount rate over the likely service life are very difficult to estimate. There is a risk that whole-life asset management models can become immensely complicated; however, if they are too simplistic this defeats the whole object of the exercise and their results may be misleading.

71.1.4 Risk assessment

The need for earthwork risk assessment arises principally in the United Kingdom to satisfy statutory safety obligations. The Management of Health and Safety at Work Regulations (1999a,b) impose a statutory duty on employers to conduct regular risk assessments.

The asset management of existing earthworks should follow the approach of reducing risk levels to 'as low as reasonably practicable' (ALARP), giving consideration to cost-benefit analysis of whether spending on remedial works will result in a cost benefit over a realistic maintenance period.

The ALARP principle allows that safety improvements should not be pursued at any cost, but only if the cost of averting a risk is not grossly disproportionate to the risk averted.

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The risk assessment process, therefore, permits a logical analysis of the safety risks; how these can be mitigated or controlled; and the associated funding needs and financial forecasts. Minimal extension of the scope of the risk analysis can have secondary benefits, such as identifying the exposure to non-safety-related business risks and helping to formulate business cases and overall spending forecasts and plans. Risk assessment is, therefore, a tool that earthworks asset managers apply to ensure that safety objectives are met within a business framework, and that funds are justified and allocated in response to safety and business needs.

The management of risk should be a key aspect of earthwork repair or renewal projects as in all construction activity (Godfrey, 1996) and should include:

- programme, quality and financial risks to ensure the successful delivery of the project;
- health and safety and environmental risks to satisfy statutory requirements.

All UK earthwork repair or renewal projects will come under the requirements of the Construction (Design and Management) Regulations (2007), known as CDM. All those involved in design and construction activities associated with earthwork repair and renewal should consider the requirements of the project and seek to follow the 'spirit of CDM' to ensure the approach taken is appropriate for the project.

A project risk register (PRR) should be established and include all risks identified by those involved with the design and construction of earthworks, and those involved in repair and renewal.

71.1.5 Sustainability

Geotechnical engineers and earthworks asset managers need to consider not just the technical aspects of the work, but how it can be undertaken in a way that will enhance the environment and maximise sustainability, e.g. by maximising the use of existing fill materials in earthwork remedial works' design and construction, and with a sensitivity to the surrounding environment. The fact that many of the organisations responsible for earthworks have environmental policies and have set specific targets relating to environmental issues is testimony to their importance.

However, it should be appreciated that this is a rapidly developing field and further changes are likely. All those involved in earthworks asset management will need to be well informed and vigilant in understanding the consequences of changing legislation on environmental issues (also see Chapter 11 *Sustainable geotechnics*).

71.1.6 Earthwork asset interfaces

Earthworks can carry a number of different infrastructure elements such as structures, services, drainage or signage. It is important to coordinate all maintenance activities so that best use is made of time and that the works to all elements are integrated to prevent damage by piecemeal working. Asset managers should be kept informed of and approve all activities to be undertaken involving earthworks, as it is possible for overall performance to be adversely affected by the activities of others, e.g. an excavation for a service at the toe of the slope could trigger slope movement.

Conversely the designer should assess the potential impacts of earthwork repair or renewal on existing infrastructure. In all these cases earthwork activities can change the loading on the existing infrastructure or modify the surface water and groundwater flow paths, both of which could be detrimental to the stability of the adjacent assets and, therefore, require consideration within the design. Where the existing slope is identified as being of poor stability then the design of the temporary works will require special consideration to ensure that stability is maintained throughout construction.

71.2 Stability and performance

71.2.1 Mechanisms

71.2.1.1 Shallow instability

Shallow instability is usually confined to the sloping part of the cutting or embankment and the slip surface does not pass through the transport corridor (railway or highway) or the associated services usually located at the toe of the cuttings or on the crest of embankments. The failure surface is generally parallel to the slope surface and has typically a depth of between 1 and 1.5 m.

In granular materials this type of failure manifests itself as ravelling of material down the slope and collecting at the toe. In cuttings this will result in debris in the margins of the transport corridor at the toe and will require to be cleared through routine maintenance.

In cohesive soils this mechanism is generally controlled by the surface saturation within the zone of seasonal variation caused by exceptionally heavy rainfall, in particular following a prolonged dry season.

Vegetation on the slope surface generally has a positive effect on this type of instability through the reinforcement provided by the roots (Coppin and Richards, 2007) and the control of the pore pressures within the slope surface. In cohesive soils seasonal shrink and swell will cause surface cracking of the slope during the dry period leading to increased permeability.

71.2.1.2 Deep-seated instability

For deep-seated instability the slip surface is more extensive, covering the whole length of the slope and may pass through the soil layer beneath the embankment or cutting. The slip surface is generally deeper than 1–2 m and affects the transport corridor or the associated services located on the crest of embankments or the toe of cuttings. This type of failure, when it happens, is more disruptive and can have serious consequences.

For free-draining granular soils deep-seated instability is not a major concern and is generally easier to predict. The

instability in this type of material manifests itself as surface ravelling of materials downslope leading to maintenance problems.

Deep-seated instability in cohesive soils can be categorised as first-time slides and slides along old failure surfaces.

First-time slides

First-time slides happen to slopes that have not experienced previous slides and, hence, do not involve pre-existing shear surfaces. First-time slides have been studied extensively in particular in relation to cuttings in stiff clays (DeLory, 1957; Skempton, 1948, 1964, 1970, 1977; Chandler and Skempton, 1974; Chandler, 1984).

The results of these studies indicated that failure generally occurs many years or decades after construction due to a very slow rate of pore pressure equilibration (delayed failure). The results also indicated that the average strength at failure was considerably smaller than the peak strength measured in the laboratory but greater than the residual strength. The results of back analysis of long-term failures in five cuttings in brown weathered London Clay by Skempton (1977) indicated the following operating effective strength parameters at failure:

$$\phi' = 20°, \qquad c' = 1 \text{ kN/m}^2$$

with the average pore pressure ratio r_u typically ranging between 0.25 and 0.35.

The peak and residual strengths measured in the laboratory were:

$\phi'_p = 20°$, $c'_p = 7$ kN/m^2 (Sandroni, 1977)
$\phi'_r = 13°$, $c'_r = 1$ kN/m^2 (Skempton and Petley, 1967).

It is now well established that the most likely reason for the difference in the peak strength and the operating strength at failure is due to the mechanism of progressive failure (Potts, *et al*., 1997; Ellis and O'Brien, 2007).

The mechanisms of delayed failure and progressive failure referred to above are defined as follows.

Delayed failure

When a cutting is excavated in stiff overconsolidated clays of high plasticity due to unloading effects, the pore pressures in the soil are reduced sometimes even to negative values. With time these pore pressures will start to increase towards the long-term equilibrium values and the soil will start to swell. This process will lead to a reduction in the mean effective stress and the onset of delayed failure. The length of time to this failure will depend on the permeability of the soil and may take several decades (Vaughan and Walbancke, 1973; Potts *et al*., 1997).

Progressive failure

Progressive failure occurs as a result of the non-uniform development of strains along a failure surface leading to non-uniform mobilisation of shear strength. This phenomenon is specific to clays that exhibit highly brittle behaviour, defined as the magnitude of the drop from peak to residual strength (Vaughan, 1994; Potts *et al*., 1997; Ellis and O'Brien, 2007).

In a cutting formed in such a clay material, a non-uniform loading due to swelling of the soil near the toe will result in the mobilisation of the peak strength in an element of the soil whilst the other parts are still in the pre-peak state of stress. This is accompanied by the development of a slip surface at this location. With further increases in pore pressure the strength at this location is reduced from the peak towards the residual strength. Further increases in pore pressures will lead to the development of the rupture surface further into the slope. This will in turn lead to larger displacements and the final failure of the slope. At this time the strength along the rupture surface will be close to residual at some points, post peak at some other points and close to peak along the rest. Hence, the average strength of the soil along the failure surface will be somewhere between the peak and residual strength of the soil.

Slides along old failure surfaces

This type of failure is generally caused by slip along pre-existing shear surfaces or partially pre-sheared surfaces. These surfaces may have developed as the result of past instability, commonly, during construction or by landslides or other geological processes. Repair work does not always remove the shear surfaces fully. Shear surfaces are sometimes present within the softer alluvial deposits underlying embankments and, hence, may have been left in place.

The stability analysis for this type of failure is carried out using residual effective strength parameters or parameters close to residual strength where partially pre-sheared surfaces are suspected.

71.2.1.3 Seasonal shrink and swell

Seasonal shrink–swell movements occur in embankments and cuttings containing high-plasticity clay soils due to seasonal changes in the soil moisture content.

The seasonal movement is strongly influenced by the presence of high-water-demand mature trees, which leads to increased changes in the soil moisture content in the proximity of the trees. In the dry summer months due to lack of rainfall and the presence of mature vegetation, moisture is removed from the soil resulting in shrinkage. This causes shrinkage cracking. In the wet autumn and winter season the soil is re-charged with moisture. This wetting also combines with less demand for moisture from the trees leading to significant increases in the soil moisture content, which in turn causes swelling to occur. The resulting movements will influence the performance of the transport corridor located on the crest of an embankment and possibly at the toe of a cutting (**Figure 71.2**).

Shrinkage cracking of the soil in the dry periods leads to the formation of water pathways and increased permeability of the soil. This in turn leads to a more rapid inflow of water into the soil and a deeper penetration depth. This allows the

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rainwater to reach significant depth in the following wet season, thus, increasing the zone of influence of shrink–swell effects.

In overconsolidated high-plasticity clays where increases in strain beyond the peak will result in a post-peak reduction in the shear strength, cycles of shrink–swell can lead to a loss of strength and onset of progressive failure.

The swelling of soils in the wet period is associated with upward and outward movements whereas the dry season is associated with mainly downward movement of the soil. This leads to a phenomenon called 'ratcheting' where the horizontal swelling that occurs in the wet periods does not fully recover in the dry summer months. Further information on seasonal shrink–swell movements and the effects of vegetation can be obtained from McGinnity *et al*. (1998), Kovacevic *et al*. (2001), Loveridge and Anderson (2007), Butcher and Pearson (2007), Take and Bolton (2004), Hudacsek and Bransby (2008) and Nyambayo *et al*. (2004).





Figure 71.2 Effect of trees on track deformation

71.2.1.4 Shoulder instability

Shoulder instability is a shallow type of instability that is more specific to embankments where, due to various factors described below, the shoulder of an embankment cannot provide the required support to the transport corridor on the crest (**Figure 71.11(a), (b)**). The main factors responsible for this type of instability are as follows:

- steep upper-slope angle, which can cause gradual ravelling of material down the slope leading to narrowing of the shoulder;
- dynamic loading of trains or road vehicles: this loading can travel laterally and mobilise the soil particles, and with time these soil particles will move down the slope under gravity;
- embankment material;
- fine granular materials such as ash are particularly prone to this type of instability; this type of material in a dry state can become very powdery and move under dynamic loading;
- seasonal effects.

71.2.1.5 Flooding and scour

Flooding and scour at the toe of the embankments can occur as the result of rivers or streams flowing parallel or through an embankment.

Increased water levels during wet seasons can cause flooding, resulting in increased water pressure at the toe of the embankment. This can result in reduced factors of safety against deep-seated failure leading potentially to the onset of instability, depending on the duration of flood.

Flooding can also cause scour erosion of the toe of the embankment, which again results in reduced factors of safety against deep-seated failure due to the reduced weight at the toe of the embankment (**Figures 71.3, 71.4**).



Figure 71.3 Scour erosion at the toe of a railway embankment



Figure 71.4 Concrete channels provide a more durable solution



Figure 71.5 A flow failure following a high-intensity rainfall event in Boston Manor (2006)
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The important factors to consider for the safe management of the slope are as follows:

- direction of flow of water in the river;
- extent of Environment Agency flood zone and capacity;
- topography and land use adjacent to the toe;
- presence, condition and capacity of drainage at the toe.

71.2.1.6 Flow failure

Flow failure is likely to occur in high-permeability granular materials as a result of rapid saturation caused by short but high-intensity rainfall or a storm event. The saturation is caused by infiltration of rain water into the slope surface but also, more significantly, from concentrated surface run-off in areas of poor drainage. Hence, this type of failure is much more likely to occur in cuttings where the topography, land use and poor drainage condition at the crest of the cutting allows a large volume of water to be directed towards the slope in a very short period of time. The high permeability of the soil within the cutting allows this water to be absorbed within the slope rapidly, leading to a mass flow of material.

From the above it can be seen that the key parameters for this type of failure to develop are:

- short-duration high-intensity rainfall event;
- high-permeability soil;
- slope geometry;
- topography;
- land use;
- vegetation;
- drainage.

Low-intensity long-duration rainfall events are unlikely to lead to a flow failure as, in a granular material, there will be adequate time for the excess water to drain. Flow failures are unlikely to develop in low-permeability soils where rapid saturation cannot take place. Finally, the right geometric and topographic conditions must exist to generate a concentrated flow of water. Hence, embankments are less likely to be at risk from this type of failure.

The consequences of this type of failure can be very significant. In a cutting a large volume of soil can flow onto the transport corridor at the toe of the cutting causing derailment of trains or accidents on the roads (**Figure 71.5**).

71.2.1.7 Frost shattering

Frost action can lead to gradual degradation of some rock cutting slopes, depending on the permeability, porosity and the joint system within the rock. Ice crystals that form within the pores and between the joints in a slope face induce strains, which result in splitting and shattering of the slope surface.

The volume change caused by the growth of ice crystals can result in an increase in the joint width and depth. Hence, a repeated freeze–thaw action can lead to an increase in the permeability of the rock mass, which in turn increases the depth of influence of the frost action.

One of the rock types particularly susceptible to frost action is chalk. Shattering or loosening of the surface of cutting slopes in chalk results in the generation of debris at the toe of the cutting, which affects the serviceability of the transport corridor located at the toe (Lord *et al*., 2002) (**Figure 71.6**).

Frost shattering can have more significant consequences in the transport corridor at the toe when loosening of the surface leads to destabilising of the vegetation and tree roots on the surface of the slope, which can fall onto the rail or road at the toe.



Figure 71.6 Frost shattering on a chalk railway cutting



Figure 71.7 Embankment failure as a result of increased pore pressures after a prolonged rainfall event (1994)

71.2.2 Limit states

71.2.2.1 Ultimate limit state

The ultimate limit state is reached when the disturbing forces acting on a slope exceed the resisting forces. Under this condition collapse, slope instability or other forms of structural failure will occur, which could significantly affect the safety of the services or the structures associated with the earthworks (Perry *et al.*, 2003a, 2003b).

For an embankment, an ultimate limit state failure will lead to significant disruption of the safe operation of the railway or highway transport running on the crest of the embankment (**Figure 71.7**).

In cuttings, the slope failure, depending on the size and extent of the rupture surface, may affect the operation and the safety of the transport infrastructure and the associated services running at the toe of the cutting. The failure could also affect the structures on the crest of the cutting such as buildings, roads, car parks and their associated services such as drains, cables, etc.

In order to prevent such a mode of failure, a margin of safety against the disturbing forces has to be introduced in the design and maintained during the life of the structure. For existing slopes this can be achieved by performing slope stability analysis. Where the stability of slopes is found to be marginal, remedial measures should be implemented to increase the margin against instability.

71.2.2.2 Serviceability limit state

A serviceability limit state is reached when deformation within an earthwork exceeds the desired or prescribed limits such that the structure can no longer perform its required functions satisfactorily.

Deformations within an embankment can affect the performance of the transport infrastructures running along the top leading to excessive settlement of roads or railway tracks or the associated services such as cable runs, signals, etc.

In cuttings, deformations within the slope can affect the buildings, roads, car parks, drains and other structures supported on the crest.

It is not common practice to carry out analysis of a slope to predict deformation. It is generally assumed that adequate factors of safety against deep-seated slope instability would ensure the satisfactory performance of the slope during its design life.

However, where required, numerical analysis can be carried out to estimate slope deformation. For existing slopes, however, any such analysis would need to model the loading history experienced by the slope since construction. The analysis would also require appropriate parameters such as strength, stiffness, permeability of the soil and the pore pressure regime within the slope in order to simulate the field conditions leading to realistic predictions.

The serviceability of slopes is affected by the following factors, some of which were discussed in the preceding sections:

- seasonal shrink–swell;
- vegetation;
- dynamic loading;
- animal burrowing;
- heavy rainfall;
- flooding;
- frost.

71.3 Earthwork condition appraisal, risk mitigation and control

71.3.1 Inspection

For an earthworks asset management system to be successfully delivered it is necessary to develop an appropriate earthwork inspection system. Its main purpose is to:

- appraise structural safety;
- detect incipient defects at an early stage and monitor the development of defects to determine the urgency for, and the nature of, corrective action;
- classify the condition of the earthworks according to a defined system;
- identify the need for preventative action.

The inspection approach varies between UK infrastructure asset owners: in all cases it involves an observation and recording of the earthwork condition at the time of the inspection.

The following factors should be considered in developing an inspection regime:

- defect definition, classification and condition rating system;
- inspection frequency;
- inspection types (cyclical or *ad hoc*);
- data-collection techniques;
- risk.

The frequency of cyclical inspections should be set considering the history and condition of the earthwork together with the likelihood and consequences of failure.

Where possible a record should be made of the drainage system as an integral part of the earthwork inspection process. The inspection should also establish the presence and overall condition of any additional stabilising structures (retaining walls, soil nails, etc.) within the earthwork. Some of the stabilising structures might require inspection by other disciplines and so harmonised inspection programmes should be considered.

The inspection should not only focus on the specific asset being inspected but should be watchful of nearby activities that may adversely affect the asset. The information collected from inspections should be used to identify defects and monitor the asset condition and degradation rate.

Remote inspection from airborne platforms is becoming an increasingly powerful tool to support traditional site inspection and aid strategic planning. It can offer a safer and more cost-effective method for identifying problem areas than visits conducted on foot. Duffell *et al*. (2005) described the use of airborne remote-sensing techniques to assist in prioritising detailed ground-based inspections of earthworks.

71.3.2 Analytical assessment

The objectives of an earthwork analytical assessment are more rigorous and detailed than an inspection and are to:

- provide data on and a detailed assessment of the condition of specific earthworks and to accurately quantify slope stability;
- provide evidence for a business case for works to improve performance of individual earthworks, based on ultimate and serviceability limit state criteria;
- identify those earthworks that need priority attention when formulating asset management plans;
- assess risks, plan maintenance budgets and programmes;
- anticipate future problems and identify a rate of deterioration;
- provide information for the design of any required stabilisation.

An earthwork assessment typically consists of the stages outlined below.

71.3.2.1 Desk study and walkover survey

A preliminary study includes a desk study, which reviews all relevant data obtained from inspection and maintenance records, and a detailed site inspection (walkover survey) to identify visually any areas of distress. In addition to geological and geotechnical data the desk study would also typically consider drainage, environmental, ecological and services information.

71.3.2.2 Ground investigation

A ground investigation considers instrumentation and monitoring, and includes exploration, sampling and laboratory testing.

71.3.2.3 Interpretation and stability analysis

Information obtained from the ground investigation and desk study, including any deformation data, must be interpreted to establish ground model and geotechnical parameters. Likely failure and deformation mechanisms must be considered and stability analysis performed.

71.3.2.4 Reporting and prioritisation

Results from the ground and desk study investigations and their interpretation, and the stability analysis are presented with an explanation of the condition and criticality of the earthworks. Recommendations and cost estimates for repair or renewal should be provided.

71.3.3 Earthwork risk mitigation and control

The principal responsibility of the asset manager is to ensure that earthworks are maintained in a condition such that the safety of users and the public is not compromised. Factors such as age, increased traffic loading, inadequate or poor maintenance and deferred repairs reduce the performance of earthworks and may ultimately compromise operational safety. Specific earthwork performance requirements are set by infrastructure owners, e.g. railway embankment deformation limits relate to maintaining track quality for a specified line speed.

When an earthwork experiences stability problems, it is often necessary to impose temporary restrictions on speed or loading, or close traffic lanes to maintain operational safety. When its structural capacity is inadequate and there is a risk of collapse, public safety is jeopardised and complete withdrawal of service may be necessary.

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If instability or other loss of functionality is predicted, the following options may be adopted in increasing order of impact:

- increased inspection frequency;
- monitoring;
- routine maintenance, e.g. unblocking drains;
- service restriction, either while mitigation is put in place or stabilisation is undertaken;
- renewal works;
- withdrawal from service.

71.3.4 Monitoring existing earthworks

The purpose of monitoring earthworks includes the following:

- mitigation of risk until repairs are implemented;
- determine the rate of movement and establish if it is constant, accelerating or decelerating;
- establish *in situ* pore water pressures and any variation over time;
- where movements have occurred, gather information on the depth, area and lateral extent;
- to establish the effects of construction adjacent to earthworks, both during construction and in the long term;
- to obtain quantitative performance data;
- to validate the output from analytical assessment, particularly where it is considered to have produced conservative results, and to confirm design assumptions.

The designer of the monitoring system should define the following aspects in advance of implementation of the scheme:

- objectives;
- techniques;
- required accuracy;
- frequency;
- trigger levels;
- data collection and reporting;
- action plans.

There is a vast range of instrumentation available and further guidance can be obtained from Dunnicliff (1998). Recent developments in this field include web-based, cable-free or fibre-optic systems, which offer remote, real-time monitoring that have the potential to be integrated to automate alert alarms. Owing to the rate of technological development in this field, discussions should always be held with specialist instrument manufacturers and installers.

In establishing trigger levels, the likely magnitude of normal baseline values should be established. For example, seasonal vertical movements of up to 50 mm have been recorded on some London Underground embankments due to the effects of vegetation on high-plasticity clay. Therefore, the monitoring system should be put in place sufficiently in advance of control periods so that the range and pattern of background movements can be established.

For new or modified earthworks, monitoring can aid in finalising the design either through the approach recommended in BS EN1997–1:2004 (British Standards Institution, 2004) or through observation itself (Nicholson *et al*., 1999).

71.4 Maintenance and remedial works

71.4.1 Preventative and corrective maintenance regimes

As discussed in section 71.1.2, maintenance forms a part of the asset management regime and is undertaken in order to maintain the assets in their current condition and to minimise asset deterioration and degradation. Preventative maintenance is carried out on a routine basis, e.g. through vegetation management and cleaning drainage systems. The frequency of preventative maintenance is governed by the severity of any potential problems and their consequences. Corrective maintenance is undertaken where defects are observed during inspection or site walkovers. Prioritisation based on risk is used to determine the maintenance programme. Maintenance for earthworks could include the following:

- vegetation management;
- debris clearance;
- drainage cleaning;
- animal burrows;
- erosion control measures;
- construction of a toe-retaining structure to capture debris;
- construction of a mesh and anchor system to contain and capture rock falls.

71.4.2 Repair and strengthening

Repair or strengthening is required in the following circumstances:

- where the earthworks can no longer provide the required function against the performance criteria;
- where the asset does not meet the minimum requirements of the standards set by the regulatory organisations;
- where a full or partial slope failure has occurred.

The aim of the remedial works is to improve the stability of the earthworks against the ultimate and serviceability limit states. However, during the design other safety and environmental features are considered and included in the remedial works in order to upgrade the asset to modern standards.

The performance criteria for remedial and strengthening design generally consist of the following:

- To improve the stability of the earthworks in order to meet the requirements for the minimum factor of safety against deep-seated instability.
- To improve the stability of the earthworks in order to meet the requirements for the minimum factor of safety against shallow instability.
- To limit the subsequent deformation such that the relevant maintenance target levels as defined in relevant stakeholder standards are not exceeded. For instance *LU Standard 1–159 Track Dimensions and Tolerances* (2011) requires the differential settlement of the outer rail to be limited to 1:500 along any 10 m length of track with a limit of 1:300 across the rails.
- To design remedial work such that it will have no detrimental effect on the stability of the existing structures within or adjacent to the earthworks.
- All permanent earthworks should achieve a specified design life that varies between 60 and 120 years depending on which design standards are being followed.

The stability of slopes can be improved by increasing restoring forces or reducing disturbing forces. The following options can be considered for the design of remedial works:

71.4.2.1 Cuttings

Re-grading

Cutting slopes can be stabilised by re-grading to a shallower angle provided sufficient room exists at the crest of the cutting.

In general this requires a significant amount of excavation and off-site disposal of large volumes of material. This option requires the complete removal of all vegetation and replacement after completion of construction. Earthworks of London Clay could also present difficult working conditions during wet periods.

Spaced bored piles

This solution consists of installing bored piles into the body of a slope in order to provide additional shear resistance against deep-seated instability. The piles are designed to penetrate the soil mass below the slip surface in order to provide adequate passive resistance. The piles can be between three and six diameters apart and the system relies on arching between the piles to support the soil mass in the active zone (Davies *et al*., 2003; Smethurst, 2003; Carder and Barker, 2005; Smethurst and Powrie, 2007).

The spacing of the spaced bored piles in the longitudinal direction is dependent on the ground conditions and individual pile capacity, and is selected so as to provide the required factor of safety against deep-seated instability. The selection of the pile diameter is affected by the sizes of the available piling rigs and the associated temporary works required. Safety issues associated with the construction of piles above an operational highway or railway are important when selecting the appropriate construction plant.

The location, pile diameter and spacing will need to be optimised in order to provide the most efficient system, as well as allowing for buildability issues such as the size of the piling rig, temporary works and the clearance from existing services and structures.

Construction of bored piles can be carried out using a mini-piling rig from a temporary bench excavated into the slope of the cutting or using a scaffolding platform. Small piling rigs, such as the TD610 for pile diameters up to 450 mm, can be used. Where the bending moment and shear forces require larger diameter piles, piling rigs such as the Klemm KR 709 or similar can be used. This type of rig has a larger size and weight and a higher mast when fully extended. Hence, the requirements for a suitable piling platform and the associated temporary works will need to be considered (**Figure 71.8**).

The advantages of this system are:

- relatively low cost;
- much of the vegetation on the upper slope can remain;
- minimised earthworks;
- access for plant and equipment;
- low environmental impact;
- ease of construction;
- design life.

Soil nailing

Soil nailing comprises the installation of nails into the body of the slope at shallow angles of between 10°–30° below the horizontal, penetrating the soil mass beyond the critical slip circle in the passive zone. Various types of soil nails are available and the choice is dependent on the particular application and the availability of space and access for installation plant, and the required design life and cost (Phear *et al*., 2005).

Hollow self-drilling soil nails are quick and easy to install in some granular soils. These are installed by simultaneous drilling and grouting, using a sacrificial bit attached to the advancing end of the hollow soil nail. Soil nailing a slope will require the complete removal of vegetation from the slope where nails are to be installed.

For sites where access is difficult, an abseiling technique can be adopted for installation of the nails. However, only a limited length of nails and depth of penetration can be achieved using this system. A soil-nailing scheme constructed under difficult access conditions using an abseiling rig is shown in **Figure 71.9**.

Design life and corrosion protection will significantly affect the financial viability of the scheme. A risk-based approach should be adopted where corrosion protection is related to the possibility of failure (Phear *et al*., 2005).

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(a)



(b)



(c)



Figure 71.8 Construction of spaced bored piles on a London Clay cutting: (a) vegetation removal and site preparation; (b) construction of the mid-slope bench for piling; (c) different size piling rigs (TD610 and Klemm709R) to suit the pile diameters

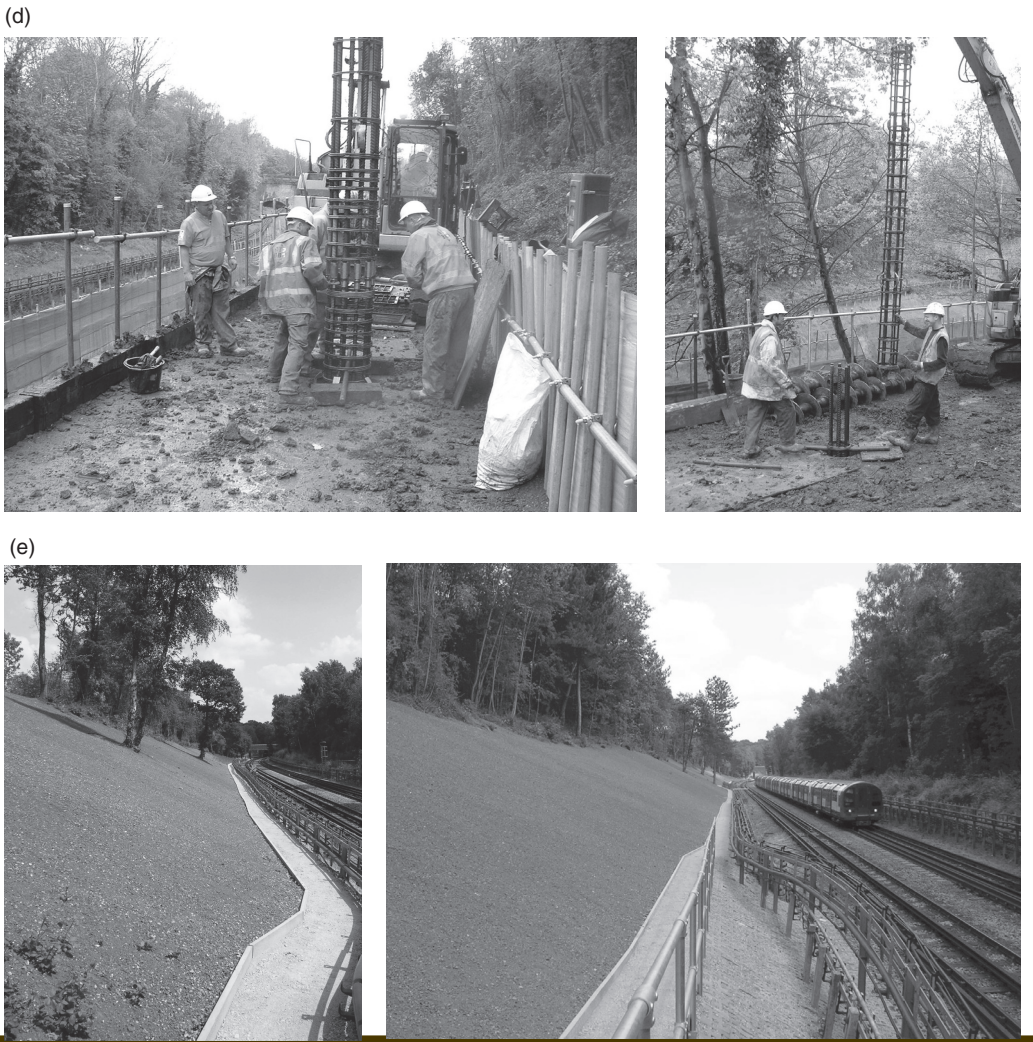


Figure 71.8 (d) lowering of the reinforcement cage; (e) slope after completion of construction

Toe retaining walls

Where space is available, the construction of a wall such as a mass-concrete gravity wall or a gabion wall at the toe of the cutting combined with backfilling with granular fill will provide additional weight at the toe to increase the factor of safety against deep-seated instability. The depth to the base of the wall should be designed such that the slip circles passing below the wall have the required factor of safety. This may require deep excavations at the toe, which may not be practical given the space constraints often existing at the toe.

Where excavation is impractical at the toe, a piled wall with a capping beam may provide a better solution.

Drainage

Where it has been established that the instability of a slope is due to excessive pore pressures within the slope, the installation of a drainage system to reduce the pore pressures may be a viable solution.

The following drainage systems may be considered:

- a cut-off drain at the crest in order to stop the flow of water into the slope;
- a counterfort drain in order to drain the body of the slope;
- a toe drain in order to keep the water level down at the toe;
- a combination of the above.

However, when designing a drainage system the following must be considered:

- need for reliable groundwater data;
- requirement for regular monitoring and maintenance of any slope drainage system in order to ensure continued operation during the intended design life;
- disposal of drainage discharge can require connection works to the existing track drainage;
- this solution may require to be combined with other remedial options in order to meet the factor of safety requirement.

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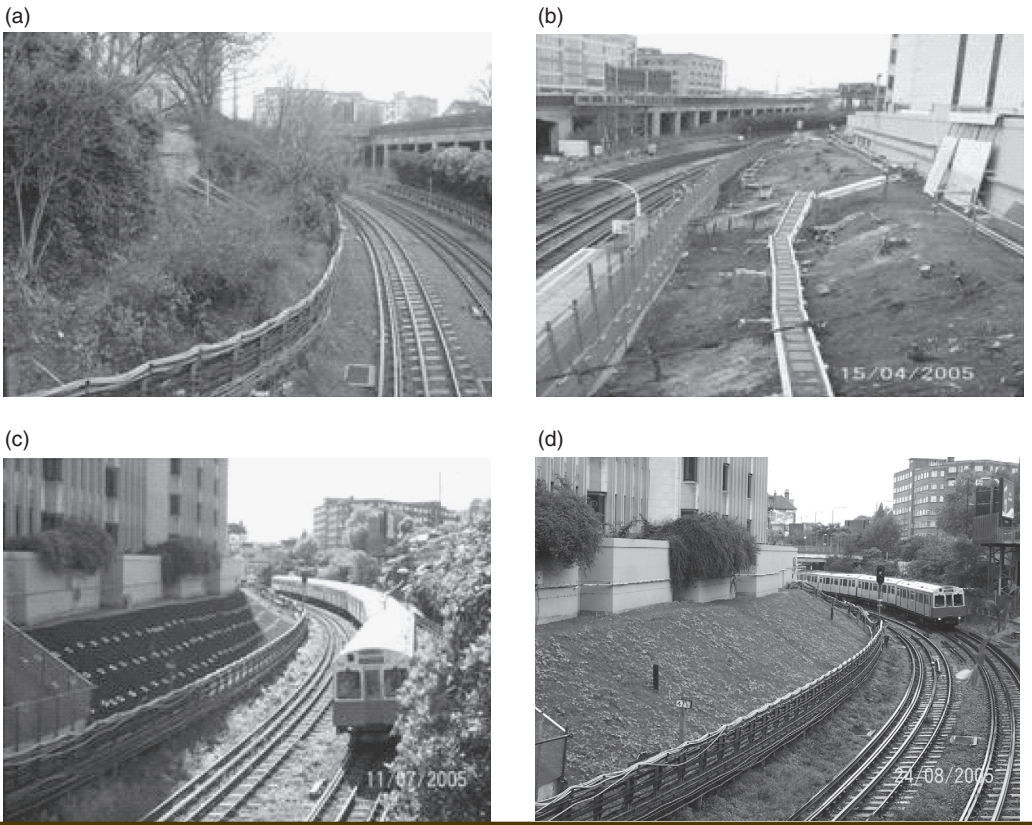


Figure 71.9 Soil nailing at West Kensington cutting, before, during and after construction: (a) slope before remediation; (b) slope during construction, due to space constraints only hand excavation was possible and a conveyor system was used to transport material away from the slope; (c) slope after completion of soil nailing; (d) final slope profile after vegetation growth

71.4.2.2 Embankments

Re-grading

Embankment slopes can be stabilised by placing a berm against the slope and re-grading to a shallower angle, provided sufficient room exists at the toe.

The berm solution can be used to widen the crest and provide more support to the shoulder of the embankment. The re-graded embankment will typically have a uniform gradient. For embankment widening and re-grading, a Class 1A granular material is generally specified. Recycled material such as crushed concrete can be utilised for this purpose provided that chemical composition of the fill is found to be acceptable for the intended purpose.

This material will be benched into the existing slope, removing any existing local over-steepening or slope bulges. The fill will be placed and compacted in accordance with the contract specification. A layer of seeded topsoil is usually placed on top of the new fill to provide vegetation (**Figure 71.10**).



Figure 71.10 Re-grading of an embankment to increase stability, incorporating a walkway at the toe and crest

Ruglei™ verge protection system

In areas where the slopes of embankments have a satisfactory factor of safety against deep-seated and shallow slips but widening of the cess is required to provide a stable shoulder, the Ruglei verge protection system can be considered.

This system consists of an angular galvanised-steel mesh and a geogrid inset to contain and support the fill (coarse granular fill or ballast) that is required for the widening of the cess. Vertical and raked piles are used as necessary to stabilise the system. Driven, old rail sections or steel H-piles can be used as

vertical piles. The type, spacing and length of the vertical and raked piles depend on the slope geometry and ground conditions. The coarse granular fill or ballast provides a free-draining medium, which would also act as a natural barrier against vegetation. Hence, the lateral drainage of the track is improved and the application of weedkillers can largely be eliminated. A rapid rate of construction per shift can be achieved, in particular if rail sections are used as vertical dowels. This system is economical and is considered suitable for areas where the only requirement is to widen the cess and to provide shoulder stability (Saffari and Smith, 2005) (**Figure 71.11**).

Anchored bored-pile wall

For this option a system of embedded vertical small diameter (of the order of 300 mm) bored piles with raking piles and a capping beam can be used to improve the stability of slopes and/or provide support for the widening of the cess at the crest. The capping beam acts as a small retaining structure to support the widened cess. The vertical piles act as a foundation for the capping beam and the raking piles provide lateral stability. Alternatively, small-diameter driven piles could be used instead of bored piles. These piles can be installed from the toe of the embankment, thereby eliminating the need for a piling platform resulting in significant cost and time savings.

Toe wall

Where space at the toe of an embankment is too limited to re-grade the slope to stable angles, a combination of a wall at the toe of slope and re-grading by placing additional fill between the top of the toe wall and the edge of the widened crest can be used. Issues regarding the design and construction of this type of wall are generally similar to those for cuttings. Space at the toe of an embankment may be more readily available to accommodate a retaining wall compared to cuttings.

King-post wall

Where the factor of safety against deep-seated instability is satisfactory and the main problem is to provide a solution to mitigate shoulder instability and widen the shoulder, king-post walls can be used.

In principle a king-post retaining wall consists of embedded vertical structural members acting as king-posts with horizontal structural members spanning between them to form a retaining function. Backfilling with granular fill can be carried out behind the wall to provide the widened shoulder.

The vertical members can be bored or driven concrete or steel piles. Along the shoulder of an embankment space and access for construction is very limited and, hence, driven steel piles such as universal bearing piles using a track-mounted rig would provide a more practical solution. This arrangement eliminates the need for a piling platform and the associated temporary works. The horizontal members can be pre-cast concrete panels fitted between the flanges of the steel pile or against the web. An example of this type of wall is shown in **Figure 71.12**.

Sheet piling

Sheet piles can be installed at the toe or crest of an embankment slope to form a retaining wall. Backfilling behind the sheet piles will then be carried out to widen the shoulder and also increase the toe weight, which contributes to improving stability. The depth of the sheet piles will need to be designed to resist lateral loading as well as the rotational slope instability. A capping beam, steel or concrete, can be constructed to connect the sheet piles together. The sheet piles will need to be designed against corrosion to provide the required design life.

Installation of the sheet piles using a standard crane-mounted vibrating pile-driving method can be unsuitable at some sites, due to restrictions on the use of crane-mounted rigs and on lifting long sections of sheet piles adjacent to railways.

An alternative installation method using the proprietary Giken GRB non-staging system can be considered under these circumstances. This is a silent press method, using installed piles to provide reaction for a hydraulic pressing system. The piling press, power pack, pitching crane and pile transport equipment are all mounted on and travel along the installed piles; hence, no temporary access or staging is required. As the pile lifting equipment is clamped to the installed piles, there is a reduced risk to the railway from toppling plant. An example of this type of sheet piling is shown in **Figures 71.13** and **71.14**.

71.4.3 Remedial work design

For existing slopes, remedial work often needs to be carried out while the transport system running above or below the earthworks remains operational. Hence, the main challenges for existing slopes can be:

- very tight and limited space;
- limited access;
- safe operation of the railway, highway and associated services;
- environmental constraints.

Hence, innovative techniques are required in order to arrive at cost-effective, efficient and safe solutions for remedial work. Many innovative solutions such as the Ruglei shoulder protection system, Giken sheet piling, soil nailing and various types of piling have successfully been implemented. **Figure 71.15** shows the replacement of an old and worn out king-post wall in a difficult access area adjacent to a congested network of cables.

In some cases due to site and access constraints, a solution can only be achieved by providing protection rather than remediation. **Figure 71.16** shows the protection provided for the steep toe of a cutting, which could not otherwise be re-graded due to site constraints, by the placing Armortec revetment blocks.

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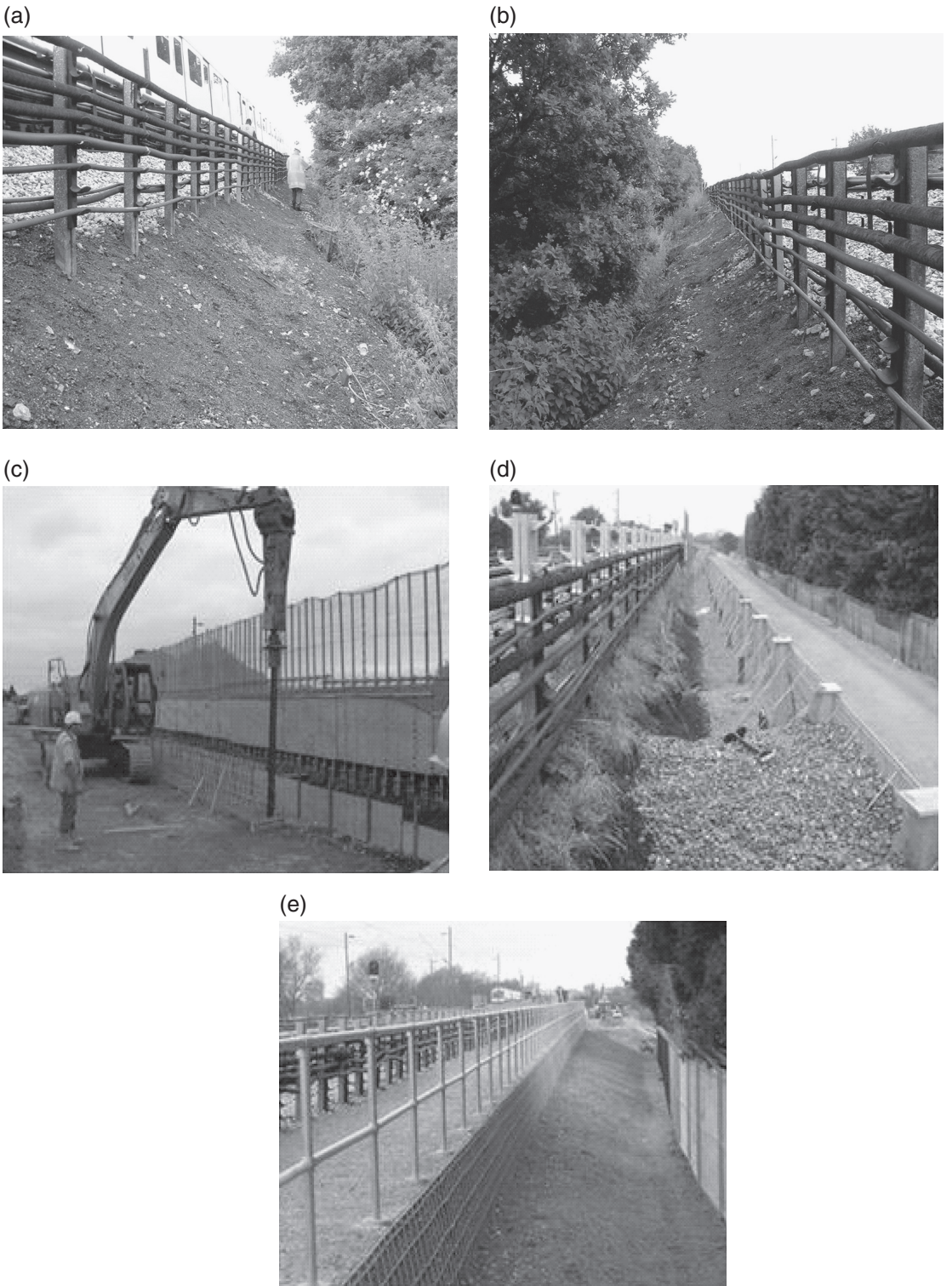


Figure 71.11 Shoulder instability on a railway embankment and construction of a Ruglei™ shoulder stabilisation system: (a) and (b) poor embankment shoulder before remediation; (c) driving of old rail section for Ruglei support; (d) backfilling of Ruglei basket with granular fill; (e) completed Ruglei shoulder stabilisation system



Figure 71.12 An example of a king-post wall installed along the shoulder of a railway embankment. The wall comprises steel universal bearing piles and pre-cast concrete panels. A handrail is fixed to the steel piles after installation of the wall and backfilling was completed: (a) installation of the piles using a track-mounted 13 t excavator over the cable run; (b) the guide rails for pile alignment; (c) installed piles and the temporary platform alongside the embankment for operatives access; (d) and (e) completed wall

71.4.4 Environmental considerations

Environmental factors could have an impact on the performance of an existing earthwork or on the maintenance or construction of the remedial works. Environmental factors could also affect the type of design solution adopted for the remedial works in order to comply with regulations.

These aspects, when identified, have the potential to significantly delay a project until approval from relevant authorities is obtained. Hence, a consideration of environmental aspects should be made from the start of any project in order to ensure that all risks are taken into account. An environmental risk assessment can be carried out at the start of a project and reviewed throughout the development of the project. A list of environmental factors that could have an effect on an earthwork is discussed below.

71.4.4.1 Vegetation

Performance

The effect of vegetation on the performance of earthworks has been discussed in the preceding sections. Whilst the presence of mature trees reduces the pore pressures within a slope leading to increased stability, on clay slopes it can cause seasonal swelling and shrinkage leading to serviceability problems.

The presence of vegetation may serve to reinforce and strengthen a slope through the root system. However, depending on the type of vegetation and the slope geometry, the root system may be quite shallow and ineffective and become unstable during storm events and fall onto the transport corridor below.

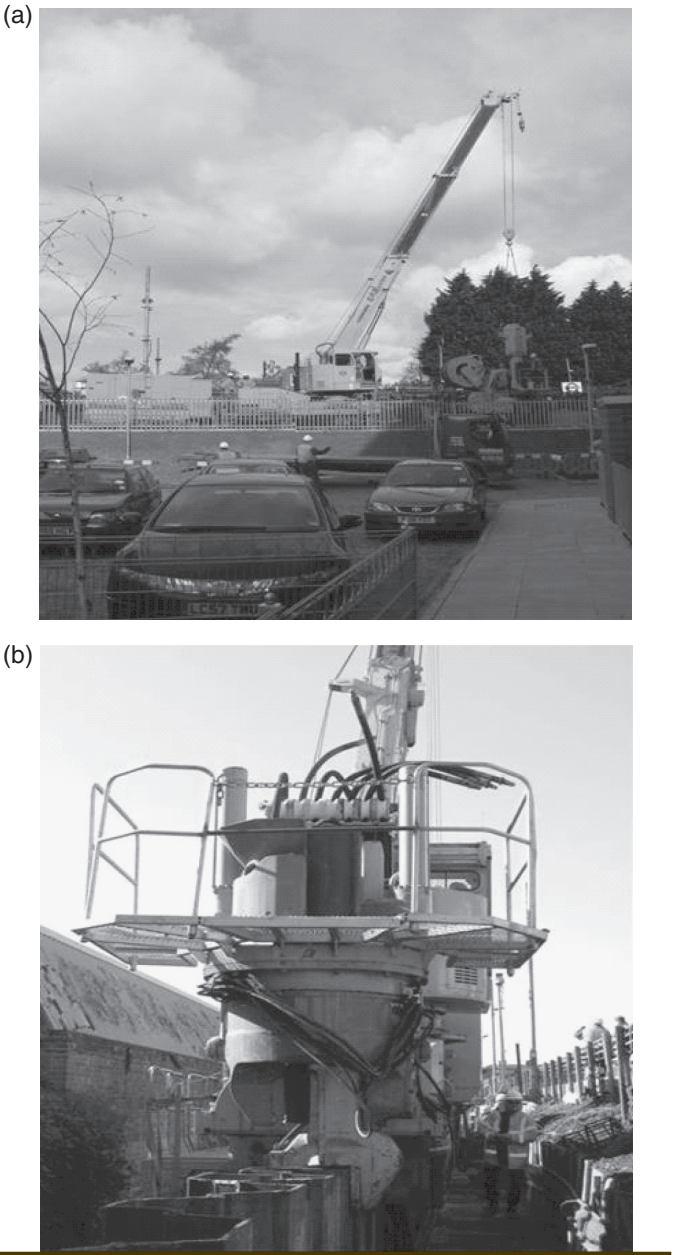


Figure 71.13 Construction of Giken sheet piling at the toe of a railway embankment within a tight space close to third-party buildings: (a) components of the Giken sheet piling rig; (b) piling rig during operation resting on adjacent piles for support

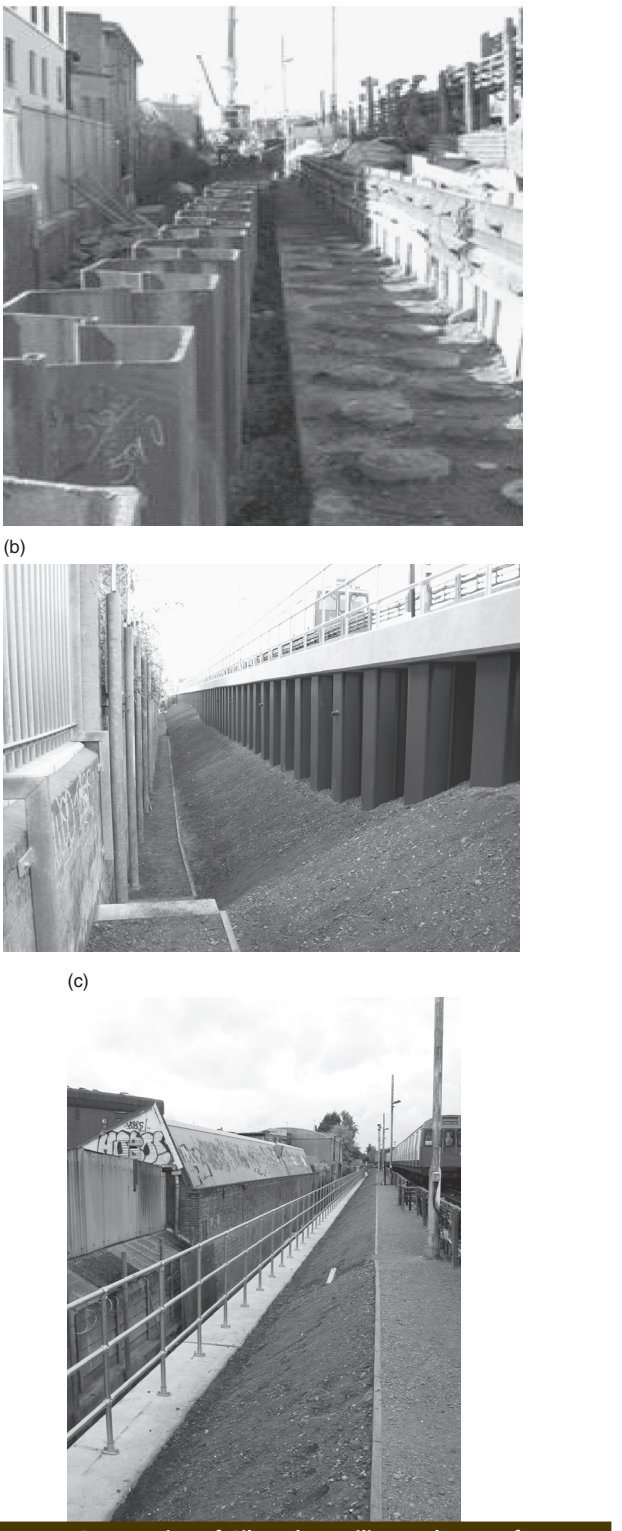


Figure 71.14 Construction of Giken sheet piling at the toe of a railway embankment within a tight space close to third-party buildings: (a) installed piles close to third-party boundary; (b) view along finished wall with capping beam; (c) view of finished embankment from above, next to the railway

Tree Preservation Order (TPO)

Trees which carry a tree preservation order cannot be removed from a construction site. There is usually an exclusion zone associated with these trees, which could be quite significant depending on the type and size of the trees and the root system. This can significantly affect the design of the remedial works in the vicinity of the tree. A preservation order on hedgerows on the crest of a cutting can significantly affect access to the cutting during construction. Hence, it is important that any

(a)



(b)



(c)

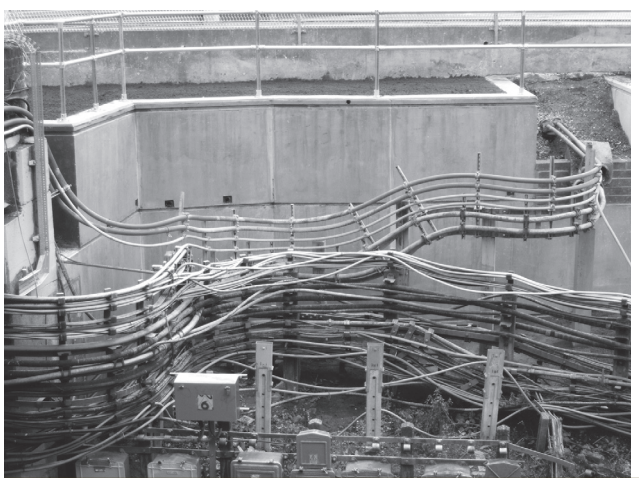




Figure 71.15 Replacement of an old king-post wall adjacent to a congested network of cables: (a) the old king-post wall next to the cables; (b) temporary soil nailed wall with timber facing; (c) constructed mass-concrete wall with cable ducts and drainage pipes

(a)



(b)



Figure 71.16 Steep toe-erosion protection where access to re-grade is not available: (a) installation of the Armortec revetment blocks; (b) a steep toe after installation of the Armortec system

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such orders are investigated at the start of a project in order to avoid any design changes and delays to the project.

Third Party

Vegetation and trees that may have a significant effect on the stability of a cutting or embankment could be located on third-party land. In this case the control of the vegetation may be outside the control owner of the infrastructure. Hence, caution must be applied during the design of the works regarding the long-term pore pressure regime within the structure.

71.4.4.2 Environment Agency (EA)

Streams and rivers located adjacent to an embankment or passing beneath it may be affected by construction activities. Drilling of exploratory holes for the design or drilling of piles may cause pollution in the river. Increasing an embankment's volume as the result of the construction of a berm may affect the EA flood capacity in the vicinity of a river. In this case, consultation with the EA during the design phase would be required. The design of a drainage scheme at the toe of an embankment may include a discharge of the collected water into the river. Prior approval from the EA would be required, including an outfall design that complies with EA requirements.

71.4.4.3 Ecological

For any investigation or remedial work, the following ecological aspects should be considered:

- nesting birds;
- badger setts;
- great crested newts;
- slow-worms.

An ecological survey would be required well in advance of the work so that appropriate mitigation measures are undertaken in order to comply with the relevant standards.

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It is recommended this chapter is read in conjunction with

- Chapter 94 *Principles of geotechnical monitoring*
- Chapter 100 *Observational method*
- Chapter 101 *Close-out reports*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 72

Slope stabilisation methods

Paul A. Nowak Atkins Ltd, Epsom, UK

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Earthworks slopes may require the use of structural solutions where geometric constraints or the proximity of settlement-sensitive structures prevent the design of a full height earthworks solution. Structural solutions may also be used for road widening where further land purchase is not possible, and for the stabilisation of existing earthworks slopes. This chapter introduces the major forms of structural slope solutions.

72.1 Introduction

Stabilised slopes incorporating structural components, nailing or reinforcing elements in new earthworks, are usually employed either:

- where long term performance of a conventional slope may impact adjacent structures e.g. adjacent buildings; or
- where geometric constraints do not allow the construction of a conventional slope that is stable over its design life.

Stabilised slopes are, however, common in stabilising existing assets and the widening of existing motorways where measures cannot extend outside the existing asset boundary.

The selection of a stabilisation method can depend on the balance of the capital expenditure to construct and maintain versus the adoption of a less expensive option that may require additional measures to be applied at a later date. This is usually the decision of the asset owner and can depend on the consequences of failure. A useful approach outlining responsibilities is given in Charles and Watts (2002).

This chapter describes briefly methods to stabilise slopes; they can be divided into:

- embedded solutions;
- gravity solutions;
- reinforced/nailed solutions;
- slope drainage.

In some cases, solutions comprise more than one of the above categories.

72.2 Embedded solutions

Embedded solutions generally comprise:

- concrete pile walls;
- sheet pile walls.

Concrete pile walls usually comprise bored cast *in situ* or continuous flight auger piles. Pile diameter and spacing will depend on the size of plant that requires access to the location and the requirements of the design with respect to bending moment and shear in the pile shaft.

Piled walls are usually designed in order that potential failure surfaces passing below the toe of the wall achieve a satisfactory factor of safety/degree of utilisation as described in Chapter 69 *Earthworks design principles*. Wall design should also consider the action of the piles to act as shear keys on potential failure surfaces passing through the wall, and should be designed as a conventional embedded retaining wall after Gaba *et al*. (2003).

Where the design of the wall cannot satisfy bending moment and shear requirements of the pile shaft, or deflection causing unsatisfactory settlement of the ground behind the wall, additional support can be considered. This would usually comprise (i) cantilever walls; (ii) ground anchors; (iii) raking piles; and (iv) tie bars; see **Figure 72.1**.

Ground anchors, installed at the head of the pile, apply a restraining force in an active manner directly after construction; the active load in the anchor being applied immediately after installation. The design of the anchor should be after BS 8081 (BSI, 1989).

Raking piles, installed in a similar manner to anchors, apply no initial restraining force and act passively whereby forward movement of the wall generates friction on the raking pile shaft. This initiates an anchoring action at the vertical pile head. The raking piles should be designed as conventional piles in tension.

Tie bars act in the same way as raking piles but can be post tensioned to act similarly to ground anchors. They require a restraint mechanism and are most practicably used in the stabilisation of existing earthworks embankments where there is a piled wall on the opposite side of the embankment. This solution has been employed in the stabilisation of London Underground embankments.

Sheet pile walls are designed in the same manner as bored pile walls, but may be precluded where vibration could affect adjacent structures if a driven method of installation is employed. This can be overcome by the use of a hydraulic jacking installation method. The use of a sheet pile solution for the remediation of a failed slope may be attractive as it can be implemented rapidly after the failure occurs.

Care should be taken in the design of sheet pile walls in that the design section chosen should be capable of being driven

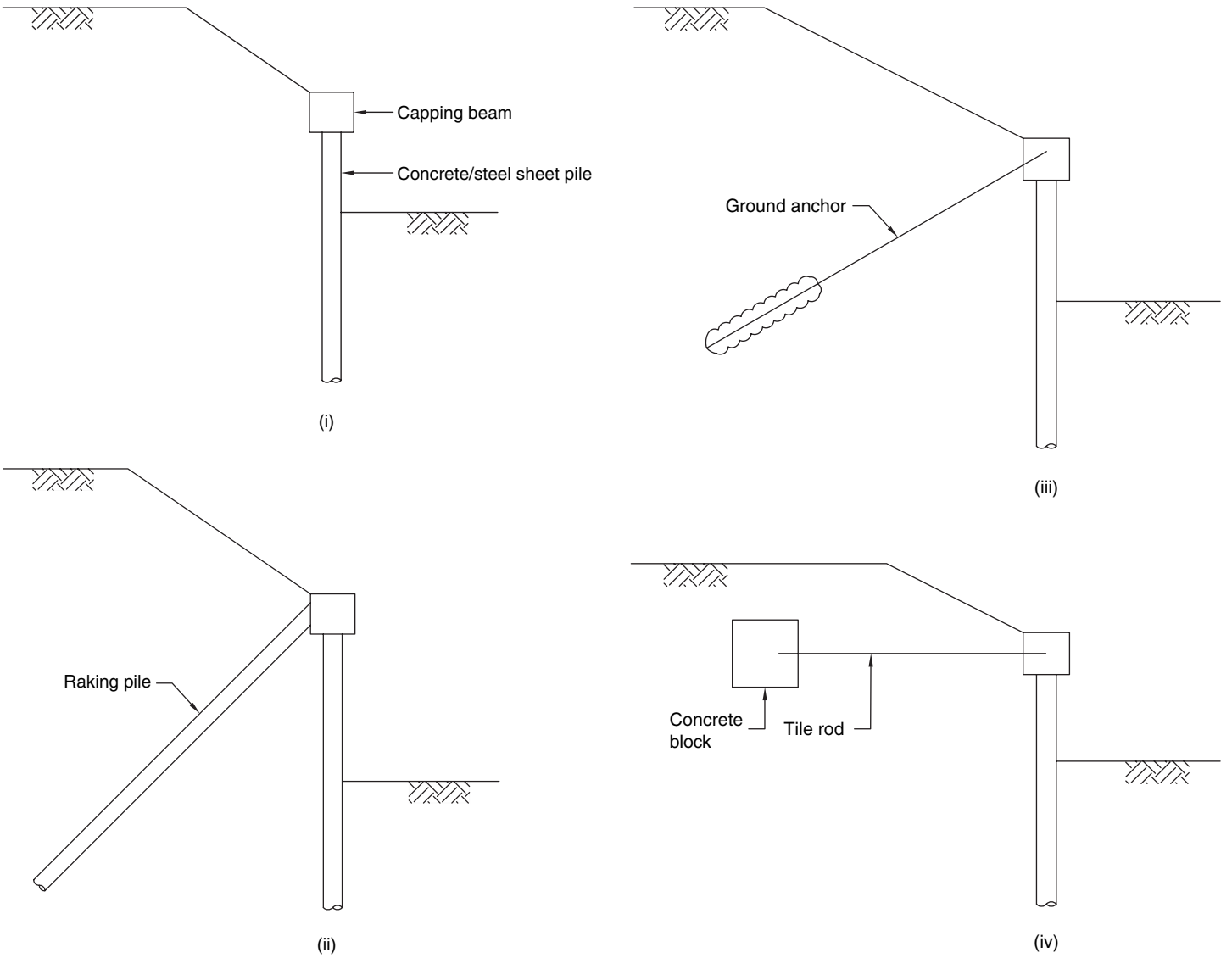


Figure 72.1 Typical embedded solutions: (i) cantilever wall; (ii) wall with ground anchor; (iii) wall with raking pile; (iv) wall with tie and deadman block

in the prevailing ground conditions. Guidance is given in Williams and Waite (1993).

Structural design of bored pile walls should be undertaken to BS EN 1992–1 (BSI, 2004a), BS8110–1 (BSI, 1997), BS8500 (BSI, 2006) or BS5400 (BSI, 2002). Structural design of steel sheet piles should be undertaken to BS EN 1993– 5 (BSI, 2007).

More advice on the design of embedded walls is given in Section 6 *Design of retaining structures*.

72.3 Gravity solutions

Gravity structures would usually comprise the following solutions: (i) reinforced concrete wall; (ii) gabion wall; (iii) dry block wall; (iv) crib wall; (v) reinforced soil wall; and (vi) reinforced earth wall. Typical solutions, without soil reinforcement, are illustrated in **Figure 72.2**.

The gravity structure performs under its own stability in terms of its resistance to sliding, overturning and bearing capacity failure. They can be designed in accordance with Chapman (2000). Reinforced earth and reinforced soil walls should be designed in accordance with BS 8006 (1995 and 2009b) as described in Chapter 73 *Design of soil reinforced slopes and structures*. More advice on the design of reinforced gravity walls is given in Section 6 *Design of retaining structures*.

In addition to the stability of the gravity structure, the designer should also consider the stability of the slope that it retains. It is common for walls greater than 3 m which retain a slope behind them to require embedment of the wall below

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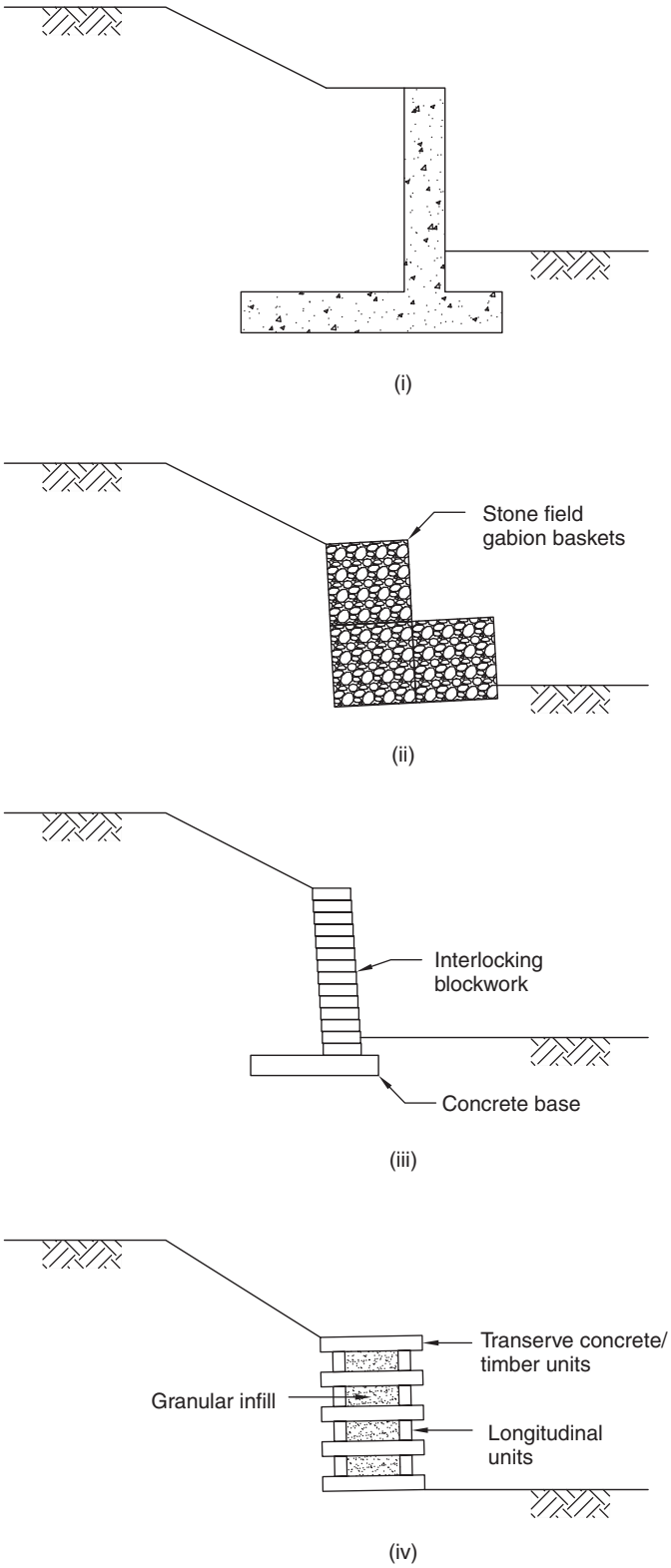


Figure 72.2 Typical gravity solutions: (i) reinforced concrete wall; (ii) gabion wall; (iii) dry block wall; (iv) crib wall

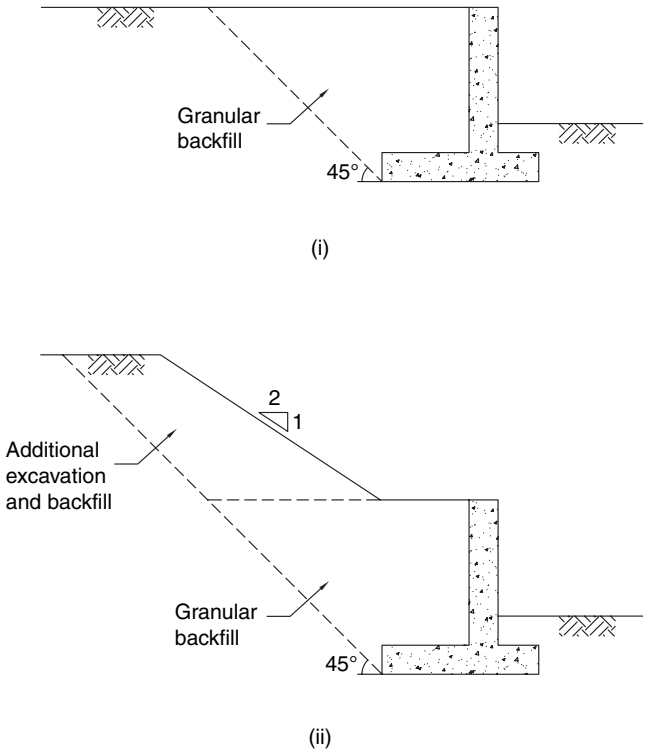


Figure 72.3 Effect of sloping ground on excavation and backfill volume: (i) gravity wall with horizontal ground behind; (ii) gravity wall with sloping ground behind

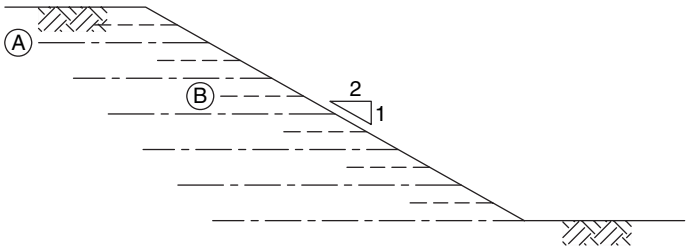
ground surface on the passive side in order to prevent overall failure of the composite slope and wall.

It is usual that gravity structures are designed with granular backfill in the area of the active wedge behind the wall. Care should be taken where a gravity structure is specified on existing or widened infrastructure, as it may result in significant excavation of the slope behind the wall as shown in **Figure 72.3**.

It is common that gravity structures, particularly masonry and brick walls, form part of an existing infrastructure asset. Where an assessment is required as part of an asset condition survey, this will be carried out to current best practice i.e. BS EN 1997 (BSI, 2004b) or, formerly, BS 8002 (BSI, 1994). It should be borne in mind, particularly with respect to railway infrastructure, that the structure may be up to 100 years old and would not have been constructed to current best practice. Assessment of the structure to BS EN 1997 or BS 8002 is likely to indicate failure of the structure or earthwork which shows no visible structural distress. In this case, the asset owner should be consulted and a sensible approach to the management of the asset agreed.

72.4 Reinforced/nailed solutions

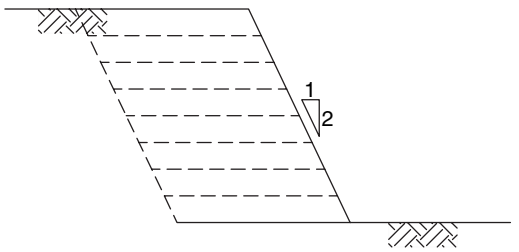
Reinforced or nailed slopes will generally comprise the use of geogrid reinforcement or a grid of soil nails to provide additional tensile strength along likely failure surfaces. Typical examples are shown in **Figure 72.4**.



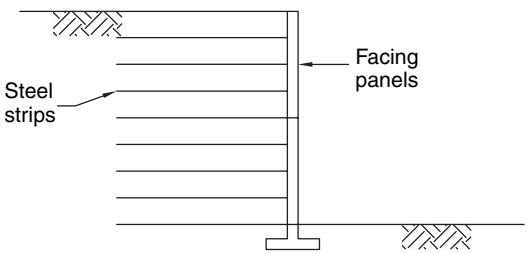
(A) Primary reinforcement

(B) Secondary reinforcement to prevent local failure of face between primary grids

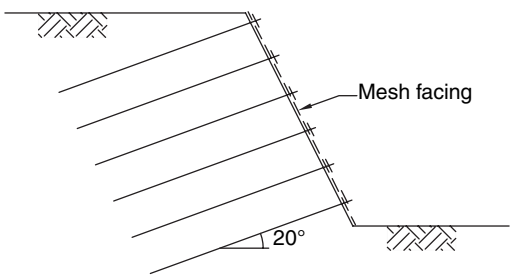
(i)



(ii)



(iii)



(iv)

Figure 72.4 Typical reinforced/nailed solutions: (i) reinforced soil slope; (ii) reinforced soil block; (iii) reinforced earth slope; (iv) soil nail slope

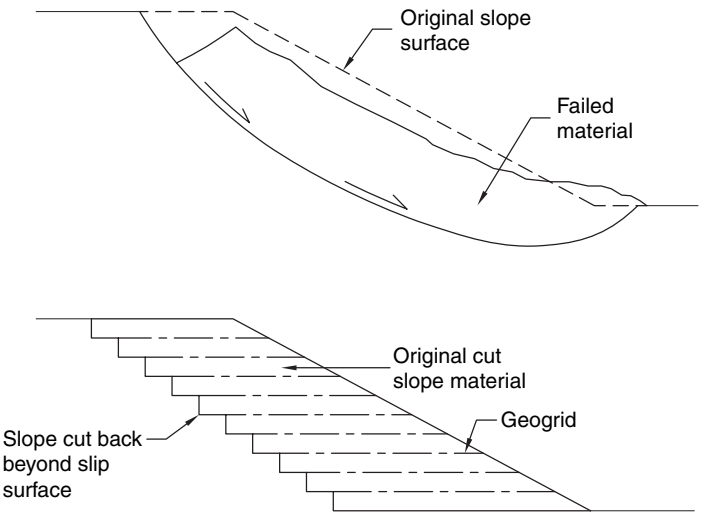


Figure 72.5 Repair of slope failure using geogrids

Design should be carried out in accordance with BS 8006 and further guidance is provided in Chapters 73 *Design of soil reinforced slopes and structures* and 74 *Design of soil nails*.

The use of soil reinforcement and nailing is not confined to steep-faced (approximately 60°) slopes; it can be applied to slopes of any angle where additional tensile strength of the reinforcement or nailing allows the adoption of a stable slope at a steeper angle than if no reinforcement were employed. On the Terminal 5 spur road project at Heathrow airport, geogrid reinforcement was employed to allow the use of 1V:2H slopes in London Clay embankments where geometric constraints prevented the adoption of 1V:4H slopes used for the majority of the embankment construction.

Geogrids have also been used to repair slope failures, particularly cutting slopes. They were extensively used on the M4 motorway in Berkshire in the 1970s and were also used in a London Clay cutting on London Underground at Hendon. The slipped material is excavated back beyond the slip surface, stockpiled and re-compacted with layers of geogrid to provide additional tensile strength to the original failed material as shown in **Figure 72.5**.

This solution may not always be practicable if there is insufficient facility to temporarily store the slipped material locally. If storage of excavated material is not available local to the failed area it is common practice for the slip repair to be effected using imported granular material.

72.5 Slope drainage

Stability of constructed slopes can be influenced by water either as groundwater (see Chapter 70 *Design of new earthworks*) or as precipitation.

The former may result in deep seated slope failure if uncontrolled and not in line with design assumptions, particularly in inter-bedded cohesive and non-cohesive strata; groundwater

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flow in the non-cohesive strata causes softening of the surrounding cohesive material.

The ingress of precipitation can cause softening of the surface layers of cohesive earthworks, which leads to reduction in any effective cohesion in the slope material and the development of shallower failure surfaces. These are commonly seen on major road networks and require remediation.

It is common practice to counteract these problems with slope drainage, either as part of the original slope construction or as a remedial works measure.

Slope drainage usually takes the form of gravel-filled drains orthogonal to the slope (counterforts) or as a 'herring bone' pattern. The drains are commonly constructed with a carrier pipe and geotextile wrap to prevent the ingress of fine-grained material which would impact on their long-term performance.

Drains are usually spaced at intervals of between 5 and 10 m along the slope, depending on the composition of the slope material. Useful guidance on their design and the philosophy behind slope drainage is provided by Hutchinson (1977).

In designing slope drainage, a suitable outfall has to be considered to prevent ponding of drainage water which will occur at the base of the earthwork, potentially resulting in the softening of the toe and increasing the potential for failure in this area.

Slope drains can be connected to a drain constructed parallel to the toe of an embankment slope, or to verge drainage in a cutting. Outfall of the system can be by means of positive flow, such as an existing river course or existing public sewers, or to a sustainable drainage system (SUDS), Woods and Kellagher (2007).

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72.6.1 Further reading

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It is recommended this chapter is read in conjunction with

- Chapter 23 *Slope stability*
- Chapter 69 *Earthworks design principles*
- Section 6 *Design of retaining structures*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 73

Design of soil reinforced slopes and structures

Sebastien Manceau Atkins, Glasgow, UK
Colin Macdiarmid SSE Renewables, Glasgow, UK
Graham Horgan Huesker, Warrington, UK

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Reinforced soil structures are composite constructions involving some form of reinforcement (usually geosynthetic or metallic), generally installed in horizontal layers within a soil mass. The reinforcement layers extend beyond the potential failure surface, absorb the tensile strains that would otherwise cause failure of the non-reinforced soil and redistribute them in the soil beyond the failure plane. This chapter aims to introduce the general concepts of soil reinforcement, the various materials and general principles involved in the design of reinforced soil structures. Reinforced soil walls and abutments, reinforced soil slopes and basal reinforcement are discussed.

73.1 Introduction and scope

The concept of reinforced soil structure consists of placing (typically horizontal) layered reinforcing elements with sufficient axial tensile stiffness within a soil mass to improve its tensile and shear capacities. This allows the construction of reinforced soil slopes and reinforced soil walls at significantly steeper angles – which would be unstable without the reinforcement. When used as basal reinforcement, this also allows the construction of embankments over poor ground or areas prone to subsidence that would otherwise fail without the reinforcement in their foundations.

The idea of introducing reinforcing elements to improve the strength of fill can be traced back to the earliest human times, when primitive people used sticks and branches for the reinforcement of mud dwellings. The earliest remaining example of reinforced fill is the Aqar Quf ziggurat built in modern Iraq by the Babylonians around 3000 BC, using clay bricks, reinforced with woven mats of reed laid horizontally on layers of sand and gravel, with plaited ropes of reed passing through the structure. Parts of the Great Wall of China built more than 2000 years ago also adopted a form of reinforced fill where tamarisk branches were used to reinforce a mixture of clay and gravel.

In the early 1960s, Henri Vidal introduced and developed the modern form of reinforced fill using flat metallic reinforced strips laid horizontally on a frictional fill. In the late 1960s and 1970s, extensive studies of reinforced soil structures sponsored by national bodies, notably the Laboratoire central des ponts et chaussées (LCPC) in France, led to a fuller understanding of the concepts involved and a better acceptance of the method. More or less simultaneously, advances in synthetic fabrics led to the development of geotextiles and geogrids. These new materials were soon put to good use in the construction of geosynthetic-reinforced soil structures and as basal reinforcements. It should be noted that reinforced soil is the general term for reinforcing elements placed within a soil mass and covers both metallic and polymeric reinforcement. The term 'reinforced earth' is the trademark for the Reinforced Earth Company – using its founder's (Henri Vidal) 'Terre Armée' concept.

This chapter introduces the various types of reinforcements and their properties, discusses the general principles of the reinforcement action and provides guidance for the design of reinforced soil structures. Reinforced soil walls and abutments, reinforced soil slopes and basal reinforcement are then covered in more detail. Particular forms of reinforcement such as anchored earth (Chapter 89 *Ground anchors construction*), soil nails (Chapter 74 *Design of soil nails*), and particular application of basal reinforcement such as load transfer platforms (Chapter 70 *Design of new earthworks*) are discussed elsewhere in this manual.

73.2 Reinforcement types and properties

73.2.1 Types of reinforcement

There are essentially two main types of reinforcement, defined by their extensibility.

- Extensible reinforcement: defined in BS 8006 (BSI, 1995) as reinforcement that sustains the design loads at strains greater than 1% and which are usually polymeric.
- Inextensible reinforcement: defined in BS 8006 as reinforcement that sustains the design loads at strains less than or equal to 1% and which are usually metallic.

The design life of the reinforcement can vary from a few months (e.g. basal reinforcement for embankment on soft ground) to up to 120 years (e.g. walls, abutments and slopes). The principal function of the reinforcement is to withstand tensile loads; the ability of the reinforcement to perform its primary function, both initially and throughout its design life, will largely depend on the material from which it is produced.

73.2.2 Properties of polymeric reinforcement

Polymeric reinforcement can take a variety of forms, such as grids, meshes, strips (commonly used to reinforce slopes and walls) and geotextile sheets (commonly used for basal reinforcement).

All polymers are essentially nonlinear viscoelastic materials, and as such are load rate dependent. In addition, polymers are subject to creep and their behaviour is therefore time dependent.

When subjected to a constant load, all materials will increase in strain over time. This phenomenon is known as creep and is significant in polymers at ambient temperatures. The tensile rupture strength of polymeric reinforcement will reduce over time, predominantly due to creep from an initial short-term ultimate tensile rupture strength (UTS) to a tensile creep rupture strength at the end of the selected design life. The propensity of polymeric reinforcement to creep will be primarily dependent on the particular polymer used to produce the geosynthetic reinforcement (e.g. polypropylenes tend to have a greater creep propensity than polyesters).

Isochronous curves plot the tensile load against the strain for a given time (during which the reinforcement has been subjected to the tensile force). The curves are derived from extensive testing and interpolations following guidelines in PD ISO/TR 20432 (BSI, 2007) and are unique to each proprietary geosynthetic reinforcement. The stress axis is generally normalised by expressing the tensile load as a percentage of the initial short-term UTS of the reinforcement.

These curves enable the designer to establish both the initial strains and post-construction strains for polymeric reinforcement at a given load/stress level; strains must remain below prescribed design limits. BS 8006 imposes varying serviceability limits on post-construction strain depending on the type of structure considered. Post-construction strains are limited to 0.5% for bridge abutments and retaining walls with permanent structural loads, and to 1.0% for retaining walls with no applied structural loading. For slopes where deformations are not critical, post-construction strains in the order of 5% may be acceptable. There usually is no limit on post-construction strain for basal reinforcement (used for embankment on soft ground) but there can be limits imposed when the basal reinforcement is used over areas prone to subsidence.

Additionally, the curves allow the designer to determine the variations of the reinforcement stiffness with time.

A typical set of isochronous curves are shown in **Figure 73.1**.

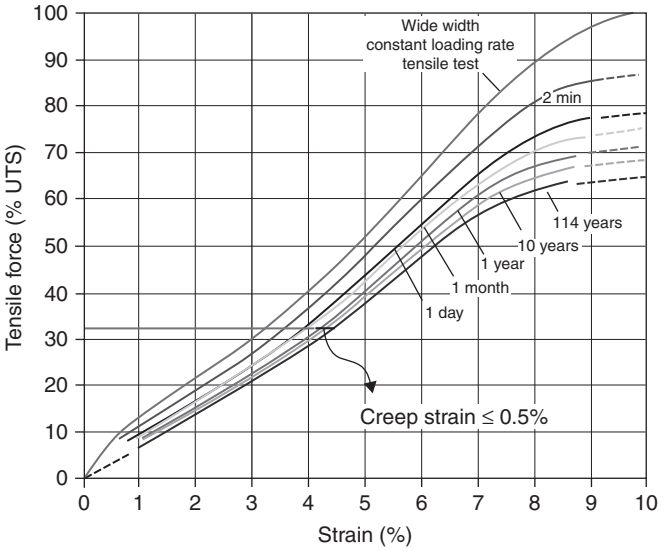


Figure 73.1 Typical isochronous curves for polyester-based reinforcement
Courtesy of Huesker (Isochrones for Stabilenka product)

73.2.3 Properties of metallic reinforcement

Metallic reinforcement can take a variety of forms, such as grids, meshes, strips, bars or rods (commonly used to reinforce walls and abutments).

Creep in metallic reinforcement is generally negligible in ambient temperatures. Metallic reinforcement can therefore be assumed to exhibit linear elastic behaviour to yield, independent of time. Hence, the tensile yield strength of metallic reinforcement will reduce over time, almost uniquely due to corrosion.

Corrosion in metallic reinforcement is essentially a process of oxidation; this forms a protective oxide layer which tends to retard further corrosion. The effects of corrosion are considered in design by allowing for a loss of the cross-sectional area of the reinforcement. BS 8006 provides guidance on the prescribed sacrificial thickness to allow for at the design stage, which varies depending on:

- particular metal or alloy and corrosion protection used;
- design service life of the reinforcement;
- corrosivity of the fill, soil or environment in which the reinforcement is placed (e.g. high acidity soil, which would be highly corrosive).

73.3 General principles of reinforcement action

73.3.1 Effects of introducing reinforcements in soil

When an inclined or vertical load (due to soil self-weight or surcharge) is applied to a soil, this will generate axial compressive strain in the soil and a corresponding lateral tensile strain. The maximum load that can be applied to a soil is limited by its internal shear strength. The introduction of reinforcement into the soil (of adequate stiffness) improves the shearing resistance of the soil (shear strength of reinforced soil = mobilised shearing in the soil + mobilised tensile force in the reinforcement) and has the effect of reducing both the axial compression and lateral deformations.

Deformation of the soil along a potential failure plane causes shear forces to develop in the soil and tensile forces to develop in the reinforcement intercepting the failure plane. The reinforcement

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is most effective when placed in the direction in which the tensile/lateral strains develop. However, in most practical reinforced soil applications, the reinforcement is installed horizontally (with the exception of soil nails which are commonly inclined at an angle between 10 and 20° from the horizontal).

73.3.2 Strain compatibility

The relative magnitude of the shear force mobilised in the soil and the tensile force mobilised in the reinforcement will be a function of deformations and the relative stiffness properties of both soil and reinforcement (stiffer reinforcement will carry a higher percentage of the mobilised force). The magnitude of mobilised soil shearing resistance and the mobilised reinforcement strain need to be considered to provide equilibrium.

Inextensible (metallic) reinforcement will absorb, through frictional contact, the disturbing forces in the active zone at low soil deformations and transfer these forces beyond the failure plane. For more extensible polymeric reinforcements, the issue of strain compatibility becomes increasingly important – to ensure design reinforcement strain limits remain compatible with mobilised soil strains.

Figure 73.2 shows the mobilised shear resistance of a typical granular soil and the typical mobilised reinforcement force (time constant).

From the graph in **Figure 73.2(a)** it can be seen that as strains develop, the mobilised shear resistance in the soil increases (and the required reinforcement force to satisfy equilibrium decreases) up to a point where the mobilised shear strength reaches a maximum or peak strength ϕ'_p. Beyond this point (ϕ_m), increasing deformations result in reducing shear strength towards a minimum value ϕ'_{cv} (shear strength at constant volume) independent of strain, with a corresponding increase in the force required to satisfy equilibrium. At the same time, as strains develop, the force mobilised in the reinforcement corresponding to a specific loading period (t_d) and temperature (T_d) increases (**Figure 73.2(b)**).

Strains develop until a strain level is reached (in both the soil and the reinforcement) whereby the force mobilised in the reinforcement is that required to satisfy equilibrium (**Figure 73.3**).

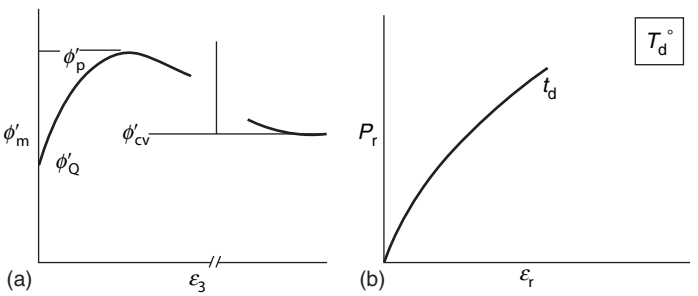


Figure 73.2 Relations for strain compatibility: (a) mobilised soil shearing resistance, (b) mobilised reinforcement force. P_r = mobilised reinforcement force; t_d = loading period; T_d = temperature; ε_3 = maximum tensile strain

Reproduced, with permission, from CIRIA SP123, Jewell (1996), www.ciria.org

At this point, further deformations will only occur as a result of additional loading, or through stress relaxation, or creep, of the reinforcement over time.

The stress–strain response of soil will vary depending on a number of factors including mineralogy and stress history; similarly, the stress–strain response of reinforcement will vary depending on a variety of factors including its form, manufacturing process and raw material. It is generally not practical to develop a set of compatibility curves for every soil type and proprietary reinforcement product available. BS 8006 provides practical recommendations for addressing strain compatibility by adopting certain values depending on the type of reinforced soil application. For example, ϕ'_p would be adopted for walls and steep slopes where mobilised strains and deformations are considered more critical to the overall performance, and ϕ'_{cv} for shallow slopes and basal reinforcement over soft soils where deformations are considered less critical.

73.3.3 Interaction between soil and reinforcement

Reinforcement develops a bond with the soil through either friction (for granular soils) or adhesion (for cohesive soils). There are two interaction modes:

- direct sliding, in which a block of soil slides over a layer of reinforcement;
- pull-out (bond), in which a layer of reinforcement pulls out of the soil after mobilising the maximum available bond stress.

73.3.3.1 Direct sliding

The sliding resistance of geotextile reinforcement is generated over the full plan area of contact. The sliding resistance of geogrids and strips is generated from both direct contact between the reinforcement and the soil, and from soil-to-soil contact through the apertures or adjacent to the reinforcement.

Modified direct shear tests are suitable for measuring the coefficient of direct sliding between soil and any type of reinforcement.

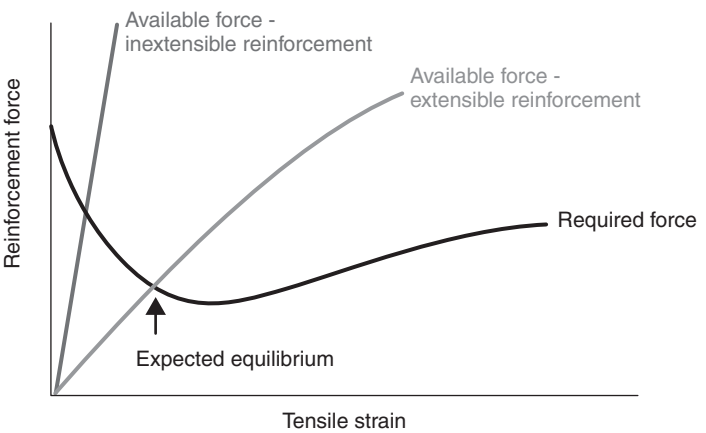


Figure 73.3 Typical compatibility curve for determining equilibrium in reinforced soil

Adapted, with permission, from CIRIA SP123, Jewell (1996), www.ciria.org

For cohesive foundation soils, subject to undrained loading, the bond stress developed between the soil and the reinforcement is directly related to the undrained shear strength of the soil by an adhesion factor, which will normally have a value less than unity.

73.3.3.2 Pull-out

The pull-out resistance of geotextile reinforcement is generated from surface shear over the full plan area. Geogrids develop pull-out resistance partly from surface shear and partly through bearing resistance from the transverse members of the geogrids. It is usually sufficient for the design to calculate the bond coefficient from theoretically derived values for these two components.

For narrow, rough, strip reinforcement embedded in dense cohesionless fill, the shear stresses developed during pull-out lead to dilatancy in the fill, which causes the vertical effective stress to locally rise above the overburden pressure. This gives rise to an enhanced pull-out resistance. It is generally recommended that field pull-out tests be carried out to verify the bond strength of these types of reinforcement.

Pull-out tests are relatively difficult to perform and can be significantly influenced by the boundary conditions of the test. They can, therefore, be unreliable.

73.4 General principles of design

The design of reinforced soil structures and slopes is not covered by the Eurocodes. In the UK, design of reinforced soil structures follows BS 8006 *Code of Practice for Strengthened/Reinforced Soils and Other Fills* (or HA68/94 *Design Methods for the Reinforcement of Highway Slopes by Reinforced Soil and Soil Nailing Techniques*; Highways Agency, 1994). BS 8006 follows a limit state approach and provides partial factors of safety calibrated for reinforced soil structure design through the assessment of historical data. It should be noted that the standard was first published in 1995 and that a new version has been published in 2010. The following text applies to both standards as there have been minimal changes between the two.

73.4.1 Limit state approach

Reinforced soil structures are designed against the occurrence of ultimate limit states (ULS) and serviceability limit states (SLS). ULSs are associated with collapse, rupture or major damage, and SLSs with excessive settlement or deformations.

73.4.2 Partial factors of safety approach

BS 8006 provides various partial factors of safety which are used to generate the overall safety of reinforced soil structures. Although BS 8006 does not specifically group them in this fashion, it is convenient to think of the partial factors of safety as being introduced to reduce material properties and resistances, and increase loads (a similar approach to that used in the Eurocodes).

A partial factor for ramification of internal failure (f_n) is also introduced to account for the ramifications (consequences) of internal failure. The value of f_n depends on the class of risk for the structure and introduces additional safety for structures where the consequences of failure are most onerous. This factor is applied both as an additional material factor and as an additional resistance factor on the reinforcement strength and fill reinforcement interactions, respectively.

73.4.2.1 Partial material factors

A partial material factor of safety on reinforcement (f_m) is applied on the reinforcement capacity to account for uncertainty on the reinforcement properties (manufacture and extrapolation of test data), susceptibility to damage during construction (e.g. due to shape and size of fill used) and environmental effects (e.g. temperature, exposure to ultraviolet radiation).

The partial factor for ramification of internal failure (f_n), as discussed above, is also applied to further reduce the reinforcement design strength and therefore acts as an additional material factor.

A partial material factor of safety is applied on soil strength parameters (f_{ms}) to account for uncertainty on the characteristic value of the soil parameters considered.

73.4.2.2 Partial load factors

Partial load factors are applied to increase the soil self-weight (f_{fs}), the external dead loads (f_f) and the external live loads (f_q). For walls and abutments, the worst load combination for the various design criteria must be identified and considered in the design.

BS 8006 uses high values for partial load factors on fill or soil self-weight, which is one of the parameters that is likely to be well known and not subject to large variations. However, it should be borne in mind that the values of partial factors of safety provided in BS 8006 have been specifically calibrated for reinforced soil structures. Even if the logic behind a particular partial factor looked at in isolation is not always apparent, the overall application of the full set of partial factors of safety leads to safe designs.

73.4.2.3 Partial resistance factors

For the internal failure mechanisms (i.e. within the soil/reinforcement block) involving fill/reinforcement interaction, a partial factor of safety on pull-out resistance (f_p) is applied to the resistance generated by fill/reinforcement interaction behind the potential failure surface, when this intersects a layer of reinforcement. A partial factor of safety on sliding resistance along reinforcement (f_s) is also applied to the resistance generated by fill/reinforcement interaction when the base of a potential failure surface coincides with a layer of reinforcement.

The partial factor for ramification of internal failure (f_n) is also applied to further reduce the fill/reinforcement resistances (pull-out and direct sliding) and therefore acts as an additional resistance factor.

For the external mechanisms (e.g. sliding or bearing capacity), a partial factor on sliding resistance along soil (f_s) is applied

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to the resistance generated by soil/soil interaction on any horizontal plan, either within or at the base of the reinforced soil structure where there is soil to soil contact. A partial factor on bearing capacity resistance (f_{ms}) is also applied to the resistance generated by the ultimate bearing capacity. Please note that BS 8006 considers this partial factor of safety as a material factor (hence the notation f_{ms}) however, bearing capacity is not an intrinsic soil property and, therefore, in the following the partial factor on bearing capacity is considered as a resistance factor.

73.5 Reinforced soil walls and abutments

73.5.1 General

Reinforced soil walls and abutments are composite structures consisting of fill, reinforcement and a facing, as illustrated in **Figure 73.4**. Provided the reinforcement layout and properties are adequate to prevent an internal failure of the reinforced soil block, the block of reinforced soil retains the soil at its back as a mass gravity structure, and transmits and spreads the effects of surcharges and earth pressures over its wide base. Reinforced soil walls are cost-effective soil retaining structures that can tolerate large settlements. Reinforced soil structures with a face within 20° from the vertical can be analysed as reinforced soil walls.

73.5.2 Materials

73.5.2.1 Fill

Both the mechanical and chemical properties of the fill need to be assessed. Generally, frictional class 6I/6J fill of the *Specification for Highways Works* (Highways Agency, 1993) is used in the construction of reinforced soil walls and abutments. When steel metallic reinforcements are used, the electro-chemical properties of the fill also need to be considered.

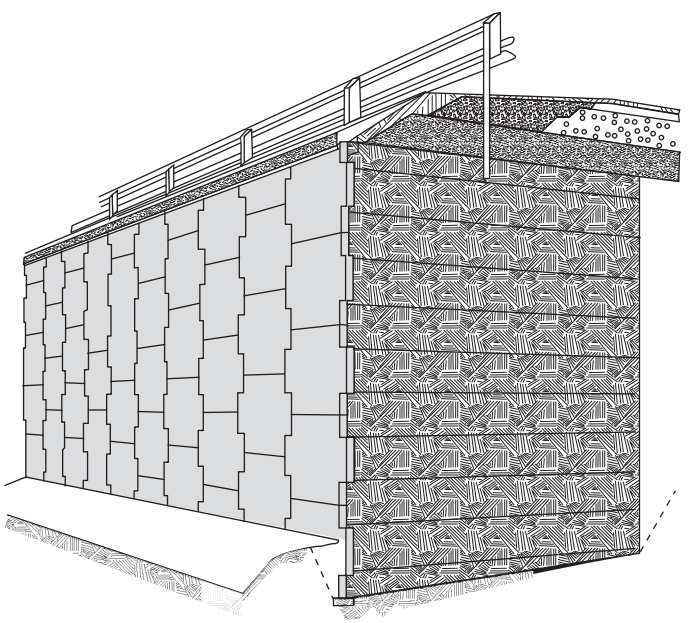


Figure 73.4 3D section through reinforced soil structure
Courtesy of Terre Armée Internationale

Polymeric reinforcements are not influenced by electro-chemical properties, but the influence of chemicals on the proprietary polymeric reinforcements need to be assessed.

In certain circumstances, provided durability and compatibility with the reinforcement have been considered, cohesive fills, pulverised fuel ash, colliery spoil, argillaceous materials and chalk can be used (for details, refer to BS 8006, section 3). The selection and specification of fill for reinforced soil structure should be considered as early as possible in the project, and aspects such as sustainability, availability, impacts on design and costs should be borne in mind. The use of material other than class 6I/6J fill (provided durability and compatibility with the reinforcement have been considered) may lead to a design requiring longer, more closely spaced and stronger reinforcements, and to increased earthworks fill testing requirements during construction. However, the use of alternative material could also lead to a significant reduction in the quantities of selected fill to be imported – if site-won material can be re-used (thus reducing the quantity of material to be disposed of off-site). It could also reduce haulage costs if sources of acceptable fills are identified in the vicinity.

73.5.2.2 Reinforcement

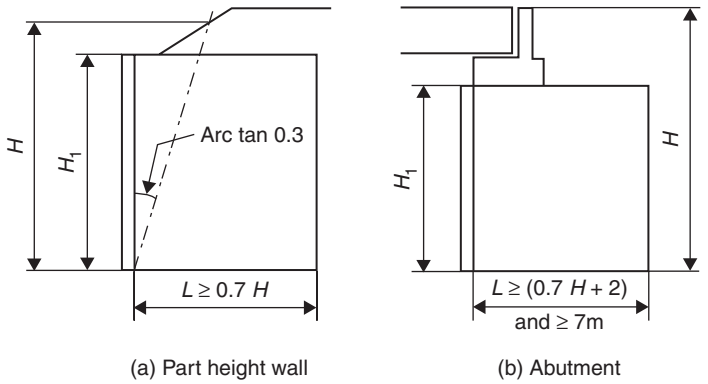
The reinforcement in reinforced soil walls and abutments consists of either metallic strips or polymeric geogrids. The reinforcement properties and the principles of their interaction with the fill are discussed in sections 73.2 and 73.3 above.

73.5.2.3 Facing

The facing provides local support to the fill between reinforcement layers, prevention against weathering of the exposed soil and an aesthetically acceptable finish. Common facing types include full height panels, discrete panels, segmental blocks and wrap-arounds. They are constructed of concrete (reinforced or mass), steel, wood or polymers. Discrete panels (in hexagonal, square or cross patterns) are the most commonly used with metallic reinforcement, and segmental blocks are commonly used with polymeric reinforcements for permanent structures where a hard facing is acceptable. Wrap-arounds for polymeric reinforcement can provide a green finish but are more susceptible to ultraviolet degradation, accidental damage and vandalism. They are commonly used in temporary structures.

73.5.3 Common geometries and typical dimensions

Figure 73.5, in which L is the reinforcement length, illustrates common reinforced soil retaining wall and abutment geometries that can be used for assessing options at conceptual design stage and for preliminary sizing. These dimensions are given relative to the mechanical height H. For a wall, the mechanical height is derived as the vertical distance between the base of the reinforced soil block and the intersection of the finished profile and a line at 1V:0.3H from the front bottom corner of the reinforced soil block. For an abutment, H is the vertical height between the base of the reinforced soil block and the finished



(a) Part height wall (b) Abutment

Figure 73.5 Common geometries and typical dimensions for (a) a reinforced soil wall; and (b) an abutment
Reproduced with permission from BS 8006–1 © British Standards Institution 2010

profile at the back of the abutment. Dimensions chosen for preliminary sizing may need amending following detailed design.

Other geometries including trapezoidal walls can be designed to suit particular site requirements.

73.5.4 Elements of design

The design of reinforced soil walls considers two distinct aspects.

- The external stability covers the design of the reinforced soil structure as a unit and follows the principles used for a general mass gravity retaining structure.
- The internal stability covers the internal mechanisms within the reinforced soil block and deals with considerations of stress distribution and reinforcement layout.

73.5.4.1 External stability

For external stability the following ULSs and SLSs should be considered:

- bearing and tilt failure – ULS;
- sliding – ULS;
- slip failure – ULS;
- settlement – SLS.

The factored applied bearing pressure at the base of the reinforced soil block (q_r) is derived using a Meyerhof distribution (equivalent rectangular distribution on a reduced area) as:

$$q_r = \frac{R_v}{L - 2e} \qquad (73.1)$$

where

R_v resultant of all factored vertical load components;
L reinforcement length at the base of the wall;
e eccentricity of resultant load R_v about the centre line of the base of width L.

The bearing capacity of the foundation soil derived using traditional bearing capacity equations must be greater than the applied bearing pressure (refer to Chapter 53 *Shallow foundations* for further details on bearing capacity).

At the base of the reinforced soil block, sliding should be checked either on a soil/soil interface or, if a layer of reinforcement is provided at the base of the reinforced soil structure, on a soil/reinforcement interface.

Any potential slip failure passing at the back of the reinforced soil structure should also be checked using traditional slope stability analysis software.

For the settlement check, both the settlement of the foundation soil and the internal settlement of the reinforced fill should be considered (refer to Chapter 70 *Design of new earthworks* for further details on fill self-settlement). However, when properly compacted, the selected fill used in reinforced soil walls and abutments will produce small internal settlements, and this component is often neglected.

In current UK practice, the internal stability of the reinforced structure is often designed by suppliers who do not generally assess the external stability. Instead, they provide target values for allowable bearing capacity of the foundation soil to prevent bearing capacity failure; target values for the shear strength of the foundation soil to prevent sliding; and SLS applied bearing pressures at the base of the reinforced soil block to allow an assessment of the settlements to be undertaken. It is critical to understand the limitations of such suppliers' designs and to ensure that:

- The external stability is adequately assessed, based on the actual ground investigation information and that, where required, ground improvement is applied to the foundation soil (or the geometry of the reinforced soil block is amended, or a combination of both) to ensure external stability is satisfied.
- The responsibilities of the various parties and the objectives/specification of the design are clearly defined (for most common applications, BS 8006 will provide a safe basis for design). Furthermore, notwithstanding the expertise of the suppliers, suppliers' designs should not be taken at face value.
- All other aspects such as drainage, services, fencing, aesthetics that could have an impact on the design are considered and communicated between the parties involved, to ensure an integrated overall design is achieved.

The court case that followed subsidence on a load transfer platform in Enniskillen, Northern Ireland (High Court of Justice in Northern Ireland, Queen's Bench Division, 2005), whilst not covering a reinforced soil wall or abutment, illustrates what can go wrong when the principles above are not followed.

73.5.4.2 Internal stability

Within the reinforced mass, there are two zones delimited by a potential failure surface. In the active zone (between the face of the wall and a potential failure surface) the fill sheds shear stress into the reinforcement and induces a tensile force in it. The reinforcement transfers this tensile force in the resistant

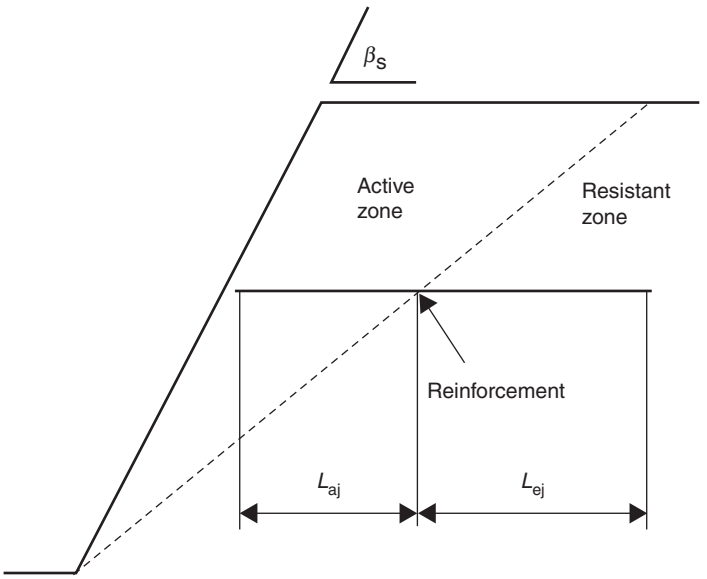


Figure 73.6 Internal stability of reinforced soil wall. L_{aj} = length of reinforcement within the active zone, L_{ej} = length of reinforcement within the resistant zone, β_s = slope angle

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zone (between the potential failure surface and the back of the wall) where the reinforcement sheds shear stress into the fill (**Figure 73.6**).

L_{aj} = length of reinforcement within the active zone.

L_{ej} = length of reinforcement within the resistant zone

β_s = slope angle

For the internal stability, the following ULSs and SLSs are considered:

- Local stability of a layer of reinforcement – ULS:
  - rupture of the reinforcement (or/and connection);
  - breakdown of the interaction between the fill and the reinforcement (pull-out or/and direct sliding);
- instability of a wedge of any size or shape assumed to behave as a rigid body – ULS;
- deformations – SLS.

Extensible reinforcements in which the design load is sustained at an axial tensile strain greater than 1% (polymeric reinforcement) are designed using the tie-back wedge method which is based on a theoretical approach.

Inextensible reinforcements in which the design load is sustained at an axial tensile strain less than 1% (metallic reinforcement) are designed using the coherent gravity method. This method is mainly empirical and is based on observations on instrumented reinforced soil structures.

Both methods follow broadly similar steps which are summarised below.

Tensile force in reinforcement

Both the tie-back wedge and the coherent gravity methods calculate the maximum factored tensile force in the *j*th layer of reinforcement T_j which is given as:

$$T_j = Tp_j + Ts_j + Tf_j \tag{73.2}$$

where:

Tp_j tensile force at *j*th layer of reinforcement generated by self-weight of fill, surcharge and retained backfill;

Ts_j tensile force at *j*th layer of reinforcement generated by vertical load applied to a strip;

Tf_j tensile force at *j*th layer of reinforcement generated by horizontal load applied to a strip.

The component due to the fill self-weight, surcharge and retained backfill is given as:

$$Tp_j = K\,\sigma v_j\,Sv_j \tag{73.3}$$

where:

K earth pressure coefficient. For extensible reinforcement (tie-back wedge method) active earth pressure coefficient K_a shall be used throughout the structure. For inextensible reinforcement (coherent gravity method) values varying between at rest earth K_0 at the top of the structure and active K_a lower down the structure shall be used.

σv_j vertical stress acting on the *j*th layer of reinforcement assuming the Meyerhof distribution. The derivation of σv_j varies slightly between the tie-back wedge and the coherent gravity method (refer to BS 8006 for details).

Sv_j Vertical spacing of the reinforcement at the *j*th level in the wall.

The component due to any vertical load applied to a strip is given as:

$$Ts_j = K\,\Delta\sigma v_j\,Sv_j \tag{73.4}$$

where $\Delta\sigma v_j$ is the increase in vertical stress generated by strip vertical loading at the level of the *j*th layer of reinforcement. The tie-back wedge method assumes a simple 2V:1H dispersion of the strip load through the reinforced fill. The coherent gravity considers the dispersal based on the Boussinesq equation for strips with an edge on the front face of the wall, and applies the principle of superposition to represent strip at a distance from the front face of the wall. The mathematics involved in the original coherent gravity method can be cumbersome and the 2010 revision of BS 8006 allows the use of the simple 2V:1H load dispersal assumption in conjunction with the coherent gravity method.

The component due to any horizontal load (F_L) applied to a strip is given as:

$$Tf_j = \alpha_j\,F_L\,Sv_j \tag{73.5}$$

where $\alpha_j\,F_L$ is the horizontal stress generated by strip horizontal loading at the level of the *j*th layer of reinforcement. The

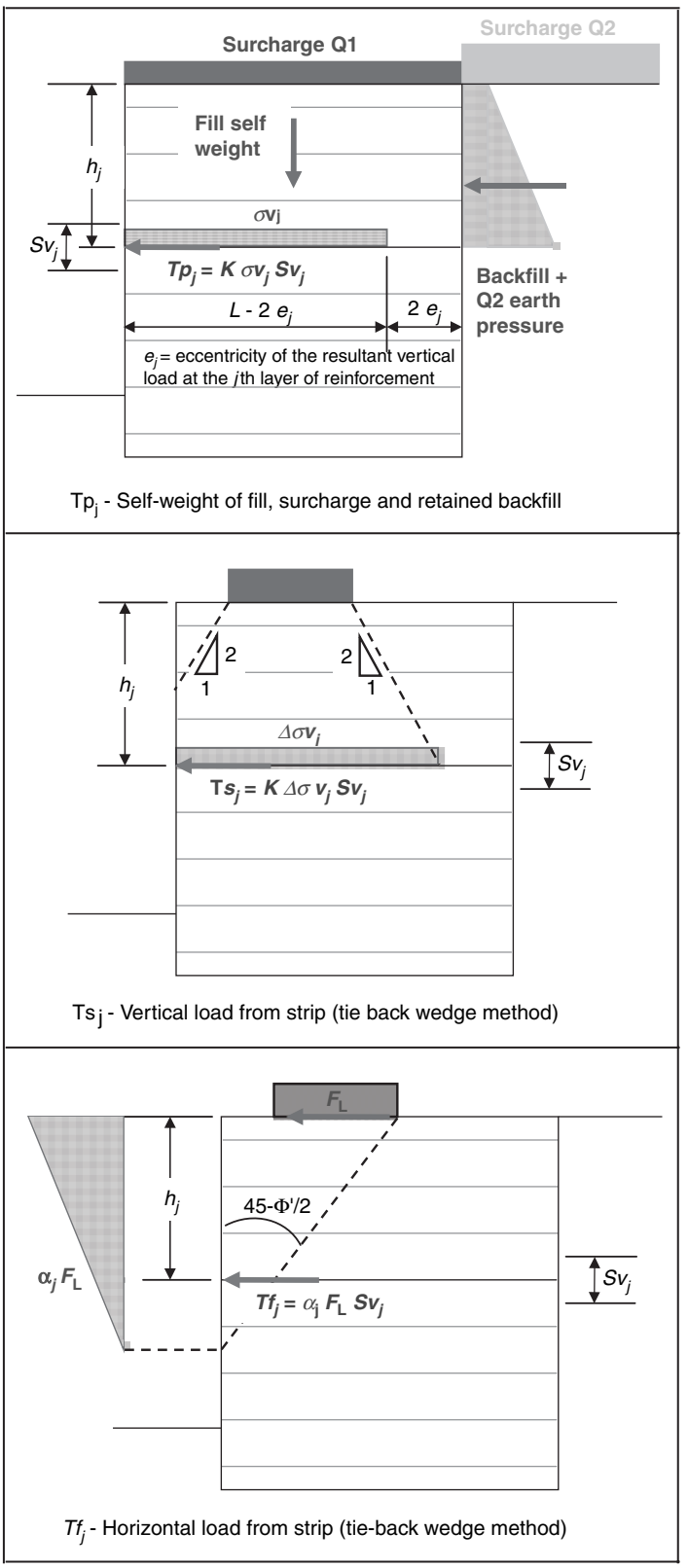


Figure 73.7 Tensile force in the *j*th layer of reinforcement

tie-back wedge method assumes a dispersion of the strip load at 45° + $\varphi'/2$ from the back of the strip through the reinforced fill. The coherent gravity assumes a dispersion of the strip load at 45° from the back of the strip through the reinforced fill.

The various components of the tensile force in the *j*th layer of reinforcement (for the tie-back wedge method) are illustrated in **Figure 73.7**.

Local stability checks

For each layer of reinforcement, check that the design tensile strength of the reinforcement is greater than the factored tensile force to prevent *rupture* of the reinforcement. Also, for each layer of reinforcement, check that the design adherence capacity of the reinforcement behind the failure wedge being considered is greater than the factored tensile force to prevent *pull-out* of the reinforcement.

For extensible polymeric reinforcement subject to creep the serviceability criterion is generally more critical than the ULS local stability check (see 'Serviceability' on page 1102 below).

Surface failure/lines of maximum tension

In the tie-back wedge method, the failure plane with the most tensile force needs to be determined by considering the stability of wedges, of any size or shape, assumed to behave as rigid bodies. The friction along the potential failure plane and the tensile resistance of reinforcement beyond the failure plane (rupture capacity or pull-out capacity, whichever is the smallest) must be greater than the disturbing applied loads.

In the coherent gravity method, results of monitoring installed along reinforcements have shown that the tensile force varies along the reinforcement. Lines of maximum tension are defined (the shape of the line considered depends on whether strip loadings are applied or not) and the tensile force in the reinforcement is calculated on these defined failure surfaces. Provided the local stability checks (rupture and pull-out) are satisfied on these defined failure surfaces, there is no requirement to carry any further wedge stability analysis (unless the geometry or loadings are unusual).

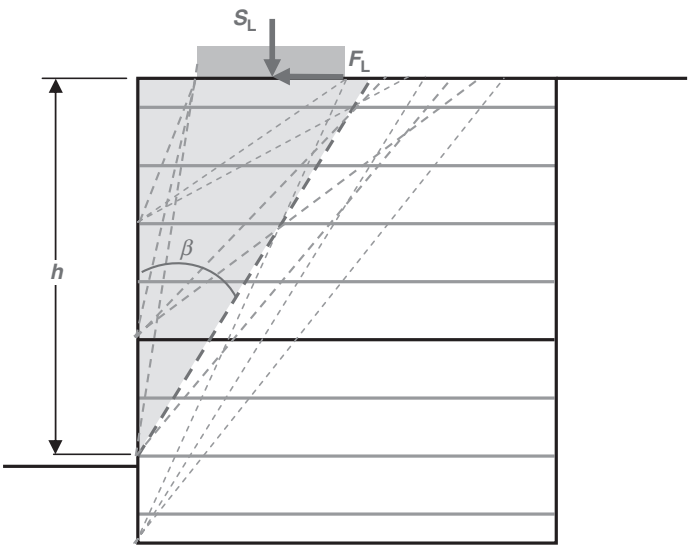
The failure plane in the tie-back wedge method and the lines of maximum tension in the coherent gravity method are illustrated in **Figure 73.8**.

Connections

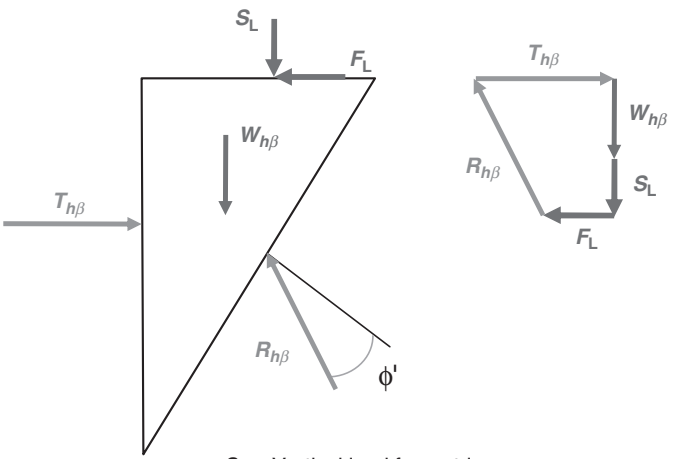
The tensile force in the connection between the facing and the reinforcement depends on their properties. Unless the facing is stiff (e.g. full height panel with no movement capacity at connections), the tensile force in the connection will be less than the maximum tensile force calculated. However, the tensile force at connection is significant and the connection has to be designed to prevent rupture under this load throughout the design life. When metallic reinforcements are used, the connection generally involves a bolting arrangement that leads to the tensile capacity at the connection being less than that of the full bar – and this detail is particularly vulnerable to corrosion.

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Tie back wedge



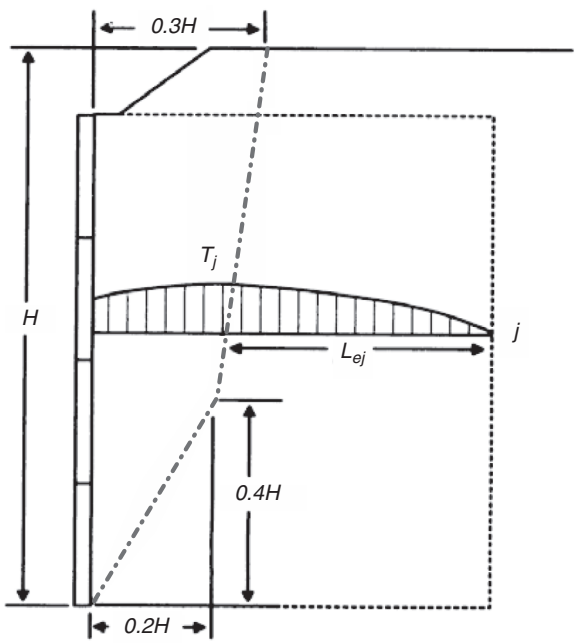
The triangular shaded area highlights the specific wedge for which the forces diagram is presented on the right hand side

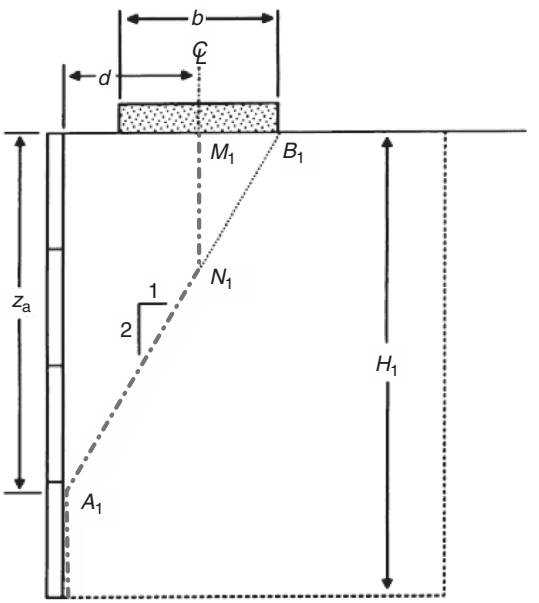


S_L Vertical load from strip
F_L Horizontal load from strip
$W_{h\beta}$ Self weight of fill in the wedge and any surcharge
$R_{h\beta}$ Resultant reaction acting on potential failure plane
$T_{h\beta}$ Tensile force to be resisted by the reinforcement

Vary h and β to obtain the maximum tensile force to be resisted by the reinforcement

Coherent gravity





z_a = min: $2(d + b/2)$ or H_1

The lines of maximum tension can be assumed as illustrated above

Figure 73.8 Surface failure/lines of maximum tension. Coherent gravity
Reproduced with permission from BS 8006–1 © British Standards Institution 2010

Facings

Finally, the facings are designed structurally to resist the connection tensile loads, any externally applied loads (e.g. impact loadings from traffic) and the effect of any construction tolerance. In the case of segmental block facings, sliding and movements between blocks and overtopping of the blocks above the uppermost layer of reinforcement must be checked.

Serviceability

Deformations post-construction can happen as the result of creep of polymeric reinforcement, where the tensile stiffness decreases with time from an initial short-term value. In order to limit movements due to creep, the capacity of the reinforcement should be considered in conjunction with a limiting post-construction value of strain (typically 0.5% for abutments and 1% for walls) at the end of the design life, using isochronous load strain curves. For metallic reinforcement, creep is negligible.

73.5.5 Construction

Factors affecting the construction of reinforced soil walls and abutments include:

- nature and properties of the soil at the base of the reinforced soil block;
- site controls and placement and compaction requirements for the specified fill;
- safe storage and handling of the reinforcement;
- facings;
- simplicity and robustness of the connection detail;
- requirements for drainage;
- site constraints;
- end use;
- erection rate.

Buildability is discussed in more detail in BS 8006 and in Chapter 86 *Soil reinforcement construction*. Recommended construction tolerances are also presented in BS 8006.

73.6 Reinforced soil slopes

73.6.1 General

Reinforced soil slopes are composite structures consisting of fill, reinforcement and facing. The front face is usually grassed up to provide a green slope finish. A distinction is made between 'shallow slopes' defined as no steeper than 45° from the horizontal, and 'steep slopes' defined as being between 45° and 70° from the horizontal.

73.6.2 Materials

73.6.2.1 Fill

Generally, frictional fill will be used as fill for reinforced soil slopes, but the range of fill that can be used is wider than for reinforced soil walls and abutments. As discussed in section 73.5.2.1 for reinforced soil walls and abutments, the selection and specification of fill for reinforced soil structure should be considered as early as possible in the project. Aspects such as sustainability, availability, impacts on design and costs should be borne in mind. Site-won material is often used as an economic and sustainable fill for reinforced soil slopes. Properties of particular significance are the drained shear strength parameters (φ' and occasionally c', although effective cohesion should be used with great care) which will govern the soil reinforcement pull-out capacity, and the maximum particle size which will govern the risk of mechanical damage during construction – and hence the reinforcement rupture capacity.

73.6.2.2 Reinforcement

For reinforced soil slopes, geogrids are predominantly used. The reinforcement properties and the principles of their interaction with the fill are discussed in sections 73.2 and 73.3 above.

73.6.2.3 Facing

For shallow slopes (no steeper than 45° from the horizontal) it is generally possible to place the fill without permanent or temporary supports. Intermediate short secondary geogrids are often placed between layers of main reinforcements to prevent small shallow seated failures in the front face. Some form of erosion mat is often provided to ensure that the topsoil placed does not erode in the short term, and to facilitate the establishment of vegetation. Health and safety issues associated with cutting grass on relatively steep slopes should be considered.

For steep slopes (steeper than 45° from the horizontal) the facing is generally built using either wrap-around or steel mesh. In a wrap-around detail, the reinforcing geogrid or geotextile is folded through 180° to form the face and is anchored back into the fill or connected to the next layer (up) of reinforcement. Temporary formwork is often required to allow placement and compaction of the fill. In a steel mesh facing arrangement, the reinforcing geogrid or geotextile can be connected directly to the steel mesh, or continued behind the face of the steel mesh and returned into the fill to allow for the future deterioration of the sacrificial steel facing. In both cases, topsoil is placed just behind the facing to facilitate the establishment of vegetation. In addition to health and safety issues associated with cutting grass on such steep slopes, the difficulties of establishing, irrigating and maintaining vegetation on them requires careful consideration.

Typical reinforced soil slope configurations are presented in **Figure 73.9**.

73.6.3 Elements of design

Similar to the design of reinforced soil walls and abutments, the design of reinforced soil slopes considers both external and internal stability. In current UK practice, the stability of reinforced soil slopes is often designed by suppliers who, unless they are provided with adequate information, tend to

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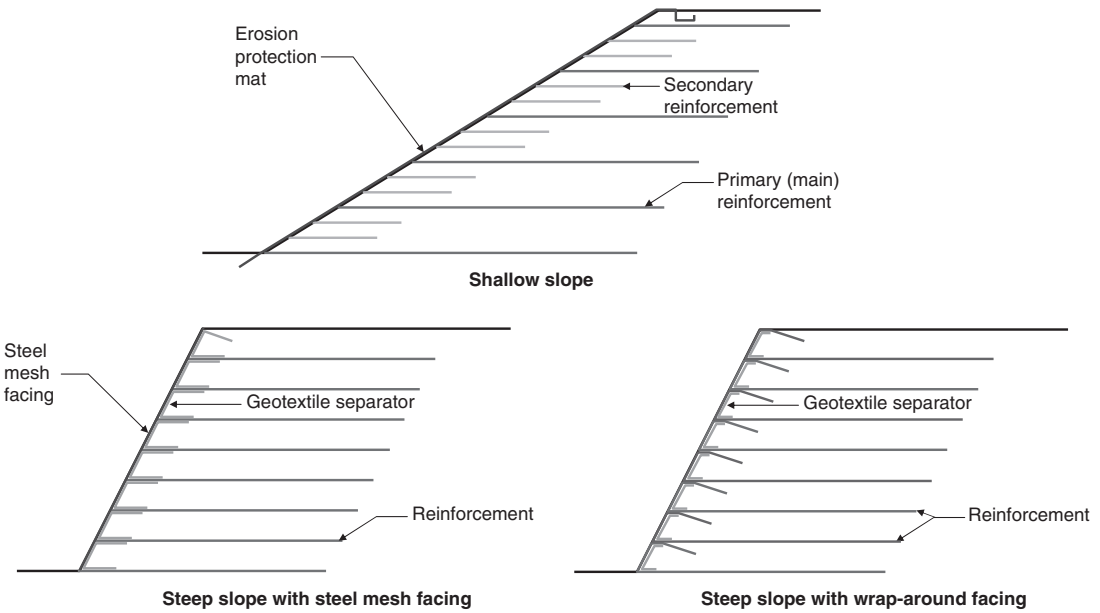


Figure 73.9 Typical reinforced soil slope configurations

concentrate on internal stability and the principles enounced in section 73.5.4.1 apply. In addition, compound stability, where the potential failure surface is partly within the reinforced soil zone and partly behind it, should also be considered.

73.6.3.1 External stability

For external stability the following ULSs and SLSs are considered:

- bearing and tilt failure – ULS;
- sliding – ULS;
- slip failure – ULS;
- settlement – SLS.

73.6.3.2 Internal stability

Similar to reinforced soil walls and abutments, a potential failure surface delimits an active zone and a restraint zone. For the internal stability, the following ULSs and SLSs are considered:

- local stability of a layer of reinforcement – ULS;
  - rupture of the reinforcement (or/and connection);
  - breakdown of the interaction between the fill and the reinforcement (pull-out or/and direct sliding);
- deformations – SLS.

Internal stability assessment is generally undertaken using one of many limit equilibrium methods available. The most commonly used are the two-part wedge mechanism and the method of slices, which are discussed below. Other methods, such as a non-circular analysis, a log spiral or a coherent gravity method, could be used.

Two-part wedge mechanism

The two-part wedge mechanism is an extension of the Coulomb wedge approach as illustrated in **Figure 73.10**. The mechanism may take any form (i.e. θ_1, θ_2 and X can vary) provided the mechanism outcrops at the toe of the slope and the inter-wedge boundary is vertical. By resolving the forces acting on the wedges (which requires assumptions on the inter-wedge boundary conditions), the out-of-balance force that needs to be taken by the reinforcement can be assessed for each mechanism. For each slope there is a unique critical two-part wedge mechanism for which the out-of-balance force that needs to be taken by the reinforcement is maximum. For more details on the two-part wedge mechanism, refer to HA68/94 (Highways Agency, 1994).

Considering the simple case of a slope with flat ground at the top and no surcharge, the maximum factored total force to be taken by the reinforcement T_{max} can be expressed as:

$$T_{max} = 0.5\, K\, \gamma\, H^2 \tag{73.6}$$

where:

K non-dimensional parameter equivalent (but not equal) to an earth pressure coefficient;

γ fill unit weight (kN m^{-3});

H fill height (m).

The factored tensile force in the jth layer of reinforcement T_j is given as:

$$Tj = K\, \gamma\, z_j\, Sv_j \tag{73.7}$$

where:

z_j depth below crest level to jth layer of reinforcement;

Sv_j vertical spacing at jth layer of reinforcement.

For each layer of reinforcement, a local stability check should be undertaken. The design tensile strength of the reinforcement should be greater than the factored tensile force to prevent *rupture* of the reinforcement. The design adherence capacity of the length of reinforcement (L_{ej}) behind the wedge considered should be greater than the factored tensile force to prevent *pull-out* of the reinforcement.

For each layer of reinforcement, the smallest of the design tensile strength and the design adherence capacity can be referred to as the design capacity of the *j*th layer of reinforcement: *Tdesj*. For the local stability check, it should be confirmed that for each layer of reinforcement:

$$Tdes_j \geq T_j \tag{73.8}$$

The overall reinforcement provided should be sufficient to prevent rupture of the two-part wedge mechanism, and it should be checked that:

$$\sum_{j=1}^{n} Tdes_j \geq T_{max} \tag{73.9}$$

Slice method for circular slip mechanism

The method of slices is traditionally used for unreinforced soil slopes and is adapted to take into account the effects of introducing the reinforcement layers. The additional restoring forces and moments provided by the reinforcing elements beyond the slip failure are considered in the calculations of the slope stability factor of safety. BS 8006 only considers moment equilibrium and suggests that inter-slice forces are ignored in the calculations. However, most current slope stability software allows for analyses using both force and moment equilibrium, and will consider some form of inter-slice interaction.

For each layer of reinforcement, a local stability check (**Figure 73.11**) should be undertaken and it should be verified that the design capacity of the reinforcing layer (either the pull-out resistance or the rupture strength of the reinforcement) is greater than the tensile load generated in the layer.

The overall reinforcement provided shall be sufficient to prevent slope failure and it shall be verified that:

$$F_{RS} + \sum_{j=1}^{n} Tdes_j \geq F_D \quad \text{(force equilibrium)} \tag{73.10}$$

$$M_{RS} + \sum_{j=1}^{n} \left(Tdes_j \times Y_j\right) \geq M_D \quad \text{(moment equilibrium)} \tag{73.11}$$

where

Y_j vertical distance from centre of rotation of slip to the reinforcement layer under consideration;

F_{RS} (M_{RS}) restoring force (moment) due to shear strength of soil;

F_D (M_D) disturbing force (moment) due to the weight of soil and surcharge.

73.6.3.3 Compound stability

Compound stability considers failure mechanisms where the potential failure surface is partly within the reinforced soil zone and partly behind it. The principles for analysing compound stability are similar to those for analysing internal stability.

73.7 Basal reinforcement

73.7.1 General

Reinforcement can be introduced at the base of embankments over poor ground to avoid ULS failure through shear in the foundation of the embankment and/or SLS failure through excessive deformations. It should be noted that BS 8006 still limits the applications of basal reinforcement to foundations for earthworks. The common applications are:

- Basal reinforcement over soft to very soft foundation soil to control initial ULS stability of the embankment without controlling the settlement.

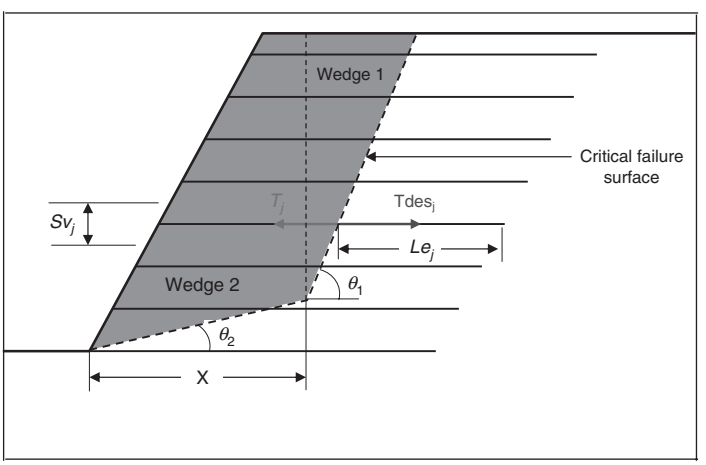


Figure 73.10 Two-part wedge mechanism

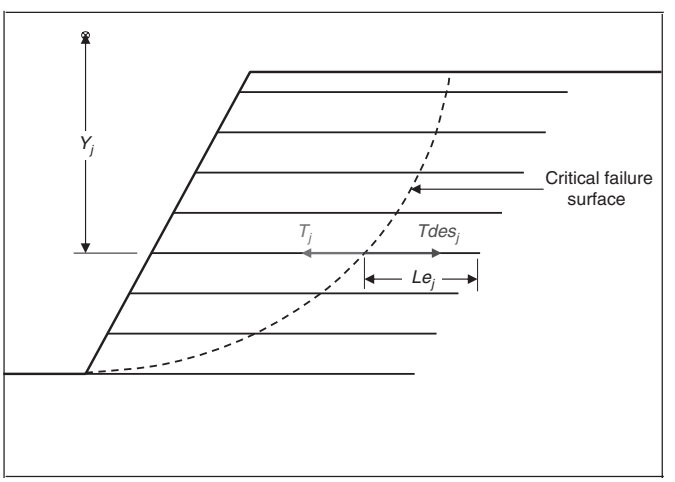


Figure 73.11 Circular slip failure mechanism

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- Basal reinforcement combined with another form of ground improvement technique over soft to very soft foundation soils (typically piled embankment with basal reinforcement acting as a load transfer platform) to control both the initial ULS stability and long-term SLS settlements of the embankment.
- Basal reinforcement over foundation soils prone to subsidence (e.g. mining) where the reinforcement is designed to span the void (ULS stability) and limit the deformations at the top of the embankment.

This chapter will focus solely on the first application of a simple basal reinforcement over soft to very soft foundation soil. For details of the other applications, refer to BS 8006.

The products used in basal reinforcement applications generally consist of geotextiles or geogrids, but products such as steel meshes are sometimes considered.

73.7.2 Basal reinforcement for an embankment on soft to very soft ground

When constructing an embankment on soft soil, shear failures are likely to develop through the foundation soil. The problem is always more acute in the short term (during construction) since soft soils will consolidate and strengthen over time due to the dissipation of the excess pore water pressures generated in the soil by the construction of the embankment. Once consolidation is complete, the basal reinforcement is often redundant. Basal reinforcement is often used in conjunction with other construction techniques (e.g. staged construction discussed in Chapter 70 *Design of new earthworks*) to limit the capacity required from the reinforcement. Strain compatibility between the reinforcement and the soft foundation soil should be satisfied in order to achieve a maximum bond coefficient. This is particularly the case when dealing with sensitive foundation soils that exhibit rapid strength loss post-peak, hence the strain in the reinforcement (SLS case) should not exceed the strain at which the peak shear strength is mobilised in the foundation soil (see section 73.3.2).

The ULS limit equilibrium mechanisms to consider are:

- deep-seated failure;
- lateral sliding;
- extrusion.

73.7.2.1 Deep-seated failure

The analysis of deep-seated failure is generally performed using the method of slices for a circular failure mechanism described in section 73.6.3 above. The reinforcement provided should have a sufficient design capacity *Tdes* (against rupture and pull-out beyond the failure mechanism) to provide moments and forces equilibrium. In the case of basal reinforcement, it is also necessary to ensure that the length of reinforcement within the failure surface mechanism is sufficient to prevent pull-out (**Figure 73.12**).

73.7.2.2 Lateral sliding

The reinforcement provided shall have a sufficient design capacity *Tdes* (against rupture and pull-out beyond the failure mechanism) to resist the outward thrust (active earth pressures)

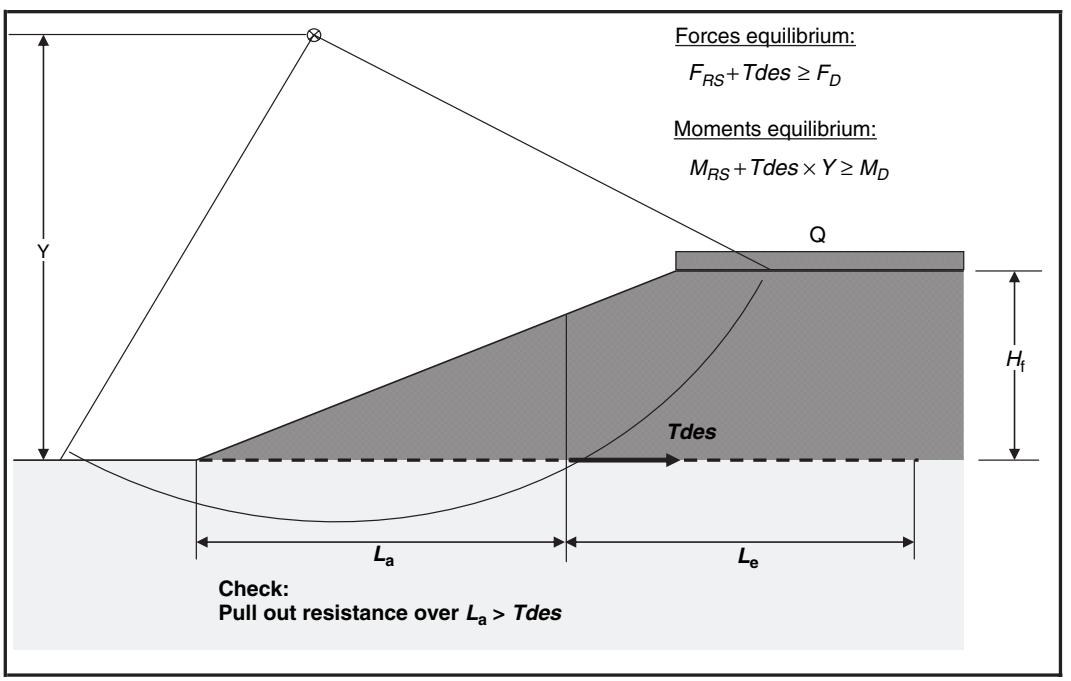


Figure 73.12 Basal reinforcement – deep-seated failure. Q = surcharge loading, H_f = height of embankment, L_a = length of reinforcement within active zone, L_e = length of reinforcement within resistant zone

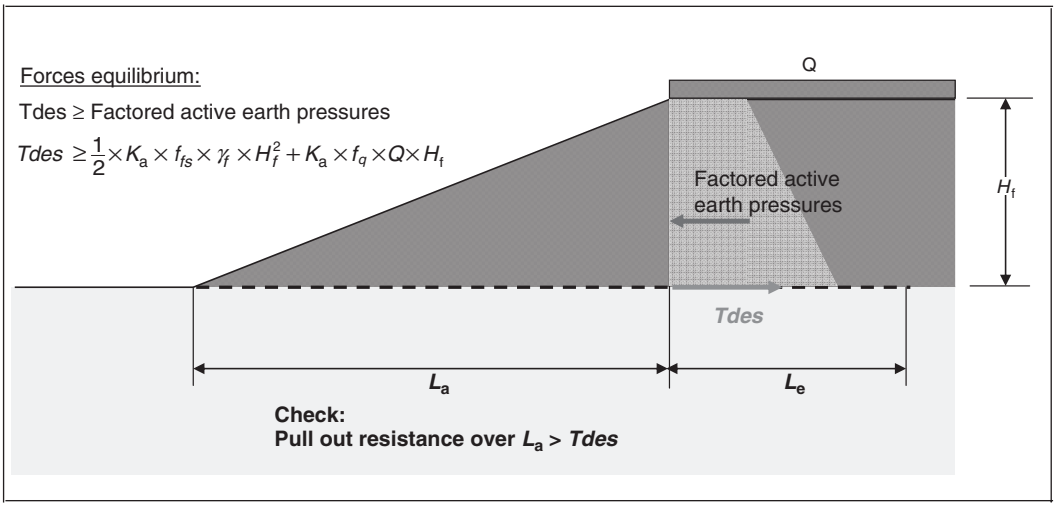


Figure 73.13 Basal reinforcement – lateral sliding. K_a = active earth pressure coefficient, γ_f = fill unit weight

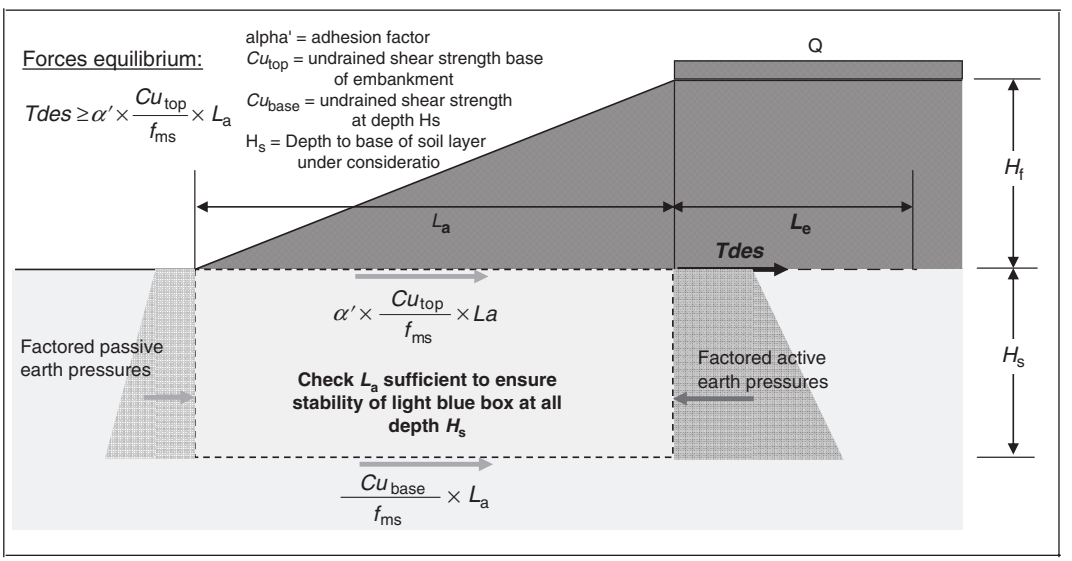


Figure 73.14 Basal reinforcement – extrusion. α' = adhesion factor, C_u top = undrained shear strength base of embankment, C_u base = undrained shear strength at depth H_s, H_s = depth to base of soil layer under consideration

from the embankment fill and any surcharge which is at a maximum at the top edge of the embankment slope. Again, it is also necessary to ensure that the length of reinforcement within the failure surface mechanism is sufficient to prevent pull-out (**Figure 73.13**).

73.7.2.3 Extrusion

Extrusion assumes the outward lateral displacement of a rectangular block of soft soil under the embankment fill. The reinforcement provided shall have a sufficient design capacity Tdes (against rupture and pull-out beyond the failure mechanism) to resist the tensile load generated by the outward foundation shear. In addition, it is also necessary to ensure that at any depth Hs within the reinforced soil, the rectangular block of soil is in equilibrium. The restoring forces are the adhesion along the base of the soil block, the adhesion between the reinforcement and the soil at the top of the block and the passive earth pressures on the outward side of the block (**Figure 73.14**).

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73.8.1 Further reading

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It is recommended this chapter is read in conjunction with

- Chapter 23 *Slope stability*
- Chapter 69 *Earthworks design principles*
- Chapter 72 *Slope stabilisation methods*
- Chapter 86 *Soil reinforcement construction*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 74
Design of soil nails

Martin J. Whitbread Atkins, Epsom, UK

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Soil nails may be used to stabilise cuttings and embankments. The design of soil nails has to take account of the ground conditions and groundwater regime to allow the determination of the global and local stability of the strengthened slope. Various slope facings may be adopted for soil nailed slopes, ranging from a green finish to shotcrete

74.1 Introduction

Soil nailing is a method of ground reinforcement comprising the installation of steel or polymer bars sub-horizontally into a slope or wall. The nail head comprises a load plate, which is tightened against the slope facing. Soil nails are generally used to:

- stabilise new cuttings that are steeper than the soil properties allow;
- remediate existing embankments or cuttings that have failed or are under distress;
- strengthen retaining walls that have suffered excessive displacement;
- increase the stability of retaining walls or other structures that are to be subjected to greater loads than envisaged in the original design.

74.2 History and development of soil nailing techniques

Soil nailing was developed from similar techniques including rock bolting, multi-anchorage systems and reinforced soil. The first soil nailed wall was constructed in France in 1972. Research programmes on soil nailing were initially carried out in Germany and then in France through the Clouterre Project (Clouterre, 1991). The Clouterre Project, which published the Clouterre Recommendations (Clouterre, 1991), reported on large-scale experiments and their findings have been adopted in the design of soil nailing in numerous countries. The observations of the first full-scale experiment are shown in **Figure 74.1**. In this experiment, a low factor of safety was adopted, together with a method of saturating the soil behind the wall, so that the wall failed. Total failure of the structure did not occur due to the presence of the embedded shotcrete wall facing.

The third experimental test was carried out to demonstrate the effect of having nails that were too short. The results of this showed that failure of the wall was due to lack of adherence (see **Figure 74.2**).

In the UK, university research and research by the Transport Research Laboratory has led to increasing confidence in soil nailing as a technique and acceptance of soil nailing for major government infrastructure projects.

74.3 Suitability of ground conditions for soil nailing

A wide range of ground conditions are suitable for soil nailing including most clays (with the exception of soft clays), sands and mixed materials. However, appropriate face treatment will be required, for example to reduce the potential for surface failure due to surface run-off causing wash-out of unconsolidated materials. Problems can arise where there are elevated groundwater levels including perched water tables and spring lines. In these cases the design should pay close attention to the

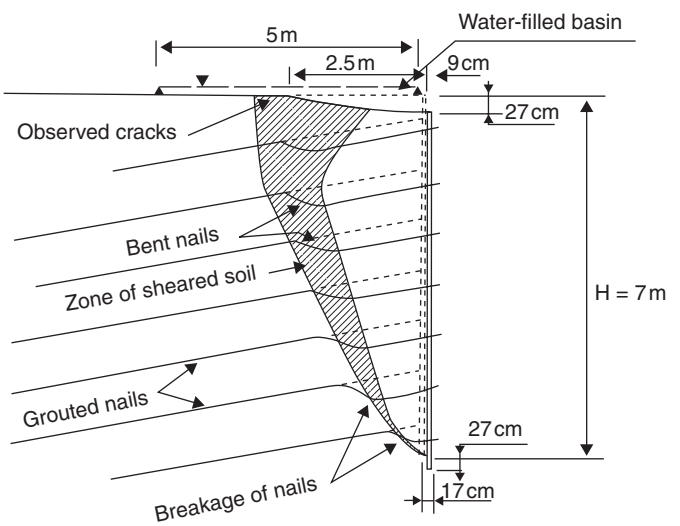


Figure 74.1 Post-failure observations of the first full-scale experimental soil nailed wall
Reproduced from Clouterre, 1991 (English translation)

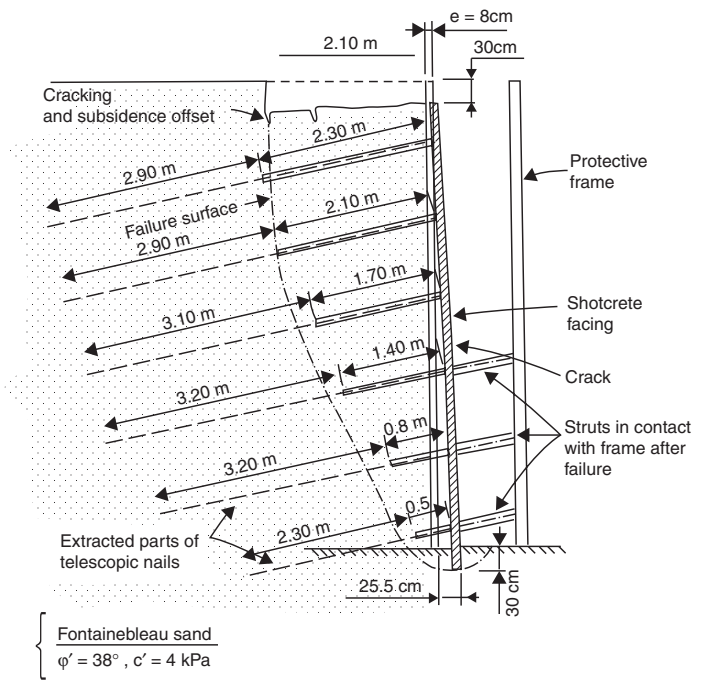


Figure 74.2 Post-failure observations of the third full-scale experiment
Reproduced from Clouterre, 1991 (English translation)

incorporation of temporary and permanent drainage including slope drainage, toe drains and crest drains.

74.4 Types of soil nails

Soil nails are installed in cohesive or granular soils or in weak rocks using one of the following methods:

- Drilling of a sub-horizontal hole followed by the insertion of a steel or polymer bar and grouting of the hole using a tremie tube. Spacers are placed around the bars to ensure that the bar is centrally placed within the hole. This method is the most economic providing the ground conditions are suitable, e.g. within firm to stiff cohesive soils where there is no risk of collapse of the hole. However, care must be taken with this method when there is a delay between the drilling and grouting of a hole resulting in the undetected collapse and blockage of the hole leading to only partial grouting of the hole.
- Simultaneous drilling and grouting whereby a hollow high-yield steel bar is used as the drill rod with a sacrificial bit to form the hole by percussive drilling. During the drilling process the grout is passed down the annulus of the bar and the hole drilled to the required depth. Grouting continues until clean grout emerges at the surface. This method is very rapid and can be used in most types of soil. The grout supports the bore at all times, preventing collapse of the hole. The disadvantage of this method is that the spacers used around the bar cannot be too large otherwise the return of the grout to the surface during drilling may be restricted and, hence, the bar is unlikely to be absolutely central in the hole.
- Driving a soil nail into a slope using ballistic methods. This method is very rare and, therefore, is not discussed further.

74.5 Behaviour of soil nails

Soil nails are typically installed at 10° to 20° to the horizontal and are able to work in tension and shear. However, at the normal installation angles stated above, the shear component is negligible and usually neglected and, therefore, nails are normally designed to act in tension only. Hence, nails are designed to penetrate through the active zone of movement and sufficiently beyond any potential slip surface into the resistant soil zone. A suitable nail arrangement is adopted to form a stabilised reinforced block. Loads on soil nails can only be induced by movement of the slope or retaining wall, with the section of the nail furthest from the surface being the first to be stressed by the movement.

For steeper slopes a shotcrete facing incorporating one or two layers of mesh may be adopted.

74.6 Design

A geotechnical desk study would be carried out prior to any design to discover any existing ground data and also to inform the design of the ground investigation where insufficient information is available.

Following the ground investigation and any monitoring (of groundwater and movement) the geotechnical parameters are determined for each soil layer. Where a slope has failed or where monitoring by inclinometers, etc. has identified a failure surface, back analysis of the slope using a slope stability program may be used to estimate the soil parameters and groundwater condition that would produce an unfactored global factor of safety against instability of just less than 1.0, i.e. an unstable condition.

The design of soil nails is most frequently carried out using limit state design and a slope stability program, e.g. Slope/W (Geoslope International) or Talren (Terrasol) and by ensuring that the internal and external factors are checked. Alternatively, HA 68/94 (Highways Agency, 1994) can be used, which describes the two-part wedge method and may be appropriate under certain conditions. An example of a soil nail analysis is shown in **Figure 74.3**.

An initial soil nail array is chosen, i.e. the horizontal and vertical spacing and the length of the nails. For the initial array it is recommended that for slopes of 45° or less, an array of nails of 1.0 m vertical spacing and 1.5 m horizontal spacing is adopted. The length of the nails should be 1.0 to 1.5 times the height of the slope depending on the ground conditions. For slopes of more than 45° to 70° an initial array of nails of 0.5 m vertical spacing and 1.0 m horizontal spacing is recommended with similar nail lengths as above. It is recommended that the maximum depth of the top row of nails from the crest of the slope should not exceed 0.5 m to ensure that the potential for ravelling of the crest is reduced. Also the maximum height from the lowest row to the toe of the slope should also not exceed 0.5 m. A typical initial array for a nailed slope is shown in **Figure 74.4** and a typical nail head is shown in **Figure 74.5**.

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The soil nail design is further refined by trial and error using the slope stability program with the geotechnical parameters and groundwater regime determined previously so that the most economic design is found. Initially it is recommended that the design is carried out to produce an overall factor of safety of 1.3. Once this initial design has been completed the analysis should be repeated using the specified partial factors on the soil parameters and external loads required by Eurocode 7 *Geotechnical Design* (British Standards Institution, 2004) or BS8006:1995 (British Standards Institution, 1995). Note that BS8006:1995 is being updated and the soil nailing part is to be issued as BS8006-2 *Code of Practice for Strengthened/Reinforced Soils and Other Fills. Part 2: Soil Nailing*. Further design advice can be found in *Soil Nailing – Best Practice Guidance, CIRIA Report C637* (CIRIA, 2005).

During the design process, due care should be taken of the site boundaries. If the soil nails pass beneath the site boundary into neighbouring properties, a wayleave will be required. However, it may be feasible to prevent the nails from passing beneath the ownership boundary by reducing the length of the upper nails, providing an adequate factor of safety is obtained, particularly in the upper section (see **Figure 74.6**).

Once the soil nail arrangement has been determined, the design of the load plate and surfacing is carried out. The load plates are used to apply a relatively low load to the slope surface and to hold the slope facing in place. The slope facing is required to stabilise the surface of the slope between the nails and to reduce the potential for any surface movement.

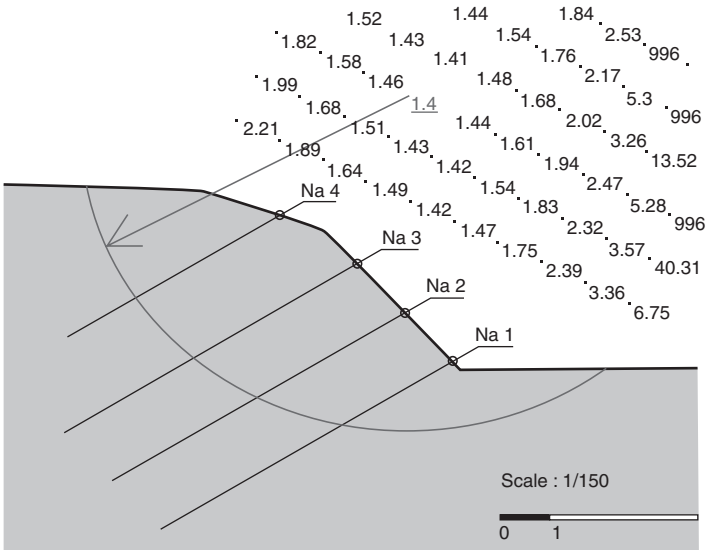


Figure 74.3 Typical soil nail analysis

74.6.1 Load plates

The design of the load plates should be such that bearing capacity failure of the slope surface does not occur during any future loading of the soil nails. There are a number of methods available to design the load plates, including that contained in Appendix E of HA 68/94 (Highways Agency, 1994). However, this method tends to overestimate the size of the plate as it does not take into account the slope facing itself. The section of the nail furthest from the slope surface tends to experience the highest load and the nail head itself may not experience the full load except in the unlikely event of approaching slope failure. The load transferred to the nail head generally increases as the slope angle increases, and the use of larger plates for shallower slopes, as in the Highways Agency guide, is likely to be conservative.

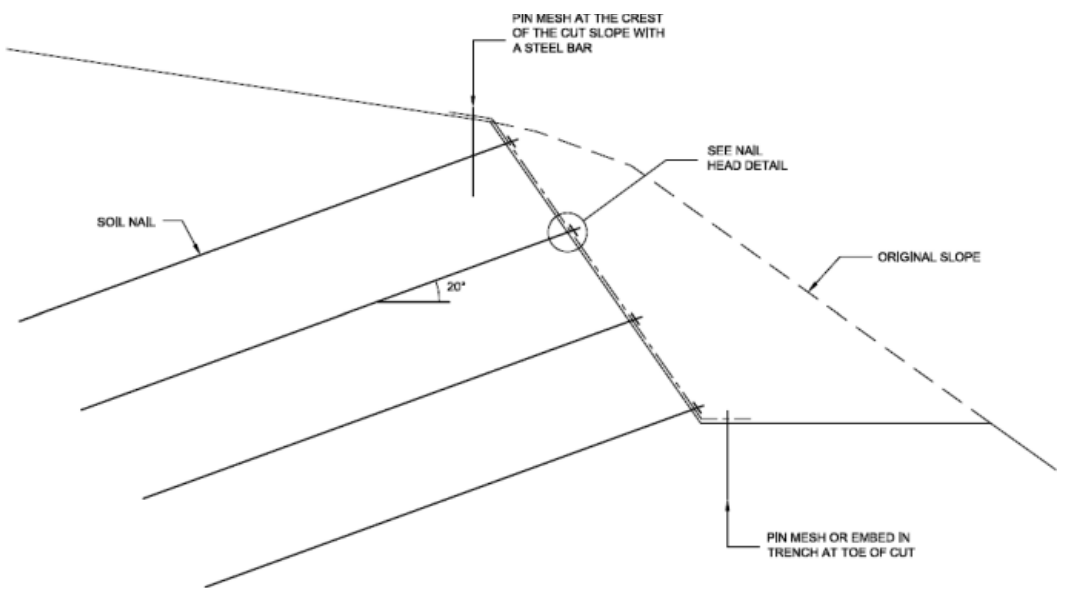


Figure 74.4 Typical soil nail array

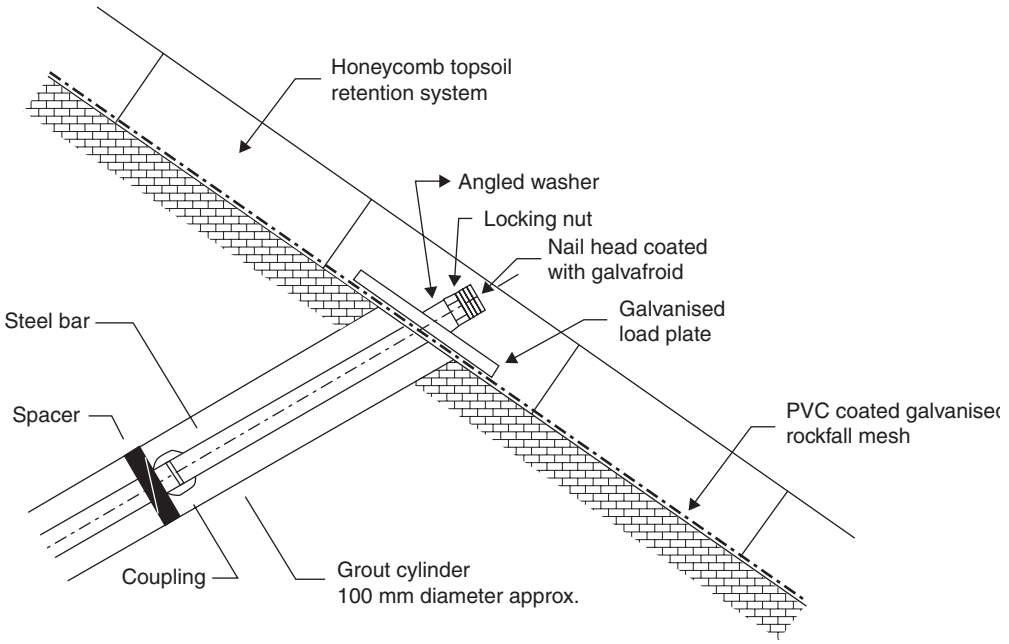


Figure 74.5 Typical soil nail head

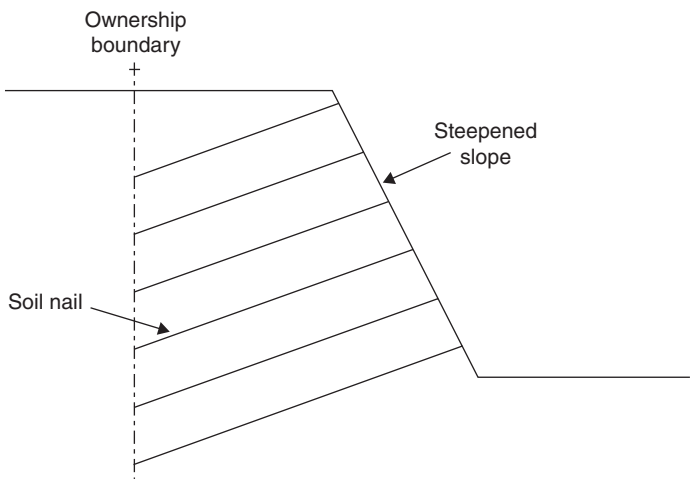


Figure 74.6 Design of soil nail array to prevent requirement for wayleave

Load plates may be mounted on the surface of the slope or cut into the slope. Generally plates of dimensions up to 300 × 300 mm are acceptable. The use of larger plates poses problems for installation due to the weight of the plates and there may be manual-handling issues.

74.6.2 Slope facing

There are no real guidelines for the design of slope facings; however, the following gives some practical advice.

For slope angles of ≤ 45°, a flexible form of surfacing can normally be adopted. This may comprise a rockfall mesh (normally PVC coated) or a geogrid, which is rolled down the slope over the nails. The nail load plates are then hand tightened against the mesh or geogrid. The slope may be covered in top soil for gradients up to 1:2 (vertical:horizontal) or hydroseeded for steeper slopes. However, where the slope is too steep and topsoil cannot be applied directly to the slope, and to prevent the potential hazard of the nail heads protruding from the slope, it is recommended that a cellular topsoil retention system is included in the design. This consists of a cellular geogrid, which is pinned to the slope. The cells are filled with seeded topsoil of sufficient depth to bury the nail heads.

For slope angles of more than 45 to 70°, a semi-rigid form of facing is generally acceptable. This type of facing may consist of two or more layers of rigid mesh or soil panels filled with topsoil or granular material, which are held on the slope by the nail heads; in the former the nail heads protrude from the slope and in the latter are concealed by the infill to the soil panels.

For slopes steeper than 70°, hard facings are generally adopted, which generally comprise sprayed concrete (shotcrete) incorporating one or two layers of mesh with the lowest layer of mesh held on the slope by the soil nail heads.

74.7 Construction

The construction of soil nailed slopes and walls are covered in Chapter 88 *Soil nailing construction*. However, generally these structures are constructed top down. Excavation is initially carried out over the top 1 m to 1.5 m to form a platform from which the nails can be installed. Alternatively the nails can be installed using long-reach plant from either the crest or toe of the slope. Once the first row of nails has been installed and the

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grout has reached its required strength, excavation of the next 1 m depth of material is carried out and the second row of nails is installed. The process is then repeated until the whole excavation is complete. The required facing is then placed and the load plates tightened.

74.8 Drainage

Drainage is likely to be required behind any shotcrete facing and may include sub-horizontal drains inclined upwards (to prevent loss of material) using the same plant used to install the soil nails. These inclined drains would lead into a drainage system behind the shotcrete and eventually into a toe drain. Any drainage system concealed behind a hard facing must include a means of maintenance, e.g. access points where the inclined drains could be rodded.

For other forms of facing, drainage comprising gravel-filled trenches down the slope leading to toe drains may be utilised.

74.9 Corrosion of soil nails

The extent of corrosion protection required for soil nails depends on a number of factors including the specified design life, the aggressivity of the ground, the gradient of the nailed slope, the soil nail installation method and the thickness of the steel. The required design life for most soil nailed slopes is 60 years; however, where soil nails are incorporated into other structures, such as retaining walls etc., a design life of 120 years may be required. Guidance on the aggressivity of the ground is given in TRL Report 380 (Murray, 1993), which also recommends thicknesses for sacrificial steel that may be incorporated as a method of corrosion protection. This allows the diameter of the steel to be designed so as to give sufficient corrosion protection over the design life and also to provide sufficient structural strength.

For aggressive ground conditions, a further layer of protection may be provided, such as steel bars that are fully galvanised or epoxy coated by the manufacturer. Alternatively, a plastic duct may be placed within the upper part of the grout column around the steel bar, as the corrosion potential is likely to be greatest in the upper part of the installation, i.e. within the zone that is most likely to be subjected to seasonal variations in the groundwater level.

74.10 Testing soil nails

Soil nails are tested by sacrificial nail testing or production nail testing, as recommended by BS EN 14490:2010 (British Standards Institution, 2010).

Sacrificial nail testing is carried out in advance of the contract nails to prove the bond strength assumed in the design. Sacrificial testing involves a pull-out test in which the displacement of the nail head is measured over two load cycles. During sacrificial testing the nail may be taken to failure. Generally the top half of the nail is debonded to prevent any beneficial effect during testing. A minimum of three sacrificial tests on nails are normally carried out.

Production nail testing is dependent on the type of the soil nailed structure. For simple or conventional structures of Geotechnical Category 1 or 2 in accordance with Eurocode 7, BS EN 1997–1:2004 (British Standards Institution, 2010) and HD 22/08 *Managing Geotechnical Risk* (Highways Agency, 2008) it is recommended that 1.5% of the number of working nails is tested (or a minimum of three nails). For a more complex Category 3 structure, the guidance is that 2.5% of the number of working nails is tested (or a minimum of five nails).

74.11 Maintenance of soil nailed structures

Soil nailed structures generally require little maintenance. However, for more complex structures or where the consequences of failure of the structure would be catastrophic, it is recommended that sacrificial nails are installed adjacent to the structure, and these can be tested at intervals throughout the required design life. Where nail heads are visible, monitoring by precise surveying may be carried out. Other monitoring or observation that could be carried out includes visual inspection of the slope facing to check for any signs of bulging, inspection of any drainage installed that is a requirement of the design and observation of abnormal vegetation growth.

74.12 References

British Standards Institution (1995). *Code of Practice for Strengthened/Reinforced Soils and Other Fills*. London: BSI, BS 8006:1995.

British Standards Institution (2004). *Eurocode 7 Geotechnical design.* London: BSI, BS EN 1997–1:2004.

British Standards Institution (2010). *Execution of Special Geotechnical Works – Soil Nailing*. London: BSI, BS EN 14490:2010.

CIRIA (2005). *Soil Nailing Best Practice Guide. Report C637*. London: CIRIA.

Clouterre (1991). *French National Research Project Clouterre – Recommendations Clouterre* (English translation). Federal Highway Administration FHWA-SA-93–026, Washington, USA, 1993.

Highways Agency (1994). *HA 68/94 Design Methods for the Reinforcement of Highway Slopes by Reinforced Soil and Soil Nailing Techniques*. London: HMSO.

Highways Agency (2008). *HD 22/08 Managing Geotechnical Risk.* London: HMSO.

Murray, R. T. (1993). *The development of specifications for soil nailing. Research Report 380*. Crowthorne: Transport Research Laboratory.

It is recommended this chapter is read in conjunction with

- Chapter 69 *Earthworks design principles*
- Chapter 72 *Slope stabilisation methods*
- Chapter 88 *Soil nailing construction*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 75

Earthworks material specification, compaction and control

Philip G. Dumelow Balfour Beatty, London, UK

This chapter describes how an effective earthworks specification can be produced for a project. It will define permitted material types, placement methods and appropriate means of control. The objective is earthworks which are safe during construction and use; durable and economically viable. Sustainable construction is a significant consideration.

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75.1 The earthworks specification

75.1.1 Necessity and purpose

It is essential that a materials specification is produced for any earthworks contract, the core requirements being defined by the client. Whenever possible, all parties in the contract should be involved in preparing and developing the specification at the earliest stage. This will include the contractor and the designer, whether appointed by the client or the contractor.

The nature of certain contracts encourages such a collaborative approach:

- Design and Build (D&B)
- Design, Build, Fund and Operate (DBFO)
- Early Contractor Involvement (ECI).

Client, designer and contractor collaboration utilises the broadest possible experience and knowledge of materials, geotechnics and construction methods to guide how the specification is written. The purpose is to deliver a project built to best practice, economically and safely.

The completed specification will define:

- the client's core requirements for the works, e.g. levels, tolerances;
- methodologies for carrying out the works, e.g. excavating cuts, filling embankments and preparing formation;
- materials suitable for constructing the works, e.g. classes of material for specific purposes;
- material factors for physical, mechanical and chemical suitability, e.g. material parameters and limits for the required performance.

75.1.2 Current UK specifications

The earthworks specifications in widest use are produced by the Highways Agency (HA):

1. **Manual of Contract documents for Highway Works (MCHW)**
   - **Volume 1: Specification for Highway Works (SHW)**
   - **Series 600: Earthworks (Figure 75.1)**

Although this has been produced by the HA for their network of trunk roads and motorways, it is used by other clients. These include those in other infrastructures and disciplines. Use of the SHW is either in full or as the basis from which a specification is developed. Throughout this chapter, SHW and related HA publications will be used for reference; engineers working in other infrastructure disciplines must ascertain what specification is applicable to them.

The SHW has been developed from the Ministry of Transport 'Specification for Road and Bridge Works', in use from the 1960s to 1983, when it was replaced by the SHW. It is freely available, in all its parts, Series 000 to 3000 (generally incrementing by 100 for each series) plus appendices, as a download from the HA's web pages:

www.standardsforhighways.co.uk/

These web pages only hold the current documents.

The SHW refers to British Standards and the European Standards, BS EN, which replace them. These are not numbered the same as the superseded BS and the requirements are often very different. Prior to commencing any earthworks project and planning any of the works, an engineer must know which specifications and standards apply to the contract. These must be acquired and the requirements understood. Not to do so is inviting failure.

In addition, SHW references publications other than BS and BS EN; these are listed in Appendix F of SHW: 'Publications Referred to in the Specification'. It is the responsibility of the engineer to be in possession of the relevant publications.

Contract specific requirements are defined in the numbered appendices prepared by the specification's compilers.

MANUAL OF CONTRACT DOCUMENTS FOR HIGHWAY WORKS
VOLUME 1 SPECIFICATION FOR HIGHWAY WORKS

SERIES 600
EARTHWORKS

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#601	Classification, Definitions and Uses of Earthworks Materials	2
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604	Excavation for Foundations	6
605	Special Requirements for Class 3 Material	7
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616	Preparation and Surface Treatment of Formation	18
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622	Earthworks for Reinforced Soil and Anchored Earth Structures	20
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634	(11/05) Determination of Intact Lump Dry Density (IDD) of Chalk	25
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637	Determination of Resistivity (r_s) to Assess Corrosivity of Soil, Rock or Earthworks Materials	26
638	Determination of Redox Potential (E_h) to Assess Corrosivity of Earthworks Materials for Reinforced Soil and Anchored Earth Structures	27
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642	Determination of the Constrained Soil Modulus (M*) of Earthworks Materials for Corrugated Steel Buried Structures	28
643	(05/01) Lime and Cement Stabilisation to Form Capping	29
644	(11/03) Determination of Sulfate Content	30
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NATIONAL ALTERATIONS OF THE OVERSEEING ORGANISATIONS OF SCOTLAND, WALES AND NORTHERN IRELAND

Scotland

Clause	Title	Page
601TS	(11/06) Classification, Definitions and Uses of Earthworks Materials	S1
632TS	(11/06) Determination of Moisture Condition Value (MCV) of Earthworks Materials in Scotland	S3

Northern Ireland

Clause	Title	Page
601NI	Classification, Definitions and Uses of Earthworks Materials	N1
607NI	Explosives and Blasting for Excavation	N3

denotes a Clause or Sample Appendix which has a substitute National Clause or Sample Appendix for one or more of the Overseeing Organisations of Scotland, Wales or Northern Ireland.

Figure 75.1 Highways Agency SHW 600 Series
Reproduced from Highways Agency, *Manual of Contract Documents for Highway Works*, Volume 1: *Specification for Highways Work*, Series 600. © Crown Copyright 2006

MANUAL OF CONTRACT DOCUMENTS FOR HIGHWAY WORKS
VOLUME 2 NOTES FOR GUIDANCE ON THE SPECIFICATION FOR HIGHWAY WORKS

SERIES NG 600
EARTHWORKS

Contents

Clause	Title	Page
NG 600	Introduction	2
NG 601	Classification, Definition and Uses of Earthworks Materials and Table 6/1: Acceptable Earthworks Materials: Classification and Compaction Requirements	3
NG 602	General Requirements	5
NG 603	Forming of Cuttings and Cutting Slopes	5
NG 604	Excavation for Foundations	5
NG 605	Special Requirements for Class 3 Material	5
NG 606	Watercourses	5
NG 607	Explosives and Blasting for Excavation	6
NG 608	Construction of Fills	6
NG 609	Geotextiles Used to Separate Earthworks Materials	6
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NG 619	Earthwork Environmental Bunds	8
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NG 625	Crib Walling	9
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NATIONAL ALTERATIONS OF THE OVERSEEING ORGANISATIONS OF SCOTLAND, WALES AND NORTHERN IRELAND

Clause	Title	Page
Scotland		
NG 601SE	(11/05) Classification, Definitions and Uses of Earthworks Materials	S1
NG 632SE	(05/01) Determination of Moisture Condition Value (MCV) of Earthworks Materials	S1

denotes a Clause or Sample Appendix which has a substitute National Clause or Sample Appendix for one or more of the Overseeing Organisations of Scotland, Wales or Northern Ireland.

Figure 75.2 Highways Agency NG 600 Series

Reproduced from Highways Agency, *Manual of Contract Documents for Highway Works*, Volume 2: *Notes for Guidance on the Specification for Highway Works*, Series NG 600. © Crown Copyright 2005

Accompanying and supporting the SHW are the HA publications:

2. **Manual of Contract documents for Highway Works (MCHW)**
   - **Volume 2: Notes for Guidance on the Specifications for Highway Works (NG)**
   - **Series 600: Earthworks (Figure 75.2)**

The clauses mirror those in SHW. The purpose of the NG is to assist in preparation of the contract; this includes commenting on the technical background underlying the SHW requirements and listing the appendices which must be produced for the contract (cf. 'Scope of the specification' below). Engineers commencing an earthworks contract must read the NG in conjunction with SHW for a full understanding of the specification.

3. **Design Manual for Roads and Bridges (DMRB)**
   - **Volume 4: Geotechnics and Drainage**
   - **Section 1: Earthworks**
   - **Part 1: HA 44/91 Design and Preparation of Contract Documents (Figure 75.3)**

This document gives guidance specifically directed to preparing the earthworks specification. This includes:

- cross-referencing the numbered appendices with the SHW Series 600 clauses;
- use of materials and construction methods;
- testing for acceptability.

Earthworks contracts in Scotland and Northern Ireland are subject to alternative clauses. These are included at the end of each series as 'National alterations of the overseeing organisation of Scotland/Northern Ireland'.

4. **Manual of Contract documents for Highway Works (MCHW)**
   - **Volume 1: Specification for Highway Works (SHW)**
   - **Series 100: Preliminaries**
   - **Series 500: Drainage and service ducts**

The engineer must be aware that other series within the SHW are related to earthworks activities, e.g. Series 500 specifies the requirements for drainage material.

The most significant of these is Series 100, clause 105. This requires the production of Appendix 1/5, which includes Table 1/5, defining the frequency at which all classes and types of material used in the works are to be sampled and tested. This includes all classes of earthworks materials.

75.1.3 Scope of the specification

The specification compilers will prepare the numbered appendices defining earthworks requirements for the contract. Sample appendices for an earthworks specification are listed in NG 600 Series (see **Table 75.1**). These inform the compiler what should be included in each appendix.

The specification compiler must also prepare an Appendix 1/5 and Table 1/5. Advice on test frequency is provided by Table 1/1 in SHW NG. Similarly to the overall specification, this compilation team should include client, designer and contractor to include the broadest knowledge and experience.

Some of the appendices are associated with tables with the same numbering. For site control of earthworks, the most important of these are:

- Table 6/1: Materials permitted in the works, acceptability limits, compaction method.
- Table 6/2: Grading limits for each material class.
- Table 6/4: Method compaction for earthworks materials: plant and methods.
- Table 1/5: Testing to be carried out by the contractor.

Extracts from a completed Table 6/1 are shown in **Figure 75.4(a)** and **4(b)**, and from Table 1/5 in **Figure 75.4(c)**.

The main material classes in Table 6/1 are shown in **Table 75.2**.

Each class may be sub-divided by a lettered suffix to describe differing material types (wet, dry, coarse, fine) or end use, especially for Class 6 materials. Class U1 material may be reverted to an acceptable class by treatment to modify its physical characteristics.

Class U2 is unacceptable due to chemical contamination. This class may also be sub-divided into material which can be treated to be acceptable and material which, either by the nature or concentration of contamination, must be removed from site to an appropriately licensed facility.

75.1.4 Key requirements for an effective specification

To produce the specification, the compilers must refer to and draw data and information from different sources. These will include:

- The client's core requirements:

  for an SHW type, this is SHW & NG 600 Series and DMRB, especially HA44/91.

- Ground investigations (GI) carried out for the project:

  both geotechnical assessment and acceptability classification of the material.

- Reports by other bodies:

  generally, these are by the Transport Research Laboratory (TRL) or the Building Research Establishment (BRE) for specific material types and conditions.

- Personal knowledge and experience:

  especially relevant if local materials merit specific consideration, including local knowledge of recycling and secondary materials sources.

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DESIGN MANUAL FOR ROADS AND BRIDGES

VOLUME 4	GEOTECHNICS AND DRAINAGE
SECTION 1	EARTHWORKS

PART 1

HA 44/91

DESIGN AND PREPARATION OF CONTRACT DOCUMENTS

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Annex A

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April 1995

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Figure 75.3 Highways Agency, Design Manual for Roads and Bridges, HA 44/91
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The complete earthworks specification, whether using the format of SHW or any other, must deliver:

- clear, unambiguous statements on how the earthworks will be carried out;
- the means by which the works can be carried out safely;
- a durable end product to the client's satisfaction;
- characteristics and limits for the acceptability of materials, without overdesigning;
- methodologies for dealing effectively with specific materials and conditions;
- the most sustainable earthworks solution.

Correlation testing may be carried out on materials, relating fundamental properties with results from the site control tests.

App No.	Appendix title
6/1	Requirements for acceptability and testing, etc. of earthworks materials
6/2	Requirements for dealing with Class U1B and Class U2 unacceptable materials
6/3	Requirements for excavation, deposition, compaction (other than dynamic compaction)
6/4	Requirements for Class 3 material
6/5	Geotextiles used to separate earthworks materials
6/6	Fill to structures and fill above structural foundations
6/7	Sub-formation and capping and preparation and surface treatment of formation
6/8	Topsoiling
6/9	Earthwork environmental bunds, landscape areas, strengthened embankments
6/10	Ground anchorages, crib walling and gabions
6/11	Swallow holes and other naturally occurring cavities and disused mine workings
6/12	Instrumentation and monitoring
6/13	Ground improvement
6/14	Limiting values for pollution of controlled waters
6/15	Limiting values for harm to human health and the environment
1/5	Testing to be carried out by the contractor

Table 75.1 Numbered appendices required for an earthworks specification

Class	General material description	Typical use
1	Granular material	General fill
2	Cohesive material	General fill
3	Chalk	General fill
4	Various materials	Landscape fill
5	Topsoil	Topsoiling
6	Selected granular material	Specific uses, e.g. starter layer, backfill, capping
7	Selected cohesive materials	Specific uses, e.g. backfill, material for stabilisation
8	Class 1, 2 or 3 material	Lower trench fill
9	Lime or cement stabilised granular or cohesive material	Capping (synonymous with 'foundation' in pavement design)
U1	Any type of material	Materials physically unacceptable for the works
U2	Any type of material	Materials chemically unacceptable for the works

Table 75.2 The main material classes in Table 6/1 (Materials permitted in the works, acceptability limits, compaction method) of the Specification for Highway Works

From these derive the limits applicable to the test used for quality control of the project.

Sustainable earthworks must be a key consideration. This can be addressed by the specification compilers aiming to:

- minimise waste by balancing the cut with the embankment/landscape fill volumes;
- maximise the re-use of the excavated materials to minimise import;
- create opportunity for the optimum use of site-won material;
- utilise locally available recycled and secondary materials;
- improve poor quality materials by treatment (e.g. modification or geosynthetics);
- involve the contractor in additional GI and materials trials.

Additionally, specification compliers must ensure, when producing Table 6/1 and 1/5, that artificial constraints are not imposed. This can occur when:

- the contractor is not involved in specification compilation;
- the contractor is required to design part of the permanent works.

The specification must not unnecessarily restrict the materials allowed. The contractor may have experience of the successful use of alternatives which deliver the same performance. These may be more sustainable or economic or the methodology may have a programme benefit, yet their use is not permitted.

Amending the specification to permit materials not included in the specification takes time, resources and cost. Furthermore, there may be contractual implications. A 'Departure from Standards' may need to be formally requested through the design protocol. Engineers must be aware that adoption of a Departure may affect the ownership of risk associated with the material or method concerned.

75.1.5 Specifying test frequency

As referred to by item 8 in 'Control of earthworks' below, SHW NG advises on frequency of testing. For many materials – including most earthworks materials – the test frequency in NG Table 1/1 – Typical testing details – is only advisory. The reason for this is defined by the notes in the key at the foot of the table:

> Where materials are known to be marginal or if initial test results show them to be such, the frequency of testing should be increased. Conversely where material properties are consistently in excess of specified minimum requirements or well below specified maximum limits, then the frequency of testing should be reduced.

As discussed in 'Compliance testing of earthworks' below, it is essential that the specification compilers understand the:

- likelihood of the occurrence of non-conformance with specification;
- implications of non-conformance on the structural integrity of the works.

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APPENDIX 6/1 REQUIREMENTS FOR ACCEPTABILTY AND TESTING ETC. OF EARTHWORKS MATERIALS

(a)

CLASS	GENERAL MATERIAL DESCRIPTION	TYPICAL USE	PERMITTED CONSTITUENTS (All subject to requirements of Clause 601 (SHW) and Appendix 6/1)	MATERIAL PROPERTIES REQUIRED FOR ACCEPTABILITY (in addition to requirements on use of fill materials in Clause 601 (SHW) and testing in Clause 631 (SHW))				COMPACTION REQUIREMENTS IN CLAUSE 612 (SHW)
				PROPERTY (See exception in previous column)	DEFINED AND TESTED IN ACCORDANCE WITH:-	ACCEPTABLE LIMITS WITHIN:		
						Lower	Upper	
2A	Wet Cohesive material	General fill	Any material, or combination of materials, other than chalk For details of properties (iv) and (v) in the next column, refer to Notes 12 and 13.	(i) grading	BS 1377 Part 2	Tab 6/2	Tab 6/2	Table 6/4 Method 1 except for materials with liquid limit greater than 50, determined by BS1377: Part 2, only deadweight tamping or vibratory tamping rollers or grid rollers shall be used.
				(ii) plastic limit (PL)	BS 1377 Part 2	-	-	
				(iii) mc	BS 1377 Part 2	-	-	
				(iv) MCV	Clause 632	8	12	
				(v) Undrained shear strength	Clause 633	50 kN/m^2	100 kN/m^2	
2B	Dry Cohesive material	General Fill	Any material, or combination of materials, other than chalk. For details of properties (iv) and (v) in the next column, refer to Notes 12 and 13. For the upper limit of property (iv) refer to note 14.	(i) grading	BS 1377 Part 2	Tab 6/2	Tab 6/2	Table 6/4 Method 2
				(ii) plastic limit (PL)	BS 1377 Part 2	-	-	
				(iii) mc	BS 1377 Part 2	-	-	
				(iv) MCV	Clause 632	12	18	
				(v) Undrained shear strength	Clause 633	100 kN/m^2	-	

Figure 75.4 Extracts from (a) a typical Table 6/1 for cohesive material compiled for a specific project; (b) a typical Table 6/1 for granular material compiled for a specific project; (c) a table in a typical Appendix 1/5 compiled for a specific project

Extracts from Highways Agency, *Manual of Contract Documents for Highway Works*, Volume 1: *Specification for Highways Work*, Series 600 © Crown Copyright 2009–10.

(b)

CLASS	GENERAL MATERIAL DESCRIPTION	TYPICAL USE	PERMITTED CONSTITUENTS (All subject to requirements of Clause 601 (SHW) and Appendix 6/1)	MATERIAL PROPERTIES REQUIRED FOR ACCEPTABILITY (in addition to requirements on use of fill materials in Clause 601 (SHW) and testing in Clause 631 (SHW))				COMPACTION REQUIREMENTS IN CLAUSE 612 (SHW)
				PROPERTY (See exception in previous column)	DEFINED AND TESTED IN ACCORDANCE WITH:-	ACCEPTABLE LIMITS WITHIN:		
						Lower	Upper	
6N	Selected well graded granular material	Fill to structures	Natural gravel, natural sand, crushed gravel, crushed rock, crushed concrete, slag, well-burnt colliery spoil or any combination thereof. None of these constituents shall include any argillaceous rock. Recycled aggregate except recycled asphalt	(i) grading	BS 1377 Part 2 (On site)	Tab 6/2	Tab 6/2	End product 95% of maximum dry density of BS 1377: Part 4 (vibrating hammer method)
					BS EN 933-2 (Off site)	Tab 6/5	Tab 6/5	
				(ii) uniformity coefficient	See Note 5	10	-	
				(iii) Los Angeles coefficient	Clause 635	-	40	
				(iv) Undrained shear parameters (c and φ)	Clause 633	-	-	
				(v) effective angle of internal friction (φ') and effective cohesion (c')	Clause 636	35?	45?	
				(vi) permeability	Clause 640	$5x10^{-5}$ m/s	-	
				(vii) mc	BS 1377: Part 2	OMC - 2.5%	OMC +1%	
				(viii) MCV	Clause 632	-	-	

Figure 75.4 (Continued)

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APPENDIX 1/5 TESTING TO BE CARRIED OUT BY THE CONTRACTOR

(c)

Clause	Work, Goods or Material		Test	Frequency of Testing	Test Certificate	Comments
Series 600						
601, 631 to 637 & 640	Acceptable material		For recycled aggregate, see sub-clauses 601.12 and 601.18. Cross-reference should be made to any requirements in Appendix 6/1			
	Class	General Description				
	1	General granular fill	Grading / uniformity coefficient	Twice a week	Required	
			MC (N)	2 per 1000m³ up to max of 5 per day per source		
			OMC (N)	Once per week per source		
		1C only	Los Angeles coefficient (N)	Weekly		
	2	General cohesive fill	Grading	Twice a week	Required	
			MC (N)	2 per 1000m³ up to max of 5 per day per source		
			PL/LL (N)	Weekly		
			MCV (N)	2 per 1000m³ up to max of 5 per day per source		
			Undrained shear strength – Hand Shear Vane	2 per 1000m³ up to max of of 5 per day per source		See Note 13 to Table 6/1
			Undrained shear strength of remoulded material (N)	2 per 5000m³ up to max of 2 per week per source		See Note 13 to Table 6/1
	2E	Reclaimed pulverised fuel ash cohesive material	MC (N)		Required	
			Bulk Density (N)	-		
	4	Fill to Landscape Areas	Grading	Daily	Required	
			MCV (N)	Daily		
	5	Topsoil	Grading	Daily	Required	

Figure 75.4 (Continued)

Two examples:

1. Primary aggregate Type I sub-base is supplied from a major quarry, subjected to a mature quality assurance system and CE marked.
2. 6N/6P backfill to a structure under a motorway is being supplied from multiple sources, processing to produce recycled aggregates.

In the first example, the likelihood of occasional non-conformance is low. In this situation, the deleterious effect on the works of, say, the product's grading analysis being 2% to 3% non-compliant on one of the coarser sieves is insignificant.

In the second example, it is not the author's intent to imply that the use of recycled aggregate is less suitable – quite the contrary, recycled and secondary aggregates should be used wherever possible for sustainable construction – but a specification compiler potentially in this situation must consider:

- multiple sources are being used;
- recycled material is produced from a variable feedstock;
- possible structural failure from frequent non-conformance;
- the nature and criticality of failure (e.g. slope instability, settlement).

Both compliant materials may be completely acceptable in performance terms and suitable for use at the locations. But, to manage risk effectively, it would be reasonable to expect material being used in example 2 to be sampled and tested more frequently than that in example 1.

75.2 Compaction

Compaction is the final action (other than trimming of the uppermost layer) in the sequence of earthworks events which commenced with excavation or import, followed by haulage to the point of deposition, tipping/discharge and spreading. Throughout these processes, the material has been disturbed and loosened, has high air voids and is mechanically unstable.

Compaction is achieved by the application of a force to the surface of the spread material. On major earthworks projects, this is usually in the form of a heavy roller, either towed or self-propelled and deadweight or vibratory. Where space is very restricted, or on small projects, point load plant may be used instead.

Compaction does not necessarily produce a 'dense' material; the term is subjective. The purpose of compaction is to produce a material which has, by packing the soil particles more closely together:

- air voids lower than a required maximum;

and/or

- dry density higher than a required minimum;
- predictable settlement and consolidation characteristics within prescribed limits;
- increased mechanical stability under load.

Further consequences may be, depending on the material type:

- increased impermeability;
- improved resistance to erosion;
- greater strength or stiffness.

75.2.1 The mechanics of compaction

75.2.1.1 Terminology

- moisture content: the mass of water in a soil as a percentage of the mass of dry soil;
- bulk density: the mass per unit volume of soil including water;
- dry density: the mass per unit volume of the soil excluding water, expressed by the equation:

$$\text{Dry density (Mg/m}^3) = \frac{[\text{Bulk density (Mg/m}^3) \times 100]}{[\text{Moisture content (\%)} + 100]} \quad (75.1)$$

Any earthworks material comprises solid particles, water and air voids. Compaction increases the packing of the solid particles and reduces air voids: water lubricates the soil particles and displaces air. From a dry condition, increasing water (or moisture) content increases the particle packing – measured as the dry density – and reduces air voids when a soil is compacted.

For any soil and applied compactive effort there is a condition when the water in the soil is just sufficient for the particle packing to be maximised and air voids minimised. This condition is the Maximum Dry Density (MDD). The water content at which MDD is achieved is Optimum Moisture Content (OMC).

At moisture contents below OMC, there is neither sufficient water to lubricate the particles for maximum density, nor to fill all the voids between the particles. Above OMC, water displaces the solid particles. As a result, dry density decreases and, with increasing moisture content, the material becomes increasingly unstable (e.g. deforming more under load).

The relationship between moisture content, dry density and air voids is demonstrated by drawing a compaction curve. This is from a series of tests measuring the dry density, at varying moisture contents, achieved by compacting each specimen in an identical manner, to the method defined in BS 1377 (see **Figure 75.5**).

MDD and OMC are a function of the compactive effort applied to any soil. Increasing the compactive effort raises MDD and lowers OMC. The compaction curve, whilst maintaining approximately the same shape, moves up and left. It does not approach the 0% air voids line more closely. A state in which air voids equal zero is not physically attainable with practical earthworks compaction methodologies.

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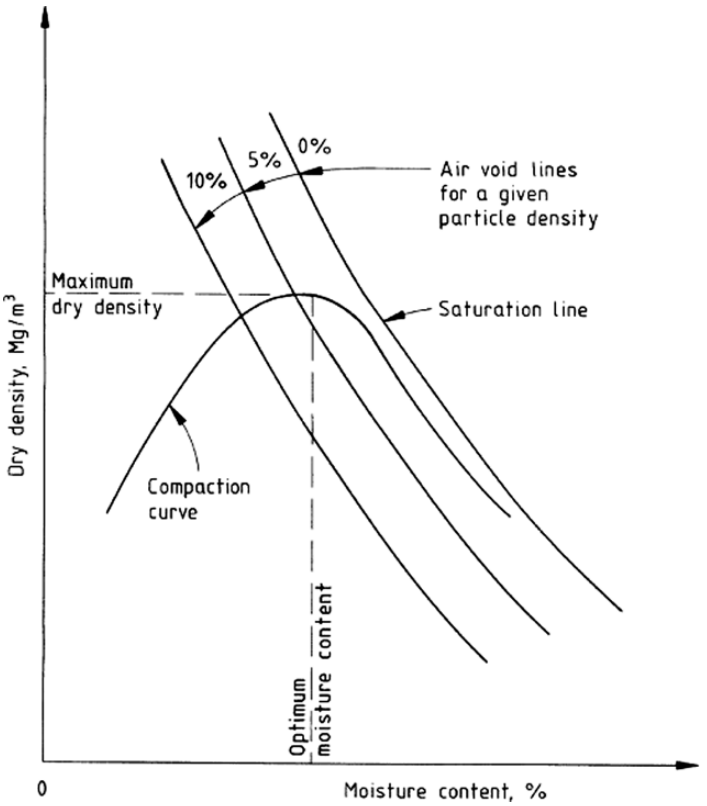


Figure 75.5 Dry density, moisture content and air voids relationship
Reproduced with permission from BS 1377–4. © British Standards Institution 1990

MDD, OMC and the shape of the curve are influenced by physical properties of the soil:

- particle shape;
- particle size;
- range of particle sizes.

Generally, angular or fine soils have a higher water demand than rounded or coarse soils. Well-graded materials, where particle size distribution is over a wide range of sizes, tend to have steeper curves than uniform materials, with particle sizes over a small range. It would appear from this that the permutations for OMC / MDD would be infinite; however, general ranges can be outlined for different material types:

Material	OMC range (%)
Clay	25–35
Silty, sandy clay	20–30
Pulverised fuel ash	15–25
Fine sand	10–20
Coarse sand	5–15
Sand and gravel	5–10
Crushed rock	5–10

The compaction curve is used by the specification compiler to set the moisture content acceptability limits for a soil type or class. The lower limit should ensure that air voids are less than the maximum acceptable for the design. The upper limit should be before increasing moisture causes any significant instability.

75.2.2 The compaction specification

The compaction specification must define the requirements for each class of material allowed in the project, as defined in Table 6/1.

75.2.2.1 Method compaction

For each type of compaction plant and the amount of compactive effort imparted (e.g. mass per metre width of roll), the requirements are defined for:

- the maximum thickness of the layer being compacted, when compaction is complete;
- the number of passes (roller) or applications (point load plant).

Table 6/4 lists the permitted permutations for each material class and Method number in Table 6/1 – see **Figure 75.6(a)** and **6(b)** for Methods 1 to 6 (Method 7 omitted). SHW permits plant and methods other than those in this table, if the effectiveness of the methodology is proven by site trials.

Method compaction is generally used for bulk earthworks operations. The methods specified in Table 6/4 assume a conservative approach and have been determined to deliver maximum air voids of between 5% and 10%, depending on the type of material and material class.

75.2.2.2 End Product compaction

The required level of compaction is specified. In earthworks this is usually a minimum *in situ* density, expressed as percentage of the MDD of that material. This percentage is 90% to 100%, depending on the material type and class. The MDD is obtained from the laboratory test defined in Table 6/1; either:

- BS 1377: Part 4 (2.5 kg rammer method) – usually used for cohesive materials; or
- BS 1377: Part 4 (vibrating hammer method) – usually used for granular materials; also imparts a greater compactive effort than the 2.5 kg rammer test.

The BS 1377: Part 4 (4.5 kg rammer method) test is not used by SHW Table 6/1.

The type of plant used, the thickness of the final compacted layer and the number of passes are not specified. Selection of these is the responsibility of the contractor carrying out the compaction operation, guided by the compaction plant manufacturer's recommendations.

Table 6/1 also defines the maximum and minimum moisture contents at which a material is acceptable to be placed in the works and compacted.

Table 6/4: method compaction for earthworks materials: plant and methods (method 1 to method 6) (this table is to be read in conjunction with sub-clause 612.10)

(a)

Type of compaction plant	Ref no.	Category	Method 1		Method 2		Method 3		Method 4		Method 5		Method 6		
			d	n #	d	n #	d	n #	d	n	d	n	n for d = 110 mm	n for d = 150 mm	n for d = 250 mm
Smoothed wheeled roller (or vibratory roller operating without vibration)		mass per metre width of roll:													
	1	over 2100 kg up to 2700 kg	125	8	125	10	125	10*	175	4	unsuitable		unsuitable	unsuitable	unsuitable
	2	over 2700 kg up to 5400 kg	125	6	125	8	125	8*	200	4	unsuitable		16	unsuitable	unsuitable
	3	over 5400 kg	150	4	150	8	unsuitable		300	4	unsuitable		8	16	unsuitable
Grid roller		mass per metre width of roll:													
	1	over 2700 kg up to 5400 kg	150	10	unsuitable		150	10	250	4	unsuitable		unsuitable	unsuitable	unsuitable
	2	over 5400 kg up to 8000 kg	150	8	125	12	unsuitable		325	4	unsuitable		20	unsuitable	unsuitable
	3	over 8000 kg	150	4	150	12	unsuitable		400	4	unsuitable		12	20	unsuitable
Deadweight tamping roller		mass per metre width of roll:													
	1	over 4000 kg up to 6000 kg	225	4	150	12	250	4	350	4	unsuitable		12	20	unsuitable
	2	over 6000 kg	300	5	200	12	300	3	400	4	unsuitable		8	12	20
Pneumatic-tyred roller		mass per wheel:													
	1	over 1000 kg up to 1500 kg	125	6	unsuitable		150	10*	240	4	unsuitable		unsuitable	unsuitable	unsuitable
	2	over 1500 kg up to 2000 kg	150	5	unsuitable		unsuitable		300	4	unsuitable		unsuitable	unsuitable	unsuitable
	3	over 2000 kg up to 2500 kg	175	4	125	12	unsuitable		350	4	unsuitable		unsuitable	unsuitable	unsuitable
	4	over 2500 kg up to 4000 kg	225	4	125	10	unsuitable		400	4	unsuitable		unsuitable	unsuitable	unsuitable
	5	over 4000 kg up to 6000 kg	300	4	125	10	unsuitable		unsuitable		unsuitable		12	unsuitable	unsuitable
	6	over 6000 kg up to 8000 kg	350	4	150	8	unsuitable		unsuitable		unsuitable		12	unsuitable	unsuitable
	7	over 8000 kg up to 12000 kg	400	4	150	8	unsuitable		unsuitable		unsuitable		10	16	unsuitable
	8	over 12000 kg	450	4	175	6	unsuitable		unsuitable		unsuitable		8	12	unsuitable
Vibratory tamping roller		mass per metre width of a vibrating roll:													
	1	over 700 kg up to 1300 kg	100	12	100	12	150	12	100	10	unsuitable		unsuitable	unsuitable	unsuitable
	2	over 1300 kg up to 1800 kg	125	12	125	12	175	12*	175	8	unsuitable		12	unsuitable	unsuitable
	3	over 1800 kg up to 2300 kg	150	12	150	12	200	12*	unsuitable		unsuitable		8	12	unsuitable
	4	over 2300 kg up to 2900 kg	150	9	150	9	250	12*	unsuitable		400	5	6	10	unsuitable
	5	over 2900 kg up to 3600 kg	200	9	200	9	275	12*	unsuitable		500	6	6	10	unsuitable
	6	over 3600 kg up to 4300 kg	225	9	225	9	300	12*	unsuitable		600	6	4	8	unsuitable
	7	over 4300 kg up to 5000 kg	250	9	250	9	300	9*	unsuitable		700	6	3	7	12
	8	over 5000 kg	275	9	275	9	300	7*	unsuitable		800	6	3	6	10

Figure 75.6 Extracts from Highways Agency SHW 600 Series, Table 6/4
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TABLE 6/4: Method Compaction for Earthworks Materials: plant and Methods (Method 1 to Method 6)
(This Table is to be read in conjunction with sub-Clause 612.10)
(b)

Type of Compaction Plant	Ref No.	Category	Method 1		Method 2		Method 3		Method 4		Method 5		Method 6		
			D	N#	D	N#	D	N#	D	N	D	N	N for D = 110 mm	N for D = 150 mm	N for D = 250 mm
Vibratory roller		Mass per metre width of a vibratory roll:													
	1	over 270 kg up to 450 kg	unsuitable		75	16	150	16	unsuitable		unsuitable		unsuitable	unsuitable	unsuitable
	2	over 450 kg up to 700 kg	unsuitable		75	12	150	12	unsuitable		unsuitable		unsuitable	unsuitable	unsuitable
	3	over 700 kg up to 1300 kg	100	12	125	10	150	6	125	10	unsuitable		16	unsuitable	unsuitable
	4	over 1300 kg up to 1800 kg	125	8	150	8	200	10*	175	4	unsuitable		6	16	unsuitable
	5	over 1800 kg up to 2300 kg	150	4	150	4	225	12*	unsuitable		unsuitable		4	6	12
	6	over 2300 kg up to 2900 kg	175	4	175	4	250	10*	unsuitable		400	5	3	5	11
	7	over 2900 kg up to 3600 kg	200	4	200	4	275	8*	unsuitable		500	5	3	5	10
	8	over 3600 kg up to 4300 kg	225	4	225	4	300	8*	unsuitable		600	5	2	4	8
	9	over 4300 kg up to 5000 kg	250	4	250	4	300	6*	unsuitable		700	5	2	4	7
	10	over 5000 kg	275	4	275	4	300	4*	unsuitable		800	5	2	3	6
Vibrating plate compactor		Mass per m^2 of base plate:													
	1	over 880 kg up to 1100 kg	unsuitable		unsuitable		75	6	unsuitable		unsuitable		unsuitable	unsuitable	unsuitable
	2	over 1100 kg up to 1200 kg	unsuitable		75	10	100	6	75	10	unsuitable		unsuitable	unsuitable	unsuitable
	3	over 1200 kg up to 1400 kg	unsuitable		75	6	150	6	150	8	unsuitable		unsuitable	unsuitable	unsuitable
	4	over 1400 kg up to 1800 kg	100	6	125	6	150	4	unsuitable		unsuitable		8	unsuitable	unsuitable
	5	over 1800 kg up to 2100 kg	150	6	150	5	200	4	unsuitable		unsuitable		5	8	unsuitable
	6	over 2100 kg	200	6	200	5	250	4	unsuitable		unsuitable		3	6	12
Vibro-tamper		Mass:													
	1	over 50 kg up to 65 kg	100	3	100	3	150	3	125	3	unsuitable		4	8	unsuitable
	2	over 65 kg up to 75 kg	125	3	125	3	200	3	150	3	unsuitable		3	6	12
	3	over 75 kg up to 100 kg	150	3	150	3	225	3	175	3	unsuitable		2	4	10
	4	over 100 kg	225	3	200	3	225	3	250	3	unsuitable		2	4	10
Power rammer		Mass:													
	1	100 kg up to 500 kg	150	4	150	6	unsuitable		200	4	unsuitable		5	8	unsuitable
	2	over 500 kg	275	8	275	12	unsuitable		400	4	unsuitable		5	8	14
Dropping-weight compactor		Mass of rammer over 500 kg weight drop:													
	1	over 1 m up to 2 m	600	4	600	8	450	8	unsuitable		unsuitable		unsuitable	unsuitable	unsuitable
	2	over 2 m	600	2	600	8	unsuitable		unsuitable		unsuitable		unsuitable	unsuitable	unsuitable

Figure 75.6 (continued)

Recent changes to SHW have incorporated BS EN standards and greater emphasis on suppliers to provide conformity compliance control. Performance-based specifications allow a wider range of materials (e.g. recycled and secondary aggregates) to be more widely used.

75.3 Compaction plant

Many types and sizes of compaction plant are available. The Method Compaction options in Table 6/4 permit many of these for each material class, though experience has proved some to be more effective than others. For End Product compaction, not all of these are suitable for all materials, locations and conditions. Selection of plant for End Product compaction should be made with careful reference to the plant manufacturer's recommendations and, more importantly, an experienced earthworks manager.

The drum roller depicted in **Figure 75.7** is towed behind a tractor dozer, often the same plant used for spreading the tipped material. This plant can be obtained either as a deadweight roller or with a vibratory mechanism fitted.

This plant is normally deployed as the compaction plant where large volumes of Class 2 material (cohesive general fill) are to be placed and compacted to a Method Compaction specification.

As it is towed by a tracked machine, this plant can be used on a wide range of soft materials where a self-propelled roller might become bogged or cause rutting. A single drum roller, towed by a tractor, may have greater productivity than a self-propelled roller (see **Figure 75.8**), but is less manoeuvrable.

The self-propelled roller is effective in compacting most material types in many construction operations, including earthworks. The roller is normally fitted with a vibratory mechanism, which can be selected to work at various amplitudes and frequencies to suit the material. The manufacturer will provide data on the correct selection of these and an experienced earthworks manager should also be consulted.

Table 6/1 precludes the use of vibratory rollers on cohesive materials above a certain Liquid Limit. This plant may be operated as a deadweight roller to satisfy this requirement.

The driving wheels of this plant are pneumatic tyres with deep treads. These provide reasonable traction on soft cohesive materials. The self-propelled smooth drum roller is also available as a twin drum roller.

Available in deadweight or vibratory form, the tamping roller, commonly referred to as a sheepsfoot roller (**Figure 75.9**), is very effective at compacting high Plasticity Index clays. It can also be effective on sands and gravels, though usually those with a high silt and clay content. It is rarely used on granular fills, where a smooth drum vibratory roller is more effective.

A drawback of tamping rollers is the depressions left in the surface, which can trap water, leading to a softening of the material. If rain is expected, it is good practice to compact the surface with a smooth drum roller or, if this does not remove the depressions, blade off the surface and recompact with a smooth drum.

As with the drum roller, Table 6/1 precludes using this plant in vibratory mode on clays above a specific Liquid Limit to avoid damage to the structure of the clay.

The grid roller (**Figure 75.10**) is normally a towed item of plant. It can be obtained in a range of roll widths and masses. It is not usually available to operate as a vibratory action.

Grid rollers are particularly effective on coarse rockfill, e.g. starter layers to SHW Class 6B and 6C. They can be effective on sands and gravels, although other types of plant are more often used on this material.



Figure 75.7 Smooth single drum roller – towed
Courtesy of Balfour Beatty



Figure 75.8 Smooth single drum roller – self-propelled
Courtesy of Balfour Beatty

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The roller depicted in **Figure 75.11** is also available as a heavier item of plant with wider drums. This is rarely used in major earthworks/bulk fill operations due to the lack of traction from two smooth drums.

The smaller unit pictured here is used where the area to be compacted precludes the use of larger plant. This will typically be used in small general fill activities, trench backfill and backfill to structures. The size of this plant restricts the permitted layer thickness, constraining productivity where large areas are to be compacted.

The self-propelled, pedestrian-controlled single and twin drum rollers depicted in **Figure 75.12** often have lifting hitches (visible in both **Figures 75.11** and **75.12**) for plant to be craned into otherwise inaccessible locations – e.g. for compacting backfill to structures, where they are widely used. However, the compactive effort of these items of plant is limited: care has to be taken to ensure that the limit for layer thickness is not exceeded. This plant is available as remote controlled, for compacting material in a trench without operator access.

Along with the vibro-tamper (not pictured), the vibrating plate and power rammer depicted in **Figure 75.13(a) and (b)** are the lightest items of compaction plant in use in UK construction. Use of these is generally confined to narrow, shallow, trench backfill.

Neither is recommended for compacting material to an End Product compaction specification.

Although a permitted option in Table 6/4, the pneumatic tyred roller (PTR) (**Figure 75.14**) is rarely encountered in UK earthworks. It is frequently used to compact cement or bituminous bound pavement layers, where the kneading action of the tyres is effective in compacting stiff materials.

(a)



(b)



Figure 75.9 Tamping roller (sheepsfoot) – (a) towed and (b) self-propelled
Courtesy of Balfour Beatty



Figure 75.10 Grid roller – towed
Courtesy of Broons



Figure 75.11 Smooth twin drum roller – self-propelled
Courtesy of Balfour Beatty

(a)



(b)

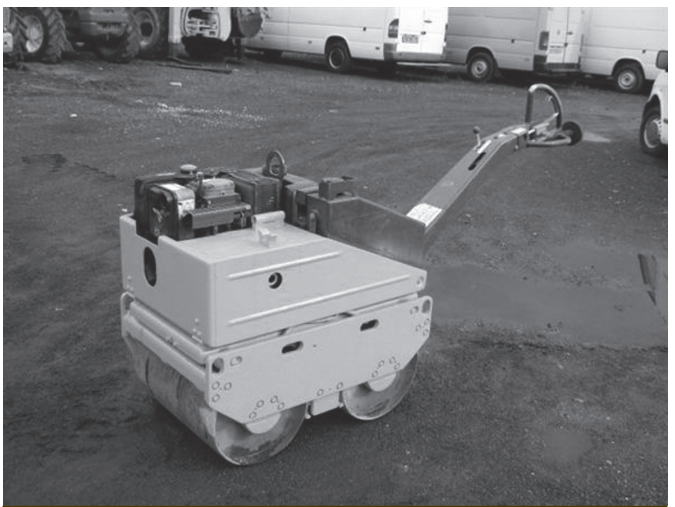


Figure 75.12 Single (a) and twin (b) smooth drum – self-propelled, pedestrian controlled
(a) Courtesy of Terex; (b) Courtesy of Balfour Beatty

(a)



(b)



Figure 75.13 Vibrating plate compactor (a) and Power rammer (b)
Courtesy of Wacker Neuson

75.4 Control of earthworks

Without a strategy to control an earthworks operation in place prior to the works commencing, non-conformances with the specification are inevitable. These may take the form of:

- unacceptable material placed in the works;
- the wrong class of material placed in the works;
- inadequate compaction.

Any of these occurring can result in some form of physical failure in the works. This might result in:

- physical injury to workforce or the public;
- prosecution;
- poor perception by the public of the works;
- deterioration in client–contractor relationship;
- damage to both client and contractor reputations;
- increased cost and delays.

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Figure 75.14 Pneumatic tyred roller (PTR)
Courtesy of Balfour Beatty

Protocols and processes must be established. For the quality control of the earthworks, these must identify:

1. The client's Specific/Core Requirements.
2. Which specifications, and which revision of these, are relevant to the contract.
3. Whether the drawings have the right information, and there is no ambiguity.
4. Who contractually is responsible for risk.
5. Management and supervision.
6. A clearly defined chain of command from site operatives to the earthworks manager.
7. The need for a UKAS accredited site laboratory.
8. Inspection and test plans to comply with the client's requirements.
9. Additional testing requirements to manage specific risks.
10. Where Value Engineering could be used to benefit the project.
11. A system for rapidly reviewing test results to monitor for trends.
12. Transparency of test results between contractor, designer and client.
13. A system for reporting, addressing and closing non-conformances.
14. A no-blame culture, especially to allow early warnings to be acted upon.

Considering the above in detail:

1. **The client's specific/core requirements**
   A proposed method or material, an alternative to criteria here, may require a formal departure from standard to be approved by the client. All parties must be fully cognizant of how adopting such a departure alters the balance of risk ownership (see 4).

2. **Which specifications, and which revision of these, are relevant to the contract**
   Every engineer's office on a construction project should have a hard copy of the client's overall specification and the contract specific documents. These should be issued by one person on the contract – a document controller – responsible for ensuring all documents are kept up to date. This also applies to external specifications.

3. **Whether the drawings have the right information, and there is no ambiguity**
   Not simply a task delegated to the document controller, there should be a central register of dated current drawing revisions to which any engineer can refer to confirm the right one is being used.

4. **Who contractually is responsible for risk**
   A risk register compiled before the works commence must identify this issue. It must not be seen as a buck-passing exercise, but so that the parties responsible can be prepared and plan the mitigation.

5. **Management and supervision**
   A senior manager must be appointed to oversee the planning, management or coordination of the earthworks operations. On most projects, a works manager for earthworks will be appointed to supervise site operations.

6. **A clearly defined chain of command from site operatives to the earthworks manager**
   If an earthworks sub-contractor is used, this must include how its managers, staff and workforce collaborate with the main contractor. An organogram is essential. It should name staff and provide job titles for all parties, client, designer, contractor and sub-contractor. The inclusion of mobile phone numbers is useful, as are photos of staff at the start of a contract. The organogram should be displayed in a part of the office accessed by all personnel.

7. **The need for a UKAS accredited site laboratory**
   The specification will state whether or not this is a core requirement of the client. Most contracts require the contractor to carry out all earthworks testing; on any medium to large project adequate control is not feasible without a laboratory presence on site, if purely for reaction time to any situation.

8. **Inspection and test plans to comply with the client's requirement**
   SHW requires the production of Appendix 1/5: Testing to be carried out by the contractor. This includes tests to be carried out, at defined sampling rates, for the materials, including bulk earthworks, used in the project. The compilers take advice on test rates from SHW NG and HA44/91. It is strongly advised that, as with other parts of the earthworks specification, the contractor should contribute to the compilation of this appendix. This will draw on the broadest experience for the highest risk materials to be controlled and risk minimised.

9. **Additional testing requirements to manage specific risks**
   This is in addition to test rates defined in Appendix 1/5. Some materials can be realistically viewed as very low risk of non-conformance. These typically include processed and blended material imported from external quarries, covered by the supplier's Quality Plan and, often, EC certificated. The test regime for these need be no more than minimal. Other materials carry more risk of non-conformance. These include site-won materials where the deposit is known, from the GI, to be variable (including sands and gravels) and imported recycled material, due to the variation of feedstock. There must be a person appointed to know from where materials are sourced. This person, usually the site materials engineer, must be sufficiently experienced to assess the level of risk and how much the sampling rate should be increased above the minimum to mitigate this.

10. **Where value engineering could be used to benefit the project**
    This can be as simple as reviewing updates in specifications and standards and, where this offers a benefit to the project, for client, designer and contractor collaborating for their implementation. It can also be as complex as a major re-design of an element in the works. In any case, it is incumbent on all parties to encourage creative thought amongst staff.

11. **A system for rapidly reviewing test results to monitor for trends**
    A large earthworks project will generate a lot of test data, whether from a site or external laboratory. This is useless unless subjected to frequent, regular review. A UKAS accredited laboratory will, for each test carried out, produce a hard copy test report. This is a requirement of accreditation, but does not make review of multiple results realistic. A simple spreadsheet, on which earthworks testing – *in situ* and in laboratory – can be entered allows rapid review of all testing, for whichever material or property is filtered. Non-conformances can be quickly identified and, more importantly, trends identified, so early action can be taken to avoid non-conformance.

12. **Transparency of test results between contractor, designer and client**
    Having created a results database, it should be reviewed by all parties with a stakeholding in the success of the project. If a project has a local electronic network, it should be stored so that the client, client's agent, designer and contractor have open access to the data. Security measures must prevent alteration by those other than appointed persons.

13. **A system for reporting, addressing and closing non-conformances**
    A system should exist for openly addressing non-conformances to allow closure to the satisfaction of the client and designer. A non-conformance must be seen as an opportunity to improve, not as means to apportion blame for a failure. This can only work effectively in…

14. **A no-blame culture, especially to allow early warnings to be acted upon**

75.5 Compliance testing of earthworks

All testing, whether for compliance of the material or to measure the level of compaction, must have a minimum frequency specified as stated above. The frequency of testing must take into account all risks associated with the material:

- Likelihood of structural failure through non-conformance:

  some elements are critical, e.g. backfill to a structure under a motorway pavement.

- Potential variability of material composition:

  recycled aggregate will vary more than a quarry product from primary aggregate.

- Potential variability of material properties, e.g. moisture content:

  recycled aggregate is likely to be more variable than primary aggregate.

75.5.1 Material compliance

All earthworks materials, whether re-used, site-won or imported, must be used within the limits defined for various parameters by the client's specification and the designer's geotechnical design report. For projects using the SHW, these are specified in the 600 series and associated appendices and tables.

There is a wide range of physical, mechanical and chemical soil characteristics for which tests are carried out to ascertain compliance with specified suitability limits. It is outside the scope of this chapter to list all these, but **Table 75.3** gives tests which may be used to control the physical suitability of cohesive material for earthworks general fill – i.e. SHW Class 2.

75.5.1.1 Compaction compliance: Method Compaction

Although the tests quoted in **Table 75.4** may be carried out on Method Compaction earthworks, this is usually to confirm the

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Test	Advantages of test	Constraints of test	Typical acceptable Limits
Moisture Condition Value (MCV)	Rapid – test takes about 15 min. Can be carried out on site.	Must be correlated against fundamental property tests to define limits. Not especially effective for granular soils, or very silty or sandy clays.	8–16
Hand Shear Vane (HSV)	Rapid – test takes 1 min. Can be carried out on site. Direct measure of cohesion/undrained shear strength C_u.	Not suitable for clay containing much sand or gravel.	>70 kPa
Plastic Limit (PL)	Used to define material properties at design stage – not often used for earthworks control.	Must be carried out in laboratory. Test can take 24 hrs from sampling to result.	See MC
Optimum Moisture Content (OMC)	Effective for granular soils, or very granular clays (e.g. Glacial Till) where MCV or HSV may not be suitable.	Must be carried out in laboratory. Test can take >48 hrs from sampling to result.	See MC
Quick undrained triaxial	Used to define material properties at design stage – rarely used for earthworks control. Direct measure of cohesion/undrained shear strength C_u.	Must be carried out in laboratory Test can take >48 hrs from sampling to result.	>70 kPa
Moisture Content (MC)	Fundamental soil property which should be measured for every soil sample taken. Can be rapid if microwave method used (correlated against oven dried).	Requires another test, e.g. OMC or PL, to provide control reference.	< PL x 1.2 >OMC-2 <OMC+3
Particle Size Distribution (PSD) – also known as Grading	Not frequently carried out on general fills (SHW Class 1 or 2) as other parameters of these materials affect suitability more. More often carried out on selected fills (SHW Class 6), where PSD has a greater effect on performance.	Generally, 48 hrs from sampling to result as two drying cycles are required for materials which have to be washed of clay and silt. A sample size of at least 80 kg is needed for material of 90 mm nominal size.	See SHW Table 6/2
California Bearing Ratio (CBR)	CBR is not a routine acceptability test, but engineers must be aware of the test. CBR is an empirical measure of material shear strength. It is used to design the thickness of the foundation layer of a pavement. It also identifies soft areas which require treatment (replacement or improvement) prior to placing the foundation. A rapid assessment of CBR can be made using a hand-held MEXE probe.		>2.5%
Falling/Light Weight deflectometer (FWD/LWD)	Like CBR, this is not a routine acceptability test but is used to measure the *in situ* stiffness of soils. This provides data for pavement foundation design and design verification testing.		–

Table 75.3 Tests which may be used to control the physical suitability of cohesive material for earthworks general fill (SHW Class 2)

conclusions of a geotechnical design, rather than as a control. The key control of Method Compaction is to ensure that the methodology is correctly carried out. This entails checking:

- that the method is suitable for the material class (SHW Table 6/4);
- that the type of compaction plant is suitable;
- the compactive effort (mass/m roll width etc.) of the plant.

This can be obtained by reference to the contract specification and to the plant manufacturer's performance data sheets.

In SHW, for each method and category of compactive effort, there are limits for the maximum thickness of compacted layer and minimum number of passes. These should be ascertained by direct measurement. Daily records must be kept of all observations and measurements.

75.5.1.2 Compaction compliance: End product compaction

Where a minimum level of compaction is specified for a material the compacted material must be tested *in situ* to ascertain compliance with the specification. It is strongly recommended that material is not overlaid with the next layer until it has been confirmed that the layer under test complies with the specification.

The test methods in regular use in the UK are listed in **Table 75.4**.

For each test method the moisture content of the soil must be determined in laboratory conditions to calculate the dry density. This is compared to the MDD obtained for that material (by the method specified) to be expressed as % of MDD.

Testing with a Nuclear Density Meter (NDM) has been used in the UK since at least the early 1980s and is a methodology

Test	Advantages of test	Constraints of test
Nuclear Density Meter (NDM)	Rapid – a result from a single test point takes less than a minute. Speed of test enables many test points to be taken. Suitable for most soil types, fine and coarse grained; unbound and hydraulically bound.	Significant administrative input required to manage issues of: Heath & safety Licensing Security. NDM apparatus requires regular correlation for each material against actual density. Moisture measurement by NDM is not reliable
Core cutter	Quick and simple to carry out on site. Can be rapid if microwave method used for MC (correlated against oven dried).	Not suitable for other than fine cohesive soils.
Sand Replacement Density	Suitable for coarse-grained soils.	Slow and labour-intensive.

Table 75.4 Compaction test methods in regular use in the UK

accepted by specifiers, designers and contractors. The technology is proven for bulk density measurement but not moisture measurement; dry density must be calculated from the moisture content, tested in the laboratory, of a soil sample from the NDM test location.

The legislation regarding the ownership and storage of NDM is extensive and it is not within the scope of this chapter to discuss this in detail. Anyone considering the acquisition of NDM is strongly recommended to seek the advice of a radiation protection adviser in the first instance, as soon as possible. Registration with the Environment Agency (EA) may take four months. NDM site testing may be carried out by a hired 3rd party operator registered with the EA; this is less onerous.

75.5.2 Selection of compliance parameters and limits

The specification compiler must select the test methods to be used on the project to control the earthworks. The compiler must take into account:

- suitability of the method for the materials to be encountered;
- urgency of the result to determine compliance with specification;
- avoiding conflicting parameters/limits.

Tables 75.3 and **75.4** provide guidance on the first two of these points. It is worth stressing the importance of selecting the most rapid method appropriate to the nature of the contract and earthworks: the triaxial test will certainly define whether a clay is a suitable fill or not; specifying it as the means to control the earthworks on a project where 1 000 000 m^3 of clay has to be moved in a season is not practical.

The MCV test was developed by TRL (then TRRL) in the 1970s as a rapid method of controlling earthworks for material acceptability. The method is ideal for medium to high plasticity 'heavy' clays, e.g. Lias, London, Gault; but can be effective on very silty or sandy clays if correlation testing against fundamental properties – e.g. shear strength – is carried out prior to the works commencing.

The Hand Shear Vane, when calibrated, is an effective method for cohesive materials of varying plasticity. It is less suitable if the clay is expected to include fractions larger than medium sand: coarser particles engaging with the vanes can produce erroneous results.

The third point needs careful consideration. In allowing more than one test method as a means of control, the specification compiler can introduce a paradox where a material tested to one method may comply but, tested to the other, may not. This may be a result of limit selection being imperfect, variation in the material from that used in correlation testing or just that natural material does not necessarily conform precisely to the models made for it.

In the example specification provided (**Figure 75.4(a)**), this possibility was addressed by a footnote to the table. This allowed earthworks control by Hand Shear Vane *or* MCV, so long as the Appendix 1/5 requirement for the minimum number of MCV was satisfied.

Selection of limits for a material must take account of:

- the material properties;
- results of any correlation testing;
- understanding how material will perform when subjected to the expected load;
- other physical conditions that may affect this performance;
- the acceptable level of risk/factors of safety.

Determining these limits is the role of geotechnical specialists of the designer and the client and, where possible, the contractor.

75.5.3 Dealing with non-conformance

This chapter has used 'non-conformance' and 'failure' as distinct and non-interchangeable terms. Non-conformance, where one or more of the tested properties of a material lies without

the specified limits, does not necessarily imply that the later, structural failure, is inevitable or even possible.

Whilst all non-conformances must be investigated, it must be noted that, in statistical analysis of a table of test results – the Normal Distribution Curve (Bell Graph) – a small number of 'defectives' are expected. About 4% of results of any test might be expected to be marginal non-conformances. It is not within the scope of this chapter to discuss how far outlying the specified limits constitutes a marginal non-conformance, or the statistical analysis of data.

When a test result lies outside the acceptable limits for that parameter, most quality control systems require the production of a Non-Conformance Report (NCR). This will describe the:

- material type and class (if appropriate);
- location used;
- specified limits;
- test result obtained.

This will allow the team assessing the NCR to make a judgement on the action required. This may vary and include:

- the criticality of the non-compliance, by severity and location;
- consideration that the factor of safety has not been affected and no action is necessary;
- resampling and retesting – the non-conformance may result from sampling bias;
- treatment to modify the characteristics of the material to comply with the specification;
- removal and replacement with compliant material.

As with all matters involving earthworks and the specification, the review of the NCR and the decision-making process should include client, designer and contractor. A solution that is without negative effect on the safety of the project, but is practical and commensurate with the situation is to be sought.



Figure 75.15 *In situ* NDM testing
Courtesy of Balfour Beatty



Figure 75.16 *In situ* core cutter testing
Courtesy of Balfour Beatty

75.6 Managing and controlling specific materials

If not correctly managed, all materials may create problems for the earthworks operations. Some, in certain conditions, may be considered by the engineer as 'difficult', i.e. more prone to failure, even though there may have been no non-conformance with the specification.

75.6.1 Clay

Although not necessarily a difficult material to manage, working with clay merits consideration and planning that will benefit an engineer controlling earthworks.

Clay is moisture sensitive; more so with increasing silt content. Whilst light rainfall might not necessitate a temporary halt in earthworks, an experienced earthworks manager must be consulted on when work should cease. Continuing to traffic clay with earthworks plant, especially on narrow, channelled haul roads, during rainfall, has several negative effects:

- Safety. On clay, particularly firm-stiff high PI clays, light rain will reduce traction/grip for wheeled construction plant.
- Softening of acceptable material, so that it becomes unacceptable. This is increased by rutting.
- Rutting. Once started this can lead to a rapid deterioration of the material, as water is trapped in ruts, softening the clay and leading to deeper ruts (**Figure 75.17**).

Rutting is a serious problem, as trapped water will have to be drained away. On an embankment this may be done relatively easily, but in a cutting this may need pumping and long pipe runs. This has a negative effect on the programme and entails additional cost.

(a)



(b)



Figure 75.17 Earthworks after heavy rain: (a) not well drained; (b) rolled beforehand and well drained
Courtesy of Balfour Beatty

Planning, especially if rain is forecast, can mitigate or eliminate the effects of rain, mainly by managing the earthworks operations to create drainage paths:

- *Crossfall*. Manage deposition and compaction so that there is always a fall to the outer edge of the works.
- *Longitudinal fall*. In a fill, crossfall is easily created and is effective. In cuts there is no over-the-edge drainage. In this case, whilst a crossfall will take the water initially away from the works and haul road, drainage requires a longitudinal fall to the open end of the cutting. Care must be taken to ensure there is no accidental damming.
- *Grips*. Surface drainage channels to carry water away from low spots that will inevitably occur no matter how well the works are planned.
- *Proof rolling*. Although rolling is concurrent with deposition, with rainfall imminent additional attention should be taken to close the surface of the material, to prevent ponding. A smooth surface will also shed water along the crossfall faster.

  Any action to drain rainwater must take into account that a discharge off site may require licensing by the EA. A limit on suspended solids in any such discharge may be imposed.



Figure 75.18 Lime being added to saturated earthworks – the cloud is steam from the exothermic reaction between the quicklime and water, not lime dust
Courtesy of Balfour Beatty

Haul roads on clay must be maintained to avoid rutting. If ruts are allowed to develop, haulage times are increased, more fuel is burnt by the plant and in the worst cases, plant may become stuck and need pulling or pushing out. All these reduce the efficiency of the operation, increasing cost and adding delays. Rainwater ponding in ruts accelerates any deterioration.

Allowance must be made for a grader to trim the haul road so rutting does not develop. A roller may also be required.

Wet or soft unacceptable clay, either naturally occurring or through deterioration, can be reverted, but this needs planning and has programme and cost implications:

- *Natural drying*. Wet clay is excavated and loosely spread in windrows where sun and wind can effect natural drying. This process requires confidence for a prolonged spell of fair weather; it is not an option for winter. The process can take from a few days to a week or more, depending on the condition of the clay and the weather.
- *Lime modification*. Quicklime reduces moisture content by chemical combination and by the exothermic reaction evaporating some of the water. Quicklime is spread and rotavated into the clay. It should not be compacted immediately; the steam generated must be allowed channels to escape.

Lime modification must not be confused with stabilisation. Modification does not change the material from Class 7 to Class 9 and does not require the testing for those classes of material. However, the testing required for Class 2 must be carried out after treatment to confirm the material complies with the acceptability limits; further treatment is necessary if not.



Figure 75.19 Improvement with lime addition – dissipating steam
Courtesy of Balfour Beatty

Care should be taken if the clay is suspected to contain sulphate-bearing minerals such as Pyrite or Gypsum. Although modification uses a lower addition of quicklime (typically 1–2%) than stabilisation, there is still the potential for long-term, deleterious expansion from reaction with the sulphates.

The GI should be thoroughly assessed for:

- unacceptable clay, which the earthworks strategy may need to be modified;
- gypsum or pyrites described in the BH or TP logs;
- elevated sulphates;
- any testing carried out to TRL 447.

If there is any doubt, samples should be obtained from trial pits and tests 1, 2 and 4 in TRL Report 447 must be carried out. Advice should be sought from a geotechnical specialist to interpret the results.

The earthworks programme must be reviewed. Is there a programme-critical operation from late autumn to early spring when wetter weather is more likely and natural drying not feasible? Assessment and testing are required as above.

These processes take time and must be planned and undertaken well in advance of earthworks in that area of the project.

75.6.2 Silt

All the caveats for clay apply to silt, but more so. Silt is highly sensitive to variations in moisture content. As silt is more permeable than a clay, water can penetrate the material more readily.

If, during earthworks involving silt – or a very silty clay – it starts to rain, it is prudent to cease operations sooner than one would on clay. A small change in moisture content can render an acceptable fill material into slurry. Trafficking silt during rain will rapidly deteriorate the material to significant depths.

To revert unacceptable silt, lime modification can be used, but it is slightly less effective than with clay. As with clay, quicklime will reduce moisture by exothermic reaction with the free water. However, silt differs chemically from clay, and further reaction between the clay and the lime (as occurs in stabilisation) does not occur.

Natural revertion, by excavating and windrowing to dry, is just as effective for silt as it is for clay.

Draining the earthworks is even more important for silt than clay. It must be considered that silt erodes much more easily than clay and care must be taken to limit the fall of any grip to minimise this. Likewise, over-the-edge drainage of embankments must be carefully thought out to prevent the formation of deep washout rills in the batterslope and top of batter. Repairing these can be more problematic, and cost more, than the damage the drainage was intended to avoid.

Maintenance of haul roads over silt is critical to avoid rutting developing. Due to the sensitivity of the material, consideration must be made for providing a granular haul road to permit haulage in all weathers.

Haul roads can be formed from locally available material. Crushed rock, or crushed concrete/masonry is better than sand and gravel. This is due to the greater mechanical interlock of the particles providing more stability – less tendency to deformation – under the load of construction plant.

75.6.3 Fine, uniform sand

SHW Table 6/1 identifies this material as Class 1B. Two physical properties of this material characterise it:

- *Fineness*. Particle sizes are, typically, 100% <0.6 mm, 50% <0.3 mm, 10% <0.06 mm.
- *Uniformity*. The distribution of particles is over a small size range.

As a result of the effects of these characteristics, this material has:

- restricted acceptable moisture range;
- high mobility; readily moved by wind or water;
- increased risk of washout in heavy rain;
- poor traffickability by wheeled construction plant.

Techniques for managing this material generally require the close monitoring and control of moisture content:

- *Drier than OMC*. Water should be added during deposition. Water bowsers should be employed for this, and used during dry weather to prevent desiccation of the surface.
- *Wetter than OMC*. This material drains reasonably quickly. Only when excavation is below the water table is encountering this likely. If excavated and windrowed this material will drain and dry; it is important to monitor this to avoid overdrying.
- *Drainage runs*. Run-offs must be carefully planned to avoid significant falls and the risk of erosion.

- *Over-the edge drainage.* This is to be avoided, as the risk of washout is too great. To avoid the worst effects of run-off erosion, it may be better to allow rainwater to pond on the earthworks surface.

75.6.4 Chalk

Even the strongest, blockiest chalk can be reduced to a structureless mass by incorrect handling. Taking the right actions, most grades of chalk can be used in construction without increased risk:

- *Season.* Determined by the specification compilers in Appendix 6/4. This is decided by reference to rainfall and temperature records for the area and knowledge of the chalk. The usual period is March to November. However, if early spring is much wetter or colder than usual the start should be delayed.
- *Control*. Whilst Intact Dry Density and Chalk Crushing Value are used, the basic test for controlling chalk earthworks is the moisture content test. Microwave drying should be correlated to BS oven drying methods so rapid assessment can be made.
- *Excavation*. This – in fact, every part of the chalk operation – must be managed to retain as much of the chalk structure as possible. Chalk should only be excavated as a face. This should be as deep as possible; 5 m is the usual minimum, although site conditions and depth of cut may constrain this.
- *Haulage*. Chalk can lose about 2% moisture in haulage, in fair weather conditions and on an average haul distance. Chalk that is slightly wetter than the maximum limit may revert in haulage. This should be taken into account when the sampling and test plan is written.
- *Deposition.* Chalk should never be rehandled. It should be discharged from the dump truck at the point of final deposition. The lightest plant commensurate with efficiency should be used in spreading. Chalk should be spread in the thickest layer possible to maintain the structure; 0.5 m is not unreasonable.
- *Compaction*. SHW Table 6/4 permits light vibratory rollers. Unless the chalk is very strong it is normal practice to use deadweight rollers, again to minimise damage to the material. A rule-of-thumb guide is that, when the chalk starts to adhere to the drum, rolling should cease.

Any chalk fill must undergo a settlement period, before any pavement layers are added. This period is defined by the designer. It may be shortened by surcharging, which is removed prior to the addition of further layers of the permanent works.

75.6.5 Recycled and secondary materials

Engineers are likely to encounter recycled and secondary materials as fills and aggregates: there is a mature UK marketplace to supply these materials. Recycled and secondary aggregates are not interchangable terms. Neither are sourced from primary aggregate, i.e. naturally occuring sand and gravel (land or marine sourced) or crushed rock. Generally accepted definitions of each are:

> Secondary aggregate:
>
> produced as by-products of other processes.
>
> Recycled aggregate:
>
> produced from the processing of inorganic material used in a construction process.

Use of recycled and secondary aggregates is encouraged by government initiatives for sustainable construction. However, legislation and regulation require certain procedures to be carried out: there is a Duty of Care to ensure that contaminating substances, at concentrations liable to cause harm to human health or the environment, are not spread about.

The regulations covering the use of non-primary aggregates are still evolving. It would be unwise to provide definitive



Figure 75.20 Chalk cutting excavated as single deep face
Courtesy of Balfour Beatty



Figure 75.21 Chalk compaction with deadweight roller
Courtesy of Balfour Beatty

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instructions in this chapter; what is fact at the time of writing (February 2011) may easily be superseded a short time hence.

Recommendation can be given on sources of reference, the most significant of these being the Waste and Resources Action Programme – WRAP – which, in conjunction with the EA, has produced 'The Quality Protocol for the production of aggregates from inert waste'. Aggregates produced in accordance with this protocol are considered to be fully recovered and thus are not waste.

When recycled aggregates are not produced to the WRAP protocol, they are probably still considered to be waste by the regulators. Producers must then comply with the Environmental Permitting Regulations: a permit must be acquired from the EA, a process that may take many weeks to complete.

Potential users of recycled aggregate are strongly advised that, prior to importing any material to their works, they should ascertain that the supplier has produced the material in accordance with the WRAP protocol and can prove this and be subjected to audit by the purchaser. The supplier must provide test results which confirm the material complies with the protocol.

Contaminated Land: Applications In Real Environments (CL:AIRE) has, in conjunction with the EA, published "The Definition of Waste: Development Industry Code of Practice" Version 2 in March 2011. Anyone intending to re-use or recycle material is strongly recommended to acquire this freely downloadable document.

Consultation of the following is recommended to specification compilers and engineers:

- their in-house environmental nanager;
- external waste/environmental advisers;
- WRAP www.wrap.org.uk/index.html;
- CL:AIRE www.claire.co.uk/
- Environment Agency www.environment-agency.gov.uk/.

In addition to compliance with waste regulations, the use of non-primary materials may be restricted by the client's Core Requirements. The HA define their requirements for recycled aggregate in SHW clause 601.12 and Table 6/1 includes specific classes of material where recycled material is permitted.

In addition, HA DMRB Vol. 1, Sec. 1, Part 2: HD 35/04 'Conservation and the use of Secondary and Recycled Materials' provides detailed advice on using recycled materials to specifiers. Table 2.1 from this document is in part reproduced in **Table 75.5**.

It must be noted that clients other than HA may have significantly different requirements for using recycled aggregates. Before proceding, specifiers or users must ensure they understand what their client permits to be used and what may require a departure from standards, if this is possible.

Specification compilers and users should be aware that a client's Core Requirements may not include current industry best practice. For example, tyre bales are being increasingly used as lightweight embankment fill, but are not identified in **Table 75.5**.

Dicussion between client, designer/specifier and contractor is, as before in this chapter, strongly recommended to maximise the use of recycled and secondary material for sustainable construction.

75.6.6 Manufactured aggregate

The most likely material of the kind that will be encountered is lightweight aggregate. This is manufactured from sintered PFA or expanded clay and marketed under certain brand names. The materials are not included in the SHW. They are only likely to be used to resolve specific geotechnical problems and designers should seek advice from the manufacturers on the material properties and performance.

75.6.7 Peat

Although significant volumes of peat are uncommon in England and Wales, engineers expecting to work in Scotland and Ireland should be prepared to encounter this material. Peat characteristics are:

- Low cohesion/shear strength. Values of C_u are unlikely to exceed 10KPa.
- High moisture content. Values of >300% are common.
- High compressibility. When loaded, water is driven from the structure leading to a large volume change.
- Unpredictable settlement. Because peat is organic and not homogeneous, settlement is uneven over any unit of area. Actual settlement is very difficult to predict from laboratory testing.

Peat is not acceptable, in any condition, as general fill for an earthworks project. Classification of this material, according to SHW terminology, will always be Class U1. Neither is it suitable as a load-bearing material for an embankment. It must be dealt with or removed.

A strategy for dealing with peat first must consider how thick the layer is and, consequently, the volume to be removed. As it is not suitable even for landscaping, all the material may have to be removed from site. This may involve significant cost:

- there may not be a suitably licensed landfill locally;
- any landfill may have limits on the annual permitted intake of this material;
- it may attract the higher level of landfill tax, if this applies.

If peat is to be removed, it must be done with consideration for the stability of the remaining mass to each side of the excavation.

It may be preferable to leave the material in place. In this case the load of the overlying structure must be transferred to the underlying strata, usually by a piled raft. Any project encountering peat needs to be carefully planned in advance of the works.

Application and series ▶	Pipe bedding	Embankment and fill	Capping	Unbound mixtures for sub-base	Hydraulically bound mixtures for sub-base and base	Bitumen bound layers	Pq Concrete
Material ▼	500	600	600	800	800	900	1000
Blast furnace slag	✓	✓	✓	✓	✓	✓	✓
Burnt colliery spoil	×	✓	✓	✓	✓	×	×
China clay sand/stent	✓	✓	✓	✓	✓	✓	✓
Coal fly ash/pulverised fuel ash (cfa/pfa)	✓	✓	✓	×	✓	✓	✓
Foundry sand	✓	✓	✓	✓	✓	✓	✓
Furnace bottom ash (fba)	✓	✓	✓	×	✓	×	×
Incinerator bottom ash aggregate (ibaa)	✓	✓	✓	✓	✓	✓	✓
Phosphoric slag	✓	✓	✓	✓	✓	✓	✓
Recycled aggregate	✓	✓	✓	✓	✓	✓	✓
Recycled asphalt	✓	✓	✓	✓	✓	✓	✓
Recycled concrete	✓	✓	✓	✓	✓	✓	✓
Recycled glass	✓	✓	✓	✓	✓	✓	×
Slate aggregate	✓	✓	✓	✓	✓	✓	✓
Spent oil shale/blaise	×	✓	✓	✓	✓	×	×
Steel slag	✓	✓	✓	✓	✓	✓	×
Unburnt colliery spoil	×	✓	×	×	✓	×	×

KEY:

✓ Specific (permitted as a constituent if the material complies with the Specification (MCHW 1)) or
General Provision (permitted as a constituent if the material complies with the Specification (MCHW 1) requirements but not named within the Specification (MCHW 1)).

× Not permitted.

IMPORTANT NOTES:

1. Table 2.1 is for guidance only and reference must be made to the accompanying text and the Specification (MCHW 1). Materials indicated as complying with the Specification (MCHW 1) for a particular application may not necessarily comply with all the requirements of the series listed, only particular clauses. For example in the 600 series, Unburnt Colliery Spoil can satisfy the Specification (MCHW 1) as a general fill, but is excluded as a selected fill; and in Series 1000 recycled or secondary materials are not permitted within the running surface of PQ concrete. Reference should also be made to the Specification (MCHW 1) for any maximum constituent percentages of specific recycled or secondary aggregates. For example in the 1000 Series, the maximum by mass constituent of Recycled Asphalt is given under the limits for 'other material' (Table 10/2) within the Specification (MCHW 1).
2. There is no Specific or General Provision for the use of recycled glass as an aggregate in PQ concrete or Hydraulically Bound Mixtures due to the potential for deleterious alkali-silica reaction (ASR). However, its use may be permitted by the Overseeing Organisation if sufficient provisions to minimise the risk of deleterious ASR are included in the mixture design.
3. There is no Specific or General Provision for the use of steel slag as an aggregate in PQ concrete or Hydraulically Bound Mixtures due to the potential for volume instability. However, its use may be permitted by the Overseeing Organisation if sufficient assurance of volume stability is provided.

Table 75.5 Specification for Highway Works (MCHW 1): Application of Secondary and Recycled Aggregates

Reproduced from Highways Agency, ***Design Manual for Roads and Bridges***, HD 35/04. © Crown Copyright 2004

75.6.8 Topsoil

Topsoil is not used as a structural engineering soil. This is due to the organic content of the soil, which can create stability problems and, if used close to the surface, regrowth. However, topsoil is part of the earthworks and, as it is generally encountered as the first and final operations, must be considered and included in the programme.

75.6.8.1 Topsoil removal (topsoil strip)

- In cut or at grade areas: topsoil is removed before other earthworks can commence.
- In fill areas: topsoil may be removed or, when an embankment is above a minimum height, left in place. The advantages of this are:
- reduced disposal if there is a surplus of topsoil;
- reduced imported fill if there is a deficit of suitable material in the works;
- preservation of archaeology;
- programme and cost benefits.

The minimum height for this must be agreed between designer and contractor. Below this, topsoil must be removed.



Figure 75.22 Typical low-lying peat landscape
Courtesy of Balfour Beatty

Topsoil which will be re-used in the works must be stored in topsoil bunds. To avoid damage to the soil, there is a height restriction to these, which engineers must confirm with the designer before proceeding.

Topsoil strip is usually carried out by a scraper box towed behind a track-type tractor. This is effective if the topsoil bund is close to the area being stripped. However, if these are further away, stripping with a 360° excavator and haulage by articulated dump truck (ADT) is more efficient.

75.6.8.2 Topsoil replacement (re-soiling)

After completion of structural earthworks, topsoil is replaced where the contract requires. Areas to be re-soiled will include verges, batterslopes, landscaping and areas not otherwise overlaid by, for example, bound/block pavement, bound or unbound decorative finishes, surface treatments or over filter drains.

Topsoil is spread by a wheeled or tracked 360° excavator or a light track-type tractor. Topsoil is not compacted.

Engineers should note that SHW has different material classes for topsoil existing on site (5A) and imported topsoil (5B). Different parameters and limits may apply to each. Each of these classes may be sub-divided so the composition may varied to suit the ecology of a specific area.

75.7 References

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Highways Agency (2004). *Design Manual for Roads and Bridges*, Volume 7, Section 1, Part 2, HD 35, Conservation and the use of Secondary and Recycled Materials. London: The Stationery Office.

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Reid, J. M., Czerewko, M. A. and Cripps, J. C. (2001). *Sulfate Specification for Structural Backfills* (TRL447). Crowthorne, Berkshire: Transport Research Laboratory.

CL:AIRE (2011). The Definition of Waste: Development Industry Code of Practice Version 2. [Available to download from www.claire.co.uk]

75.7.1 Useful websites

Building Research Establishment (BRE); www.bre.co.uk
British Standards Institution (BSI); www.bsigroup.com
Environment Agency; www.environment-agency.gov.uk
Highways Agency standards; www.dft.gov.uk/ha/standards
Transport Research Laboratory (TRL); www.trl.co.uk
WRAP; www.wrap.org.uk/index.html
CL:AIRE Code of Practice; www.claire.co.uk/index.php?option=com_content&view=article&id=210&Itemid=82

It is recommended this chapter is read in conjunction with

- *Chapter 69 Earthworks design principles*
- *Chapter 78 Procurement and specification*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 76

Issues for pavement design

Paul Coney Atkins, Warrington, UK
Peter Gilbert Atkins, Birmingham, UK
Reviewed by **Paul Fleming** Loughborough University, UK

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There have been significant developments in the design of pavements and trackbed for roads and railways in recent years. Latest design specifications take a more rational approach than the traditional recipe method based on experience, and offer potential rewards including the wider use of more sustainable marginal materials and reduction in construction thicknesses. Pavement design involves significant geotechnical engineering and the latest specifications place even greater importance on the assessment and selection of appropriate materials for the ground conditions present. This chapter gives a brief outline of good current design practice for UK roads and railways, and references are given to some of the key literature available for further reading. The focus is on the design methods developed for motorways and trunk roads (produced by the Highways Agency) and railways (Network Rail) as the majority of research carried out in the UK has been related to these. The importance of the assessment of the sub-grade and development of a design which suits the ground conditions present is emphasised.

76.1 Introduction

Transportation is a fundamental requirement of our lives and in the UK we are never far away from a road or railway. Considering this, it may seem surprising that pavement engineering, and pavement foundation design in particular, was relatively slow to develop as a technical aspect within civil engineering. Traditionally, pavements were built based on experience or precedent, and therefore construction has tended to be conservative or in some cases inadequate. Recent years have seen significant developments of a more theoretical framework for design. This approach has been driven largely by the change in the procurement method of many civil engineering projects towards public–private partnership, design and build, and design, finance and operate type contracts. These contracts, where the construction risk is transferred from the public to the private sector, have generated a greater desire for innovation to provide cost and environmental benefits by the use of alternative cheaper construction materials, and to minimise construction thicknesses whilst considering whole-life costing.

Pavement design practice involves several civil engineering disciplines including materials and structural engineering, hydraulics and geotechnics. The geotechnical engineer is required to assess the ground on which pavement will be constructed (sub-grade). The pavement engineer (for highways) or permanent-way engineer (for railways) will be required to determine requirements for the materials directly subject to traffic loading. The boundary between the responsibilities of the two disciplines varies in practice. Like building structures, the founding soil and the pavement foundation should be considered in unison (see Chapter 3 *A brief history of the development of geotechnical engineering*). Traditionally, for roads design, the boundary has been taken as the top of the capping; the geotechnical engineer is responsible for a suitable platform for the construction of the upper layers (Highways Agency, 2009b). The latest developments towards performance-based design have potentially changed the boundaries as the analytical design (which may be carried out by the pavement engineer) will require knowledge of the properties of each pavement layer. It is clear, however, that the geotechnical engineer's involvement should not be limited to the assessment of the sub-grade conditions. In particular, a geotechnical engineer is often best placed to judge the practicality of different foundation solutions – which is vital for successful construction. Design works best when the pavement engineer, geotechnical engineer, alignment engineer and drainage engineer work together to produce a suitable design solution.

This chapter concerns the design of pavement foundations. For the purpose of this chapter the term 'pavement foundation' is used to refer to the materials below the base course for roads (i.e. usually sub-base, capping and sub-grade) and materials below the sleepers for railways (i.e. usually the ballast, sub-grade and any intermediate layers). A variety of terminology is used to refer to the construction layers, with different terms being used for roads and railways. An attempt has been made to use consistent terminology: as examples, 'pavement foundation' is also used to mean 'trackbed', and 'trafficking' is used to refer to road or rail loading, as appropriate. The aim of this chapter is to provide an outline of good current design practice for UK roads and railways, and give references to some key texts for further information. It is not possible to provide a detailed description of pavement engineering theory and references are provided to some key literature. Pavement construction for other purposes, e.g. trams and airports, is not specifically discussed; however, many of the principles apply to any pavement foundations.

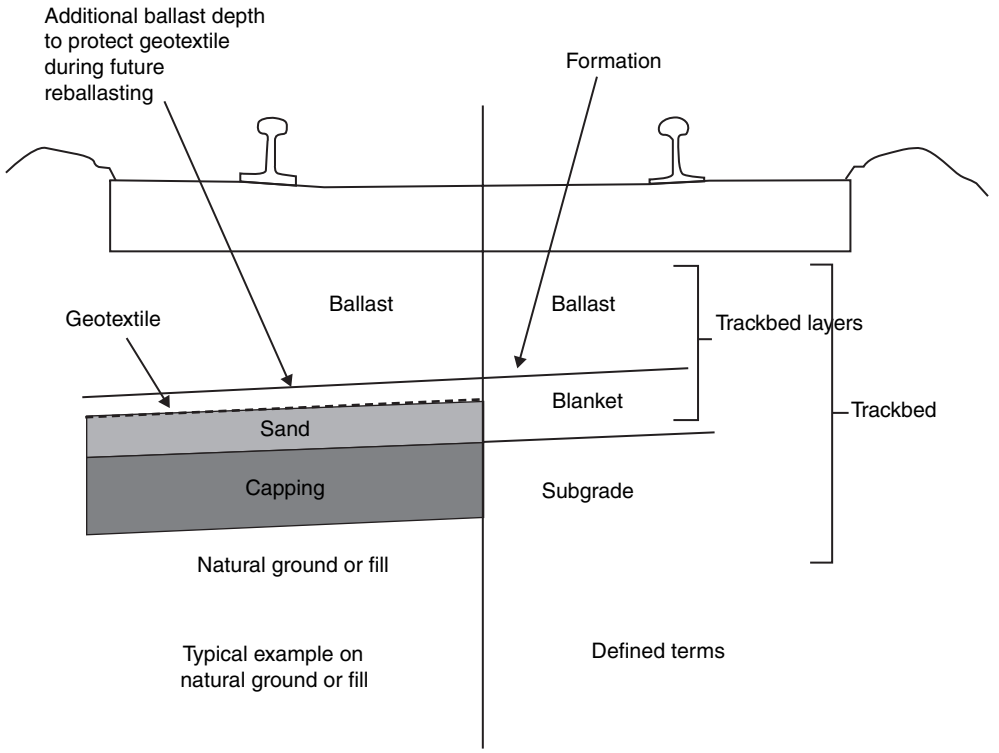


Figure 76.1 Railway trackbed terminology
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76.2 Purpose of pavement foundation

The basic terminology used for the various construction layers used in road and rail pavements is given in **Figures 76.1** and **76.2.**

Each pavement layer has different functions which are described in various sources (see Thom (2008) for roads and NR/SP/TRK/9039 (2005a); or Selig and Waters (1994) for railways). Trafficking applies a dynamic load to the pavement. This load will be distributed through the pavement to the pavement foundation. The fundamental roles of the pavement foundation are as follows:

- Provide a stable platform during construction to enable placement and compaction of the overlying pavement layers.
- Reduce the stress from traffic loading to an acceptable level for the sub-grade and prevent excessive deformation (in the short and long term).
- Maintain material properties under traffic loading. Worsening of material properties can occur either by attrition of the material particles or by migration of fines from adjacent layers.
- Resist lateral and longitudinal forces from traffic loading. This is most important for ballast which is required to maintain support to the sleepers.
- Allow drainage of water from the pavement layers to the drainage system. A separate but related topic is the drainage of the sub-grade which, in cuttings, is delivered by filter drains to avoid groundwater entering the pavement foundation.

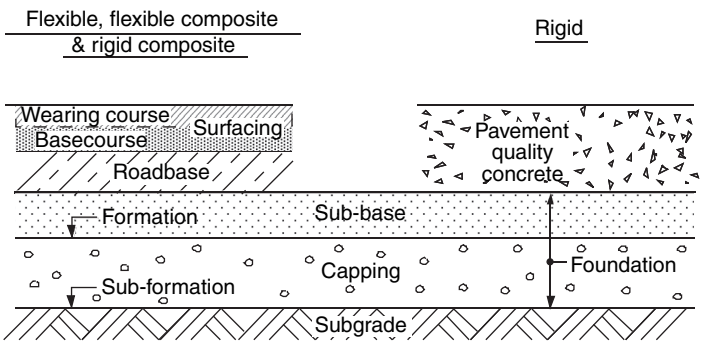


Figure 76.2 Road pavement layer terminology
Reproduced from The Highways Agency, *Design Manual for Roads and Bridges, HD 23/99* (Highways Agency, 1999) © Crown Copyright 1999

These functions are required both during construction and in service. For a pavement during construction, stresses in the foundation due to construction traffic are relatively high, although the number of stress repetitions is relatively low compared to in-service loading conditions. The extreme consequence of inadequate pavement foundations is rutting of the pavement and the underlying sub-grade. For the pavement foundation, rutting is more likely during construction and should be remedied appropriately (usually by excavation and replacement of the failed material), otherwise inadequate pavement performance is likely in the long term. Poor pavement

foundation performance commonly results in significantly higher long-term maintenance costs.

76.3 Pavement foundation theory

There is insufficient opportunity to explain the theory of pavement design in this chapter and sources such as Thom (2008) for roads and Selig and Waters (1994) for railways should be referred to for further detail. For road design, the Transport Research Laboratory (TRL) research of the 1960s to 1980s still provides much of the underlying logic that is followed today, therefore it is advisable to read TRL LR1132 (Powell *et al*., 1984) and refer to TRL LR889 (Black and Lister, 1979) for further details.

The upper bound layers (e.g. asphalt or concrete surfacing for a road, or the rails and sleepers of a railway) are designed to spread the traffic load, reducing it to a level that is tolerable for the pavement foundation. In turn, the foundation materials must be selected to perform under the design loading and reduce the stress passed to the sub-grade.

As described by Brown (1996), when a dynamic load is applied to an unbound material (soil), it will deform initially elastically (resilient deformation) and then plastically (permanent deformation), dependent on the stress path. The unloading action allows for only the elastic deformation to be recovered. The load cycles therefore cause an accumulation of permanent deformation, the magnitude of which depends on the size of the stresses applied, the number of load cycles and the stiffness and strength of the material. A typical shear stress–shear strain plot for an unbound material under dynamic loading is shown in **Figure 76.3**.

As shown in **Figure 76.4**, with increasing load applications at low levels of applied shear stress, the tendency is for plastic deformation to occur at an ever decreasing rate or actually cease altogether. At higher levels of shear stress, plastic deformation will increase at an increasing rate leading to large deformation or failure (Thom, 2008). The shear stress dividing these two conditions is called the threshold stress, which as a rough guide has been suggested as around half the undrained shear strength for a cohesive soil (Brown, 1996). The term 'shakedown limit' is also sometimes used in the available literature; it has a similar meaning to threshold stress but is arguably more complex.

The key point for the practising engineer is: if the stress applied remains below the threshold limit, then the effect of the applied stress will be to compact the soil rather than to soften it. Perhaps this logic is seen in nature by the evolution of chickens – whose feet are large enough to impart low stresses to the soil. Consequently, chickens stay clean even during rainy periods, and if you dig the ground below a chicken coop you will find the upper layer to be extremely compact.

The important parameters for pavement foundation design are therefore the elastic stiffness, some measure of plastic deformation and shear strength under repeated load. These parameters are difficult to determine in practice (and the ground will vary along the route, so many tests are required) which has led to significant simplifications in both the testing undertaken and the design.

76.4 Brief recent history of pavement foundation design

In the 1930s the California State Highways Department developed the California bearing ratio (CBR) as an index test for soils, specifically for the purpose of pavement design. The CBR test involves pushing a small plunger into soil at a constant rate and measuring the displacement. The CBR values obtained are a ratio determined by comparing the results to a 'standard' soil. The plunger penetration is due partly to elastic, plastic and shear deformation, and the CBR value obtained is an empirical value with no fundamental meaning. Despite these limitations, it has the advantages of using low-technology equipment, is quick to carry out and has become widely used. Various empirical design approaches have been developed around the world

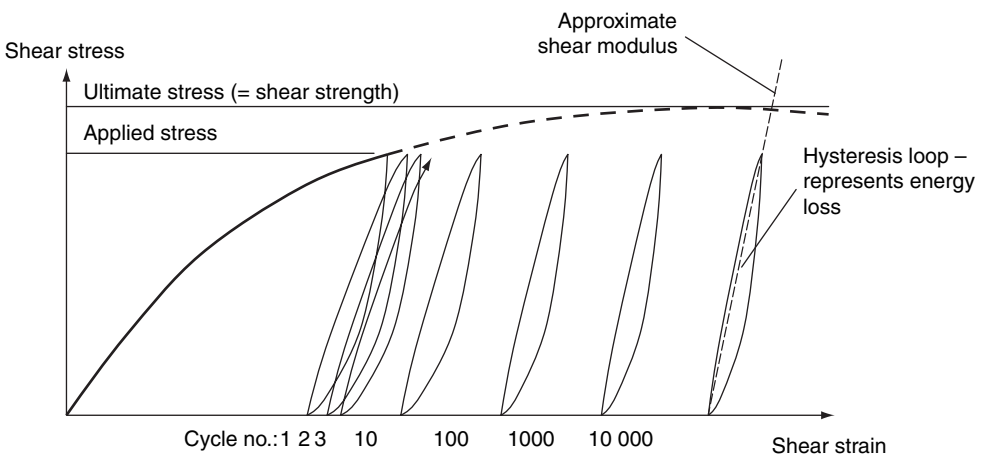


Figure 76.3 Idealised stress–strain behaviour of an unbound material under dynamic loading
Reproduced from Thom (2008)

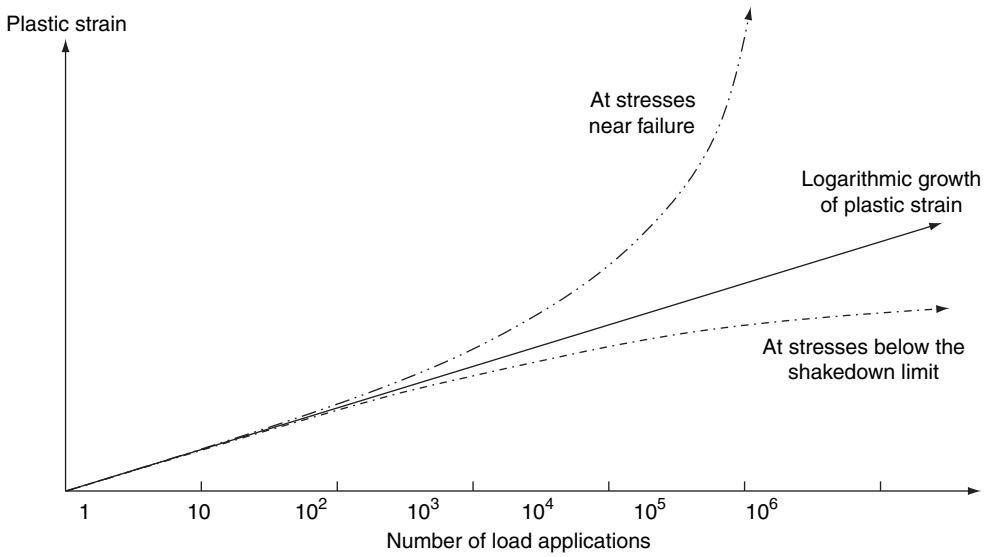


Figure 76.4 Plastic strain accumulation
Reproduced from Thom (2008)

to allow for local conditions using CBR values as a measure of stiffness and strength of a material.

The last few decades have brought important developments in the understanding of soil mechanics, testing procedures, materials research and design tools. There are now many possible test techniques which can help determine soil parameters and design can be undertaken analytically using computer models. However, the difficulty of determining the required soil parameters remains a problem; a practical testing technique has yet to be developed (see section 76.6 and Fleming *et al*., 2000). Design standards remain partially empirical and even though CBR testing is not actively encouraged by practitioners, designs standards still refer to CBR as an indirect measure of the stiffness of the sub-grade.

An underlying problem for pavement foundation design is that the stiffness of the soil prior to construction (i.e. during design) will in many soils be adversely affected by the construction process. The final stiffness of the soil will depend on many factors that the designer has only limited control over; these relate to the construction method employed, and how variations in ground and groundwater conditions are managed. Consequently, soil mechanics theory is only part of what goes into a pavement foundation design; potentially more important is engineering experience and for this reason the empirical CBR approach continues to have a place in practice.

76.5 Current design standards

Sections 76.2 and 76.3 describe how the pavement foundation must reduce the stress from traffic loading to an acceptable level (i.e. below the threshold stress) for the sub-grade. The design of the pavement foundation to perform this task is achieved by following industry standards and specifications, and more recently analytical design methods can also be used.

For UK roads and railways, several of the authorities responsible for the construction and maintenance of pavements have their own design standards. The two main standards are those produced by the Highways Agency and Network Rail, both of which are described in this section. Several local councils also have their own design standards for roads, which are mainly based on the now superseded Highways Agency HD 25/94 standard (Highways Agency, 2007) (or even its predecessor, Road Note 29), which is described below.

76.5.1 Highways Agency standards

The Highways Agency, responsible for motorways and trunk roads in England and Wales, present their design standards in their *Design Manual for Roads and Bridges*. This design standard is also used by Transport Scotland and the Northern Ireland Roads Service, who are responsible for motorways and trunk roads in their respective countries.

Until recently the Highways Agency design standard for pavement foundations was HD 25/94, which followed a method or recipe approach to ensure adequate performance. The standard was based on large-scale trials and long-term experience which is described in LR1132 (Powell *et al*. 1984). HD 25/94 included a table which relates the sub-grade design CBR value to the required thickness of sub-base and capping (see **Figure 76.5**). Options are given for a combined sub-base and capping layer, or a sub-base-only construction, i.e. a thinner but higher-quality foundation layer. The standard also provides good practical advice, such as restricting changes in foundation design along the route to prevent inconsistent ride quality. The standard was successfully used on many schemes; its main disadvantage is that no methodology was provided to allow for the different properties of foundation materials or traffic loading. An analytical approach which would take account of the

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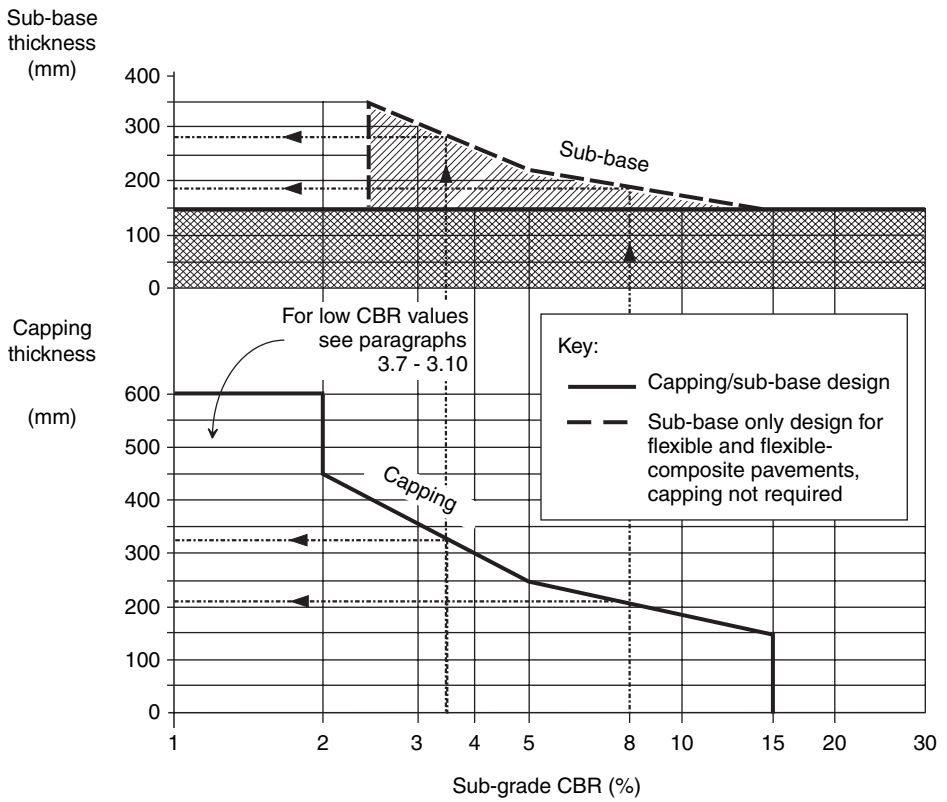


Figure 76.5 Capping and sub-base thickness design
Reproduced from The Highways Agency, *Design Manual for Roads and Bridges, HD 25/94*. © Crown Copyright 1994

engineering properties of the proposed foundation materials would potentially allow a wider range of materials to be used – and foundation thicknesses to be reduced.

In response to this need, the Highways Agency has recently introduced a new standard: interim advice note (IAN) 73/06 (Highways Agency, 2006a). This standard superseded HD 25/94 in February 2006 and was revised in February 2009 and re-issued as revision 1. Further revisions may be undertaken prior to its implementation as a non-interim standard. As well as allowing analytical design for the pavement foundations, IAN 73/06 also links to new guidance from the Highways Agency on the design of the upper road layers in HD 26/06, which permits a thinning of the upper layers depending on the stiffness of the foundation (for classes 3 and 4). However, the old HD 25/94 and previous references remain of value for the design theory and for good practical advice.

IAN 73/06 specifies four permissible foundation classes, defined by their long-term minimum stiffness at the top of the foundation level (termed foundation surface modulus):

- class 1–50 MPa;
- class 2–100 MPa (see example below);
- class 3–200 MPa;
- class 4–400 MPa.

Class 1 is acceptable only for minor roads, class 2 usually represents a granular pavement foundation, whilst classes 3 and 4 represent lime- or cement-stabilised (stiffer) foundations.

The construction target values are different to the long-term values, and allow for subsequent confinement for granular materials, or curing rate and cracking in the case of bound materials.

The foundation surface modulus is not to be confused with the stiffness of an individual layer (termed layer modulus). The foundation surface modulus is a measure of the 'composite' behaviour under a 300-mm-diameter loaded plate, and is affected by the stiffness and thickness of the foundation layers and the underlying sub-grade. Hence an unbound sub-base foundation of sufficient thickness and with a layer modulus of 150 MPa overlying a sub-grade with a layer modulus of 50 MPa (approximate CBR of 5%) would provide a class 2 foundation with a foundation surface modulus of 100 MPa. The field target for stiffness testing is, in this case, 80 MPa.

Traditionally, before the introduction of IAN 73/06, for an unbound pavement foundation, the CBR at the top of the capping was assumed to be in excess of 15% and the CBR at the top of the sub-base was assumed to be in excess of 30%. These values corresponded to the CBR of these individual layers and should not be compared to the foundation surface modulus, which, as stated above, is a composite behaviour of the various layers under a 300 mm diameter loaded plate. If the individual

foundation layers were sufficiently thick (much thicker than in normal practice) then the loading from the plate would be distributed entirely within the individual foundation layer. In this case, the foundation surface modulus measured by the plate would be the same as the layer modulus of that layer measured using the same test method.

It is important to clarify that capping cannot easily be tested for CBR because of the cause of the coarse grading of most capping materials. Static plate load testing can be used to determine an equivalent CBR, but is rarely carried out. An alternative approach is to gain an indication of the CBR using dynamic cone penetrometer testing, although this may not be appropriate in some materials.

As a rough guide, a CBR of 15% for capping may correspond to a layer modulus of around 100 MPa, and a CBR of 30% may correspond to a layer modulus of around 150 MPa. However, a note of caution is required as correlation of CBR to stiffness is not well defined for higher stiffness materials. It is also of note that IAN 73/06 seems to suggest that capping may have a stiffness of only 75 MPa which may correspond to a CBR of less than 15% (depending on the correlation used). This does not correlate well with traditionally used values; however, it may reflect the intention that capping is a locally available lower-cost material and the IAN does give the option of carrying out a performance design using a higher stiffness. Experience shows that it is important that the material selected to use as a capping has a low fines content and is from a source that can deliver a layer of fill of reasonably consistent nature. If this is achieved, then the result should be a layer of stiffness suitable for a capping layer.

Two design processes are encompassed in IAN 73/06: a 'performance' approach and a more traditional 'restricted' approach. The restricted design involves determination of the pavement foundation thicknesses based on the sub-grade stiffness using a set of charts. The 'performance approach' design methodology is presented in the flow chart shown in **Figure 76.6**. This allows many more theoretical permutations and combinations of materials and design thickness than using the restricted method or the previous HD 25/94 standard.

For the performance design, the designer initially chooses appropriate layer stiffness values for the sub-grade and foundation layers, and determines the design based either on some charts provided for some limited design permutations or, more usually, from the equations provided with the aid of a computer model. The analytical model is based on multi-layer linear elastic analysis, which involves a number of simplifications and assumptions from the actual conditions which are described in Thom (2008). Testing and assessment of both the sub-grade and foundations materials are required to determine appropriate design stiffness values and the IAN gives some suggested test techniques.

After the design has been completed, a series of demonstration trials are required. The philosophy is that the demonstration trials are situated on various sub-grade conditions, which reflect the range of conditions that will be encountered. The trials are to be constructed using the methods and materials to be used in the main works. Testing and monitoring of the trials are carried out to validate the design, primarily relating to the density and stiffness of the placed layers, and satisfactory resistance to rutting under traffic loading. A key development in the new approach is inclusion within the trials and main works of the use of *in situ* dynamic plate tests (light-weight deflectometer and falling-weight deflectometer, described in section 76.6) to evaluate the 'as built' stiffness to prove compliance with the required foundation surface modulus for the intended foundation class. The foundation surface modulus values to be achieved during the trials and works are those expected in the short term and are not a direct method for determining the stiffness in the long term once the foundation is covered by the overlying pavement layers (Edwards and Fleming, 2009).

There has been a fair amount of debate on the benefits of IAN 73 and some merits and limitations are provided in Fleming *et al*. (2008). As stated above, the merits are the potential for the use of a wider range of materials – including the reuse of materials giving environmental and cost benefits. The demonstration trials and testing during the works also offer better assurance of performance. However, a number of limitations have been suggested, including very conservative thickness requirements for the 'restricted design' and extensive costly testing for the performance design approach, which for some designs would appear inappropriate. A fundamental assumption remains that short-term behaviour will represent long-term performance. Carrying out trials which represent all the appropriate sub-grade materials in all conditions which will influence performance (such as weather wetting/drying effects) in advance of the works, will often not be realistic and a degree of judgement will be required. Encountering alternative conditions on-site may require additional trials, which are likely to have a knock-on effect to the programme and to the cost.

Whilst the move to a more satisfactory analytical design-based method is welcome, significant development is required to develop a practical design approach. It should also not be forgotten that the most important factor in pavement foundation design (whichever approach is taken) is to undertake an appropriate assessment of the ground and groundwater conditions, and to develop a design which works for these conditions and the likely method of construction (see section 76.6 below). In this regard it is useful to refer to HA 44/91 *Earthworks – Design and Preparation of Contract Documents* (Highways Agency, 1991), because Chapter 10 provides some useful background on the design and specification of pavement foundations. HA 44/91 was produced prior to HD 25/94 and therefore some of the information has now been superseded. However, several of the key principles of pavement foundation design are explained briefly (with cross reference to LR1132) (Powell *et al*., 1984) and remain relevant, e.g. purpose of foundation layers (10.1 and 10.2) and discussion on sub-grade CBR values (10.9 to 10.23).

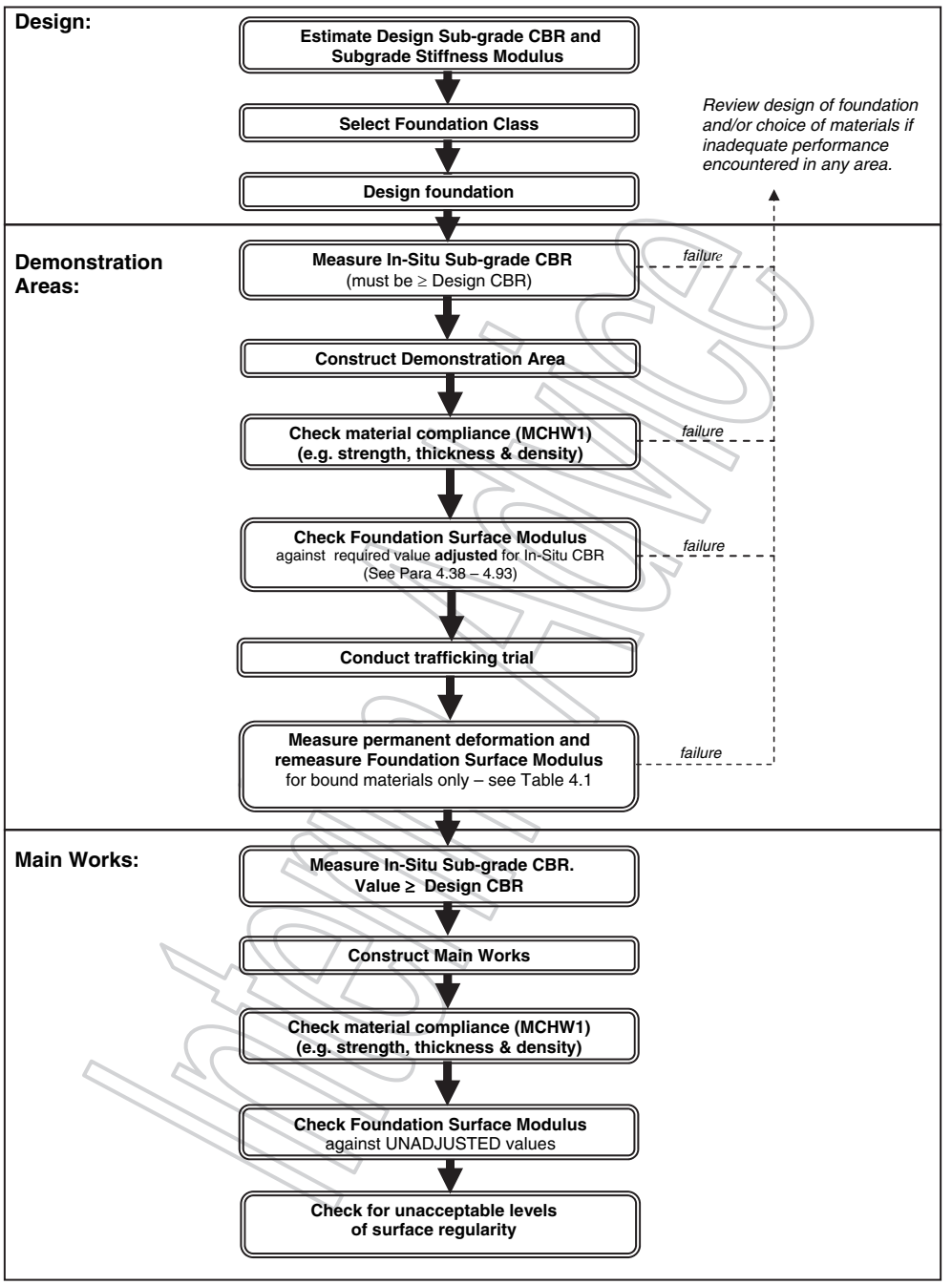


Figure 76.6 Summary flowchart of IAN 73/06 performance design
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76.5.2 Network Rail standards

There are several different Network Rail standards which concern trackbed foundation design, the most relevant of which are given below:

- RT/CE/S/101 (1997) track design requirements;
- NR/SP/TRK/102 (2002) track construction standards;
- NR/SP/TRK/9039 (2005a) track design requirements;
- NR/SP/TRK/9006 (2005b) lineside drainage.

RT/CE/S/101 gives the minimum engineering requirements and principles for the design of track construction. It is a high-level specification which refers to other more detailed Network Rail standards. For the design of the trackbed foundation it

refers to RT/CE/C/039 (now superseded by NR/SP/TRK/9039) and for the design of the ballast it refers to NR/SP/TRK/102.

NR/SP/TRK/102 gives standards for new or re-laid track and includes details such as minimum ballast depths as well as specifications for rails and sleepers. Minimum ballast depths are given depending on the category of the track (based on train speed and track usage), rail and sleeper types, and range between 200 and 300 mm. This is based on experience and on minimum depths to allow maintenance, including tamping. For the design of the sub-grade and formation, the specification refers to RT/CE/C/039.

NR/SP/TRK/9039 gives recommendations for the design of trackbed treatments. It describes the various potential trackbed problems and suggests possible investigation techniques. Advice is provided on trackbed thickness and a chart illustrates the required thickness of trackbed layers, which varies depending on the undrained sub-grade modulus and railway type. This is based on a combination of empirical data and multi-layer elastic theory, and is in general agreement with German railway standards. Sketches of standard solutions are also provided for various criteria. The document states that its remit applies to track renewals and remodelling schemes, but not to new lines. It is not clear why this distinction is made since parts of the document (e.g. table 3) suggest that it may also apply to new track. Specification for new track design is not found elsewhere in Network Rail standards.

In practice, the Network Rail standards for trackbed design are likely to continue to develop and change with further research to encompass a broader range of design cases and ground conditions. At present, the designer needs to have a good understanding of the underlying theory and the research undertaken, particularly by British Rail in the 1950s to 1980s (as presented by Selig and Waters, 1994) in order to determine an overall design in conjunction with the advice given in the current standards.

76.6 Sub-grade assessment

The most important factor in pavement and track foundation design is to undertake an appropriate assessment of the sub-grade and groundwater conditions, and to develop a design which is appropriate for these conditions. For the purposes of the assessment, the sub-grade should be considered to be the layer or layers of ground below the foundation and not just the sub-formation surface on which the foundation will be founded. This is because the material properties of the sub-grade may change with depth and along the route, and in particular the stiffness may reduce. The depth of sub-grade which needs to be considered depends on the depth to which the majority of the stress from traffic will be dissipated during construction and in service. The influence of traffic-induced stress can extend to as much as 5 m below the bottom of the sleepers for railways (Brown, 1996). However, in the majority of circumstances, stress is usually considered to be distributed to around 1.5 m below the base of the foundation. This is supported by IAN73/06 (Highways Agency, 2006a), which suggests a rigid layer is modelled for analytical design 1.5 m below the foundation.

Unlike the other foundation layers, the sub-grade is variable and its performance is influenced by many factors. The factors that apply are site-specific but are likely to include some of the following, and ground investigations should be designed to address these issues:

- soil type, grading (especially for borderline cohesive/granular soils);
- variability within the soil type;
- permeability and stiffness;
- horizontal and vertical geological variability;
- soil fabric (e.g. lamination);
- likely presence of hard/soft spots;
- groundwater conditions and drainage (prior to excavation);
- conditions that encourage elevation of pore water pressures during construction (e.g. silts/fine sand or a thin clay layer over sand);
- topography – e.g. transition zones from cut to fill;
- construction procedures adopted and skill of the site foreman to implement these;
- construction season, timing of drainage installation, exposure time of sub-grade;
- quality control systems.

It is a common misconception that the sub-grade stiffness is the only factor which needs to be considered; it is just one item on a long list. To illustrate this issue, consider projects where the sub-grade consists of glacial till which typically comprises a sandy gravelly clay/silt with cobbles and boulders, and has a relatively low fines content. The sub-grade usually has a relatively high stiffness with design CBR values – typically varying between 3 and 6%. However, due to the grading of the material which includes a low, but dominant, fines fraction, the engineering properties of the material can be very sensitive to changes in moisture content. The permeability of the material can also be reasonably high. If groundwater is high (such as in cuttings) and is not adequately controlled, the sub-grade stiffness can deteriorate rapidly on excavation.

In determining the design sub-grade stiffness, consideration should be given to both the short-term (during construction) and long-term (in service) conditions. The stiffness of the sub-grade is dependent on the moisture content, particularly in fine-grained soils. If an excavation for a road pavement is undertaken during the summer in dry conditions, the stiffness recorded *in situ* may be high, but following construction of the road in service, the moisture content is likely to increase to the equilibrium moisture content – resulting in a reduction in stiffness. Conversely, construction in wet conditions in winter is likely to find lower initial stiffness sub-grade conditions during construction and these may then deteriorate to below equilibrium moisture content stiffness. In this case the material will never recover to a significantly higher stiffness in service.

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There are many possible exploratory and testing methods to investigate the sub-grade, many of which are discussed in Thom (2008), NR/SP/TRK/9039 and IAN73/06, and are not repeated here. As with all ground investigations, the methods should be designed specifically for the ground conditions present. Practical experience shows that it is better for the ground investigation to obtain a large number of relatively simple tests along the route (such as index properties, grading and a simple measure of shear strength) than simply to focus on a small number of advanced laboratory tests to determine stiffness.

Despite significant work carried out in recent years there is no one simple method to determine sub-grade stiffness, either *in situ* or in the laboratory. IAN73/06 suggests several testing methods and specifies the use of the falling-weight deflectometer (FWD) and/or the light-weight deflectometer (LWD) during construction for performance-based design. However, a review of a wide body of data from various LWD development trials on capping materials has shown that the stiffness values determined can vary significantly. This is complicated further by the large number of factors from the material properties and material states that can influence this variability (Lambert, 2007). Similar variability is typical for other *in situ* tests and laboratory testing. It is recommended that a range of testing is carried out to allow correlation between the various methods and a degree of engineering judgement, based on experience, is usually required to determine design stiffnesses.

The requirement to determine the long-term stiffness is also problematic. The long-term stiffness should be based on the equilibrium moisture content – which is the maximum moisture content that is likely to be present during the operation of the pavement. It is possible to undertake testing in the laboratory over a range of moisture contents to reflect the conditions which are likely to be experienced during construction and operation of the pavement. The equilibrium moisture content can also be determined from measurements on a suction plate (Black and Lister, 1979). However, this is not widely undertaken in practice due to uncertainty in the equilibrium moisture content and difficulties of the testing. As a worst case, testing can be carried out on saturated samples, with a common (but unsatisfactory) example being soaked CBR testing. However, this may be too severe and can result in very low results which may be unrealistic. In response to these difficulties, the Highways Agency have produced a table which provides an estimation of the sub-grade CBR value at equilibrium moisture content for various soils (see **Table 76.1**).

These estimates are mainly used for clay soils for which the estimated CBR depends on the plasticity index, the water table, the construction conditions and the thickness of the pavement. **Table 76.1** was originally published in LR1132 (Powell *et al*., 1984) and has been subsequently included in HD 25/94 and IAN73/06 revision 0. Only a brief extract is included in IAN73/06 revision 1, but reference is given to LR1132, which remains an important document for designers to understand the underlying logic and use of this table. The table is widely used in practice to estimate the long-term CBR values for clay sub-grades and has been found to give reasonable results. It

		High water table construction conditions						Low water table construction conditions					
	Plasticity index	Poor		Average		Good		Poor		Average		Good	
Soil	%	Thin	Thick	Thin	Thick	Thin	Thick	Thin	Thick	Thin	Thick	Thin	Thick
Heavy clay	70	1.5	2	2	2	2	2	1.5	2	2	2	2	2.5
	60	1.5	2	2	2	2	2.5	1.5	2	2	2	2	2.5
	50	1.5	2	2	2.5	2	2.5	2	2	2	2.5	2	2.5
	40	2	2.5	2.5	3	2.5	3	2.5	2.5	3	3	3	3.5
Silty clay	30	2.5	3.5	3	4	3.5	5	3	3.5	4	4	4	6
Sandy clay	20	2.5	4	4	5	4.5	7	3	4	5	6	6	8
	10	1.5	3.5	3	6	3.5	7	2.5	4	4.5	7	3	>8
Silt*	–	1	1	1	1	2	2	1	1	2	2	2	2
Sand (poorly graded)	–	20											
Sand (well graded)	–	40											
Sandy gravel (well graded)	–	60											

* Estimated, assuming some probability of material saturating.
Notes: 1. A high water table is 300 mm below formation or sub-formation.
2. A low water table is 1000 mm below formation or sub-formation.
3. A thick layered construction is a depth to sub-grade of 1200 mm.
4. A thin layered construction is a depth to sub-grade of 300 mm.

Table 76.1 Equilibrium sub-grade CBR estimation
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needs to be used with caution as there are a number of complicating factors, as follows:

- Plasticity index is determined using samples of material with a grading of less than 425 μm. For well-graded soils which contain both a granular and cohesive fraction such as glacial till, the stiffness is affected by both fractions and the CBR values determined from **Table 76.1** may not be representative for material with a low-fines content.
- Plasticity index results are often variable and the designer must choose a representative value or range of values.
- For low-plasticity soils with a plasticity index of 5 to 10%, the designer must judge if the sub-grade will act as a silt (CBR of 1 to 2%) or a clay (CBR of 2.5 to 8%).
- It is also important to realise that if construction conditions are poor and groundwater is not controlled, the stiffness of fine-grained soil can easily fall below the values in **Table 76.1** and will not fully recover.
- The table also gives CBR values for granular materials which are relatively high and the actual stiffness will depend on the *in situ* density and pore water pressure. However, in practice, the requirement is simply to check if sub-grade remains above a CBR of 15%.

To obtain confirmation of the proposed design, a CBR investigation of an existing pavement of similar construction could be carried out. This approach is not often followed, and the few results obtained tend to show that the standard values are reasonable.

It is important for the designer to check that the original shear strength of the soil is sufficient to deliver the expected equilibrium moisture content CBR value, as construction will usually lead only to a softening below the undisturbed condition. In this regard, a commonly used approach is to apply an approximation for clay soils of CBR = C_u/23 (Black and Lister, 1979), where C_u is the undrained shear strength.

76.7 Other design issues

Once the ground conditions are confirmed, the majority of the design effort is often focused on providing a sufficient construction thickness to prevent (i) sub-grade strength failure; and (ii) excessive sub-grade deformation. Whilst these two failure mechanisms are obviously important, the design should also consider a range of issues as follows.

76.7.1 Materials

The design should carefully consider and specify the materials to be used in the pavement foundation. In particular, the materials specified should suit the ground and groundwater conditions present at the site. This is usually achieved by reference to material types given in standard specifications such as the *Manual of Contract Documents for Highways Works* (MCHW) (Highways Agency, 2009b) and further advice is given in BS 6031: *Code of Practice for Earthworks* (2009). The following examples illustrate a few of the most common issues to be considered.

IAN73/06 offers a reduction in foundation thickness, should a single sub-base layer be used rather than a two-layer sub-base on a capping due to the higher stiffness of the higher-quality sub-base layer. Contractors will generally prefer a sub-base-only solution based on an economic assessment of material costs. In practice, however, for many sites the use of capping is advisable to manage the sub-grade conditions as it is coarser and quicker to construct. This is particularly true where sub-grade conditions are susceptible to changes in moisture content and the capping provides a platform for the construction of the sub-base layer (which is finer and requires more precise construction). IAN73/06 also offers benefits from providing stiffer lime- or cement-stabilised foundations. In practice, it is important to consider whether the ground conditions are well suited to the use of thin sub-base only or stabilised foundation solutions and whether any additional measures (e.g. temporary drainage or modification of stabilisation mix) may be necessary to avoid problems with moisture-susceptible sub-grades.

IAN73/06 performance-based design, together with recent revisions of the MCHW, has opened up the possibility of a wide range of materials to be used in the foundation. This allows a greater potential for the re-use of locally available materials. The capping layer, in particular, is intended to be a locally available material, and the new specification provides greater potential for this to be realised.

The materials need to be selected to ensure that they do not deteriorate under trafficking. This is a common problem for railways, as the ballast is highly loaded due to its position directly under the sleeper, the heavy loading from trains, and mechanical maintenance. The ballast will break down with time and eventually become filled with fines. Once pore water pressures cannot dissipate under loading, track geometry will be affected and the occurrence of wet spots will become a common feature. This is described in NR/SP/TRK/9039 (2005a).

Geotextiles are often incorporated in pavement foundations. The Network Rail standards (see NR/SP/TRK/9039) specify the use of a separating geotextile or geocomposite where there are sub-grade erosion (pumping) issues, which may be used in conjunction with blanketing sand. The use of separating geotextiles is relatively uncommon for road pavements, as sub-grade pumping is usually less of an issue (due to the lower traffic loading and usually better drainage) although it may be considered for particular conditions. Geogrids may be incorporated in pavement foundations, particularly where the sub-grade has a very low stiffness.

Near-surface materials should not be susceptible to frost, and guidance on frost protection is given in IAN73/06. For routine cases, all material within 450 mm of the road surface should be non-frost-susceptible, although this may be reduced if the frost index is low. The frost index is a measure of the severity of a period of cold weather and provides a means of assessing likely penetration of frost into a road.

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Environmental issues should be carefully considered when designing pavement foundations. Considering that some 25–30% of all materials extracted from the ground finish up in highways pavements (Thom, 2008), the environmental impact of pavements is clearly significant. Innovation to limit the environmental impact and reduce cost is a key aspect of design. The design life is an important consideration for the upper bound layers, but is not an easy consideration for pavement foundations. For these, it is usually desirable to use locally available materials to reduce transport costs during construction. Recycled materials are also often used, particularly for capping layers in road pavements, and the Highways Agency has made significant steps in recent years to encourage recycling (e.g. the addition of a wider range of recycled aggregates to table 6/1 of the MCHW, and IAN 73/06, which permits a wide range of construction materials).

76.7.2 Drainage

Adequate drainage is of vital importance to the performance of the pavement. IAN 73/06 gives some advice on drainage and states that it should be provided during both the construction and operation of the pavement.

During construction, the sub-grade should be protected to avoid deterioration due to surface water and groundwater. Failure to adopt suitable measures can result in the requirement for extensive additional excavation and replacement below sub-formation, which can significantly add to the construction costs. For moisture-susceptible sub-grades and cutting areas where groundwater is high, deep drainage is often required before the verge to draw down groundwater prior to excavation to sub-formation level. Additional drainage in the centre of the pavement or cross-drains may also be required is some cases. Open excavations may need to be limited, particularly in moisture-susceptible soils. If a drainage layer is present at the base of the pavement foundation, the sub-formation should be sloped so that water can be shed to the sides – to prevent ponding during construction, or entrapment of water in the completed pavement. If the upper pavement foundation layers are also intended to act as drainage layers (e.g. ballast or type 1 sub-base), their base may also need to be sloped.

Drainage during the operation of the pavement should be designed to maintain groundwater level below sub-formation level (earthworks drainage) and to drain water entering the pavement foundation by seepage through the upper pavement layers (pavement drainage). The pavement foundation layers should be constructed with a fall towards the drainage. This is a particularly important consideration for widening projects and a fall should be maintained through the existing and proposed pavement foundation layers to the drains. For these schemes, this can be the governing factor that determines the thickness of the pavement foundation layers.

76.8 Construction specification

The design needs to be adequately presented using drawings and/or a specification to allow construction. Unless the scheme is small and the ground conditions are very well understood, it is likely that the conditions in some areas of the site will be different to that indicated by the ground investigation information. This presents a risk of delays on-site whilst the design is amended, or the risk that the construction will be inadequate for the ground conditions present. This risk can be addressed by adopting a conservative design which will be adequate for most conditions likely to be present, preferably by providing flexibility in the design. The flexibility, where variable site conditions are encountered during construction (or limited pre-construction site data is available), is embodied in the 'observational approach' (Nicholson *et al.*, 1999). A similar methodology has been developed for a number of design-and-build schemes (Gilbert, 2004); it has proved appropriate for large highway foundation areas and allows swift construction. This was achieved by including full details of the design assumptions on the drawings and specification, including:

- pavement foundation layer specifications and thicknesses;
- design sub-grade CBR value or stiffness;
- design sub-grade ground conditions (e.g. clay, sand);
- design groundwater level, if relevant.

A defined regime was also specified, comprising site inspection coupled with rapid *in situ* testing methods to confirm the design assumptions and identify locations where the conditions differ from those expected. Clear procedures were also developed to enable changes to be made on-site, based on the actual conditions encountered. This allows the designer to make realistic predictions of a range of pavement foundation requirements. However, for this approach to work, trust and cooperation are required between all parties (client, contractor and designer) to allow the optimum foundation for the scheme to be built.

The approach given in IAN 73/06 performance-based design is similar to Gilbert's methodology in that the design assumptions must be clearly stated and rapid *in situ* testing methods are used during construction to confirm the design. The potential drawback is that there is no simple, quick procedure to enable changes to be made on-site based on the actual conditions encountered, and as stated in section 76.5.1, additional trials could be required which are likely to have a knock-on effect to programme and cost.

In addition to the issues mentioned above, the drawings and specification should address a number of other issues for the construction, some of which are listed below:

- permanent and temporary drainage;
- measures to protect the sub-grade during construction;
- transition zones, particularly cut/fill intersections, should be given special attention to avoid significant variations in pavement stiffness;
- consideration of consistent ride quality (i.e. minimise changes in foundation along the route).

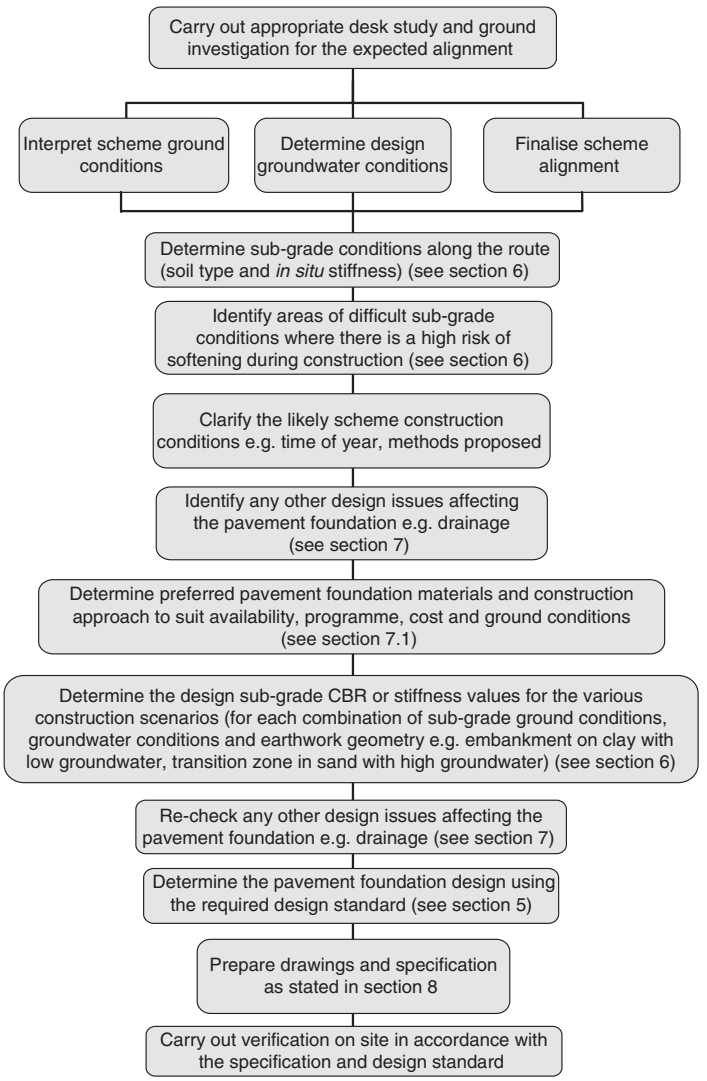


Figure 76.7 Pavement foundation design summary flow chart

76.9 Conclusion

A flow chart summarising the steps required in the design of pavement foundations, described in sections 76.5 to 76.8 above, is given in **Figure 76.7**.

There have been significant recent developments in pavement foundation design, particularly for roads. Whatever method is used for design, it should be remembered that the most important factor is to undertake an appropriate assessment of the sub-grade and groundwater conditions and to develop a design which works for these conditions.

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76.10.1 Useful websites

Design Manual for Roads and Bridges, Highways Agency; www.dft.gov.uk/ha/standards/dmrb

It is recommended this chapter is read in conjunction with

- Chapter 75 *Earthworks material specification, compaction and control*
- Chapter 99 *Materials and material testing for foundations*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Section 8: Construction processes

Section editor: **Tony P. Suckling**

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Chapter 77

Introduction to Section 8

Tony P. Suckling Balfour Beatty Ground Engineering, Basingstoke, UK

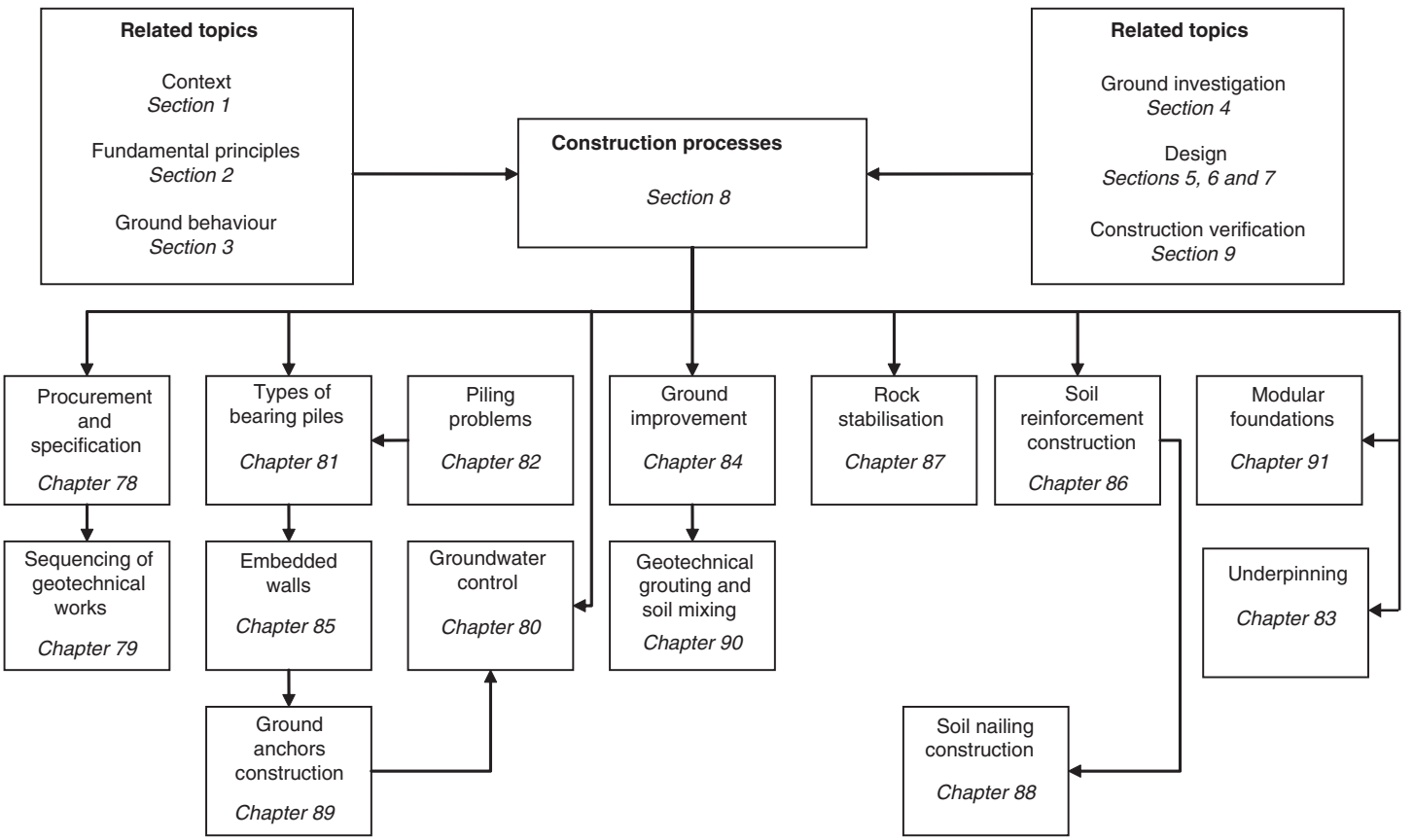


Figure 77.1 Layout of chapters in Section 8

Figure 77.1 outlines the layout and contents of Section 8 *Construction processes*.

Geotechnical engineering is different to most other forms of civil engineering in that it does not deal with man-made materials whose properties and behaviour can be reliably dictated and predicted. Man-made materials such as concrete and steel have been rigorously tested in the past and their engineering properties perfected away from the site environment. A concrete or steel member will perform the same way on different construction projects, provided the same materials are used and the same level of construction workmanship applied.

The ground is not normally man-made and so every construction project will have a unique set of ground conditions, including the groundwater regime. It is this unique combination of ground conditions and the applied load from buildings and structures, which makes geotechnical engineering so fascinating to those who work in the industry, and so frustrating to those who do not. It is no surprise that the biggest risks for new developments are often associated with the ground.

The physical characteristics of the ground need to be investigated for every new development. Geotechnical engineers use a combination of experience, empiricism and idealised engineering models to try to best predict the ground behaviour and its unique interaction with a new building or structure. This is as important for the practical aspects associated with construction as it is for design.

Ground behaviour is implicitly linked to the geotechnical construction process, for example a bored pile of a certain length will have a different load capacity to a driven pile of the same length. Consequently in geotechnical engineering

the design cannot and must not be divorced from the construction process. It is common in geotechnical engineering for specialist sub-contractors to offer both a design and construction service because of this. If the design is carried out by a party different from that undertaking the construction then the geotechnical designer must communicate the needs of the design to the contractor, and the contractor must communicate the construction process to the designer, so that both parties can ensure that design and construction are compatible with each other. This lack of compatibility is frequently the cause of disputes.

There are many different geotechnical construction processes, and even the same process can be undertaken in many different ways. Any process may be perfectly safe on one site, but have a very high level of risk on another. A process may also not be technically or practically possible in some ground conditions or site environments. It is recommended that during design development and the tender process for any new development, experienced geotechnical designers and experienced geotechnical contractors are brought together to ensure that the proposals are both viable and technically sound. It is essential that the final process is monitored during construction so that the impact on the design of any unforeseen changes in ground conditions can be appropriately managed.

For those who are not experienced in geotechnical engineering the best advice is to get yourself specialist advice. There are many geotechnical consultancies, specialist contractors and trade organisations available to provide expert advice and services. A failure to take specialist advice is fraught with danger due to the reasons described above. Beware of the World Wide Web; although there is a plethora of information available on the internet it should be used with caution due to the unique nature of every ground-engineering project.

Section 8 *Construction processes* describes the most common geotechnical processes in use today. Note that there are some very specialist processes, which are not covered here, for example ground freezing. The processes that are covered are introductions for the reader but they are absolutely not a replacement for specialist advice. This section does not consider design, which is covered in other sections. The exception is Chapter 80 *Groundwater control*, as the design of such systems is not described elsewhere in this manual.

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Chapter 78

Procurement and specification

Tony P. Suckling Balfour Beatty Ground Engineering, Basingstoke, UK

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Each construction project will have a unique set of ground conditions to be considered, and it is this unique combination of ground conditions and future applied loadings which presents the most risk for new developments. Consequently the procurement process which defines the scope of work, allocates responsibilities and risk, and chooses the contractor to do the work, is critically important. The specification which defines the minimum standard of the work is a key tool in ensuring a successful outcome. Tender documents and submissions need to be clear and non-contradictory, and the tender negotiation process must be managed to ensure that the chosen contractor has the necessary skills and tools to undertake the work in the prevailing ground conditions without unduly affecting the design. There are many forms of contract and sub-contract used in geotechnical construction, but none has been designed specifically for such specialist works. A common problem with geotechnical construction is identifying who is responsible at interfaces between different elements of work. Works at such interfaces must be clearly allocated in the contract documents, especially when one party will be relying upon another to supply a service. Enabling works can have a significant impact on geotechnical works.

78.1 Introduction

Each construction project will have a unique set of ground conditions to be considered, including the groundwater regime. It is this unique combination of ground conditions and future applied loadings (from the building or structure) which present the most risk for new developments. Consequently the procurement process which defines the scope of work, allocates responsibilities and risk, and chooses the contractor to do the work, is critically important. The specification which defines the minimum requirements for the work is a key tool in ensuring a successful outcome.

78.2 Procurement

Ground behaviour is implicitly linked to the geotechnical construction process being used. Consequently in geotechnical engineering, design cannot (and must not) be divorced from the construction process being used. Because of this, it is common in geotechnical engineering for specialist sub-contractors to offer both a design and construction service. If the design and the construction are being carried out by different parties, the geotechnical designer must communicate the needs of the design to the contractor, and the contractor must communicate the construction process being used to the designer. This is to ensure that design and construction are compatible with each other. Lack of compatibility and appropriate communication is frequently the cause of disputes.

There are various forms of procurement and employers have their own preferences. In general, the forms of procurement are:

- competitive tender – cheapest price;
- competitive tender – best value;
- preferred supplier;
- partnering arrangement.

After a competitive tender process, ‘cheapest price wins’ is the most common method of procurement in construction. However, most disputes over geotechnical works are associated with this method due to the inherent risks and uncertainties of working with the ground. Most experienced geotechnical engineers are not keen on this system; they can foresee that ground-related problems will necessarily increase their costs, and so are unlikely to offer the cheapest price and be successful.

The disadvantages of cheapest-price competitive tendering for geotechnical works can be overcome if the tenders are not assessed on cost alone. Other aspects to be considered may include construction experience in the appropriate ground conditions, design experience in the appropriate ground conditions, safety records, quality systems, plant reliability, and the ability to offer other solutions if unforeseen conditions were to arise. This is known as ‘best value’.

Some employers have their own preferred supply chain, which comprises organisations that meet criteria that they have decided are important. These criteria vary and may include safety records, quality assurance (see Chapter 93 *Quality assurance*), relevant experience, design capability and environmental systems. The scope of the works is identified and the preferred supplier provides the works. The costs for doing the work may be agreed by a competitive tendering process or by direct negotiation.

Larger employers who undertake construction works on a regular basis may enter into a partnering arrangement with a preferred supplier. Again, the scope of the works is identified and the preferred supplier provides the works. The costs for doing

the work are likely to have been agreed by negotiation and if the scope of the work changes, the cost implications and associated responsibilities will already have been identified in the arrangement. The advantage of this form of procurement is that the risks are normally identified and allocated early in the process.

Chapman (2008) states that geotechnical mitigation is better than the management of risks by contingency, and hence argues strongly against awarding a tender on cost rather than on the quality of the design and construction.

A modern approach for large projects has been to assess tenders on a declared split between cost and quality. This may be, say, 80% on cost and 20% on quality, but for complex projects it has been known to be 20% on cost and 80% on quality. However, this process of assessing the tenders is undermined if the assessment of quality is not considered in enough detail to prevent all the tenderers from gaining a similar score.

78.2.1 Tender process

The employer, i.e. the body that is funding the work, normally needs to appoint a professional team that comprises someone on their behalf to look after the technical aspects of the work (normally the engineer) and someone to look after the commercial aspects of the work (normally the quantity surveyor), as a minimum. A document describing the employer's requirements is produced and normally the engineer then designs the building or structure to meet these requirements.

Bills of quantities are then prepared, normally in conformity with a standard method of measurement, approved for use with the relevant form of contract. However, some of the provisions of these standard methods are not well suited to geotechnical processes and need to be amended. The type and number of amendments vary according to the specific requirements of the geotechnical work, the ground conditions, and for each type of construction process. Quantities of geotechnical work should always be billed as 'provisional' and measured and valued as 'executed', since it is only in exceptional circumstances that the ground is homogenous enough that the design quantities will be precise. There will normally be variations or additions to the design quantities, which should be anticipated and allowed for.

The employer's requirements: the bills of quantities, a description of the works (normally as drawings), a specification describing the minimum standard of work that will be acceptable, contract conditions and information on the expected ground conditions are then provided to prospective contractors with an invitation to tender.

The contractor will normally provide a tender, but it is often not fully in accordance with the original documents. Technically this counter-offer may be for a different standard of work, or a different construction process. Commercially, this counter-offer may be for a different level of risk or responsibility. Whatever the basis of the counter-offer, it is key that the professional team understands what is different, the reasons for the differences, and the impact these may have. The process of post-tender negotiation and agreement is especially important for geotechnical works as every project will be dealing with a unique set of ground conditions. If the professional team does not include a geotechnical advisor, as defined by the ICE site investigation steering group, then one should be considered.

78.2.2 Tender documents and submissions

A more detailed list of the documents which would normally form part of a contract is given below, but it is not intended to be comprehensive. This is to give the reader an indication of what is likely to be needed for complex geotechnical works where specialist advice must be taken.

The Form of Tender (a formal bid for the work) should include the period for which the tender is valid, who is engaged to supervise the works, and the results of any relevant on-site testing.

General and project-specific specifications should be provided, including either a tender schedule or the performance criteria which the design needs to achieve. If the contractor is required to design the works, this should be expressly stated.

Drawings should include details of underground services, existing structures and other known obstructions. Site-specific information and ground investigation data should include existing ground levels, ground levels at borehole and other test positions, and a drawing showing the positions of such boreholes relative to the outline location of the works.

The conditions of contract should also include the location of the site and access to it, any limitation on working hours, any special conditions limiting noise and vibration or other environmental constraints, any phasing of the works, and the available working and storage areas.

In the case of sub-contract documents, any known special conditions or restrictions should be described, and information on such matters as insurances, retention, liquidated damages, program requirements and bonds should be provided. The bill of quantities and the form of acceptance are also required.

The tender offer should include a method statement giving a description of the proposed type of equipment to be used for the execution of the works, the method of construction and any assumptions made. Also needed are an estimate of the time required for completion of the design, the period of notice required before commencement, and the contract period for working.

Part A of the Institution of Civil Engineers' *Specification for Piling and Embedded Retaining Walls* (ICE, 2007) gives details which are relevant to deep foundation works.

78.2.3 Common forms of contract

Contractors may be invited to tender to an employer using one of the established forms of contract, or a contract document drawn up especially for management-type contracts. There are four basic types of contractual arrangement under which construction work may be undertaken:

(i) Civil engineering works with an engineer responsible to an employer for design and supervision.

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(ii) Building works with an architect responsible to an employer for design and supervision, advised by an engineer who may also be responsible to the employer for structural engineering elements of the works but who has no formal status under the conditions of contract. Under such a contract, it is desirable that the architect authorises an experienced geotechnical engineer to act as his representative in connection with the geotechnical works.

(iii) Building or civil engineering works with a contractor responsible to an employer for design and construction. The contractor may appoint an engineer to undertake the engineering duties appertaining to the geotechnical works, or may entrust those duties to a suitably qualified engineer of his own.

(iv) Building works with an architect responsible to an employer for design and supervision, but having no engineering advisor. The architect should consider the responsibilities which devolve upon him in these circumstances and should recognise that such an arrangement may not be in the best interests of either the employer or himself. This is not recommended for complex geotechnical works.

Part A of the *Specification for Piling and Embedded Retaining Walls* (ICE, 2007) gives further details relevant to deep foundation works.

78.2.4 Common forms of sub-contract

The use of specialist sub-contractors is very common for geotechnical works and such organisations often offer both a design and construction service. Sub-contract tenders for geotechnical works may be invited in the following ways:

(i) Tenders may be invited from specialist contractors for nomination as a sub-contractor; in such cases it is normal for a prime cost item to be included in the main contract bills of quantities. It is necessary to ensure that the specification and other relevant items from the sub-contract documents are included in the main contract tender documents – so that tenderers for the main contract are aware of, and can price for, their responsibilities to and their attendance on the sub-contractor. Equally, the sub-contract tender documents should include information regarding the form of the main contract, including the appendix to the contract, and sufficient detail to ensure that the sub-contractor is aware of their responsibilities and liabilities at the time of preparing the tender.

(ii) The geotechnical works may be measured and included in the bills of quantities for the main contract, and the document may stipulate that these works shall be executed by any one organisation from an approved list of contractors. The contractor may have the option of proposing further companies for approval.

(iii) The geotechnical works may be measured and included in the bills of quantities for the main contract, but without any list of approved sub-contractors. In such cases, the contractor will seek prices from their own selection of sub-contractors, for whom it is normal practice to seek approval before they are employed.

(iv) Following appointment, the contractor may be instructed to carry out additional works not included in the contract. Should this additional work include geotechnical work, tenders for such works will normally be invited from a list of companies agreed by the contractor and the engineer. It is most desirable that the tender document should be approved by the engineer before enquiries are issued.

With procedures (ii), (iii) and (iv), the selected sub-contractor is normally appointed as a domestic (direct) sub-contractor of the contractor.

Part A of the *Specification for Piling and Embedded Retaining Walls* (ICE, 2007) gives further details relevant to deep foundation works.

78.3 Specifications

78.3.1 Model specifications

There are many specifications published for use with specialist geotechnical works and most are updated on a regular basis; some examples follow.

78.3.1.1 Execution codes

BS EN 1536 *Execution of Special Geotechnical Work – Bored Piles* (BSI, 2000d)
BS EN 1537 *Execution of Special Geotechnical Work – Ground Anchors* (BSI, 2000b)
BS EN 1538 *Execution of Special Geotechnical Work – Diaphragm Walls* (BSI, 2000a)
BS EN 12063 *Execution of Special Geotechnical Work – Sheet Pile Walls* (BSI, 1999)
BS EN 12699 *Execution of Special Geotechnical Work – Displacement Piles* (BSI, 2001)
BS EN 12715 *Execution of Special Geotechnical Work – Grouting* (BSI, 2000c)
BS EN 14199 *Execution of Special Geotechnical Work – Micropiles* (BSI, 2005b)
BS EN 14731 *Execution of Special Geotechnical Work – Ground Treatment by Deep Vibration* (BSI, 2005a)

78.3.1.2 Specifications/codes of practice

American Petroleum Institute Specification for Line Pipes 5L (API, 2004)
BS EN 12794 Precast Concrete Products, Foundation Piles (BSI, 2005c)
BR 391 Specifying Vibro-Stone Columns (Building Research Establishment, 2000)
BR 458 Specifying Dynamic Compaction (Building Research Establishment, 2003a)

BR 470 Working Platforms for Tracked Plant (Building Research Establishment, 2004)
BRE 479 Timber Pile and Foundations (Building Research Establishment, 2003b)
ICE Specification for Piling and Embedded Retaining Walls ICE, 2007)
ICE Specification for Ground Treatment (ICE, 1987)

78.3.2 Design responsibility

The model specifications described above are written to be used in conjunction with all forms of contract. Hence they do not state who should have design responsibility and consequently this must be clearly stated in the contract documents.

In most cases, the engineer will design the superstructure and the part of the sub-structure which is above ground – which may include spread footings, pile caps and ground beams. A specialist sub-contractor or trade contractor will normally be asked to design the specialist geotechnical element of the works, such as the piling, ground treatment, grouting or embedded retaining wall. The engineer will nevertheless be expected to take overall responsibility for the sub-structure design and, in some cases, may be required to adopt any contractor design elements. Where the design roles are thus split, clear communication of all the design requirements is essential.

In certain circumstances the engineer may consider that it is preferable to design the complete foundation. This is usually in more complex circumstances, such as slope stabilisation, or where the geotechnical works will interact with other structures such as existing tunnels, or where it may be difficult to define the constraints adequately.

A good example is *Specification for Piling and Embedded Retaining Walls* (ICE, 2007) which requires an explicit award of responsibility for the design of the piles and foundation, and for the construction method.

78.4 Technical issues

78.4.1 Common engineer/main contractor issues

Below is a list of the issues that regularly cause dispute between the engineer and the main contractor. These are best prevented by early appointment of the contractor so that he can work with the engineer and influence the design during its development. In the absence of construction advice during the design process, the engineer may find that:

- his design is not practically buildable;
- his design is not safe to build;
- his design does not consider the latest technologies;
- instrumentation is not installed correctly or timely;
- monitoring of instrumentation is not carried out.

78.4.2 Common main contractor/sub-contractor issues

Below is a list of the issues that regularly cause dispute between main contractor and sub-contractor, and these are best prevented by early appointment of the sub-contractor and clear discussions regarding responsibilities and reliance:

- site not ready for sub-contractor upon arrival;
- working platform not adequate for the specialist plant;
- working platform not adequately maintained during the geotechnical works;
- below-ground obstructions either not removed, or removed causing disturbance to the ground or adjacent structures.

78.4.3 Common sub-contractor/sub-contractor issues

Below is a list of the issues that regularly cause dispute between the sub-contractors or trade contractors on a site. These are best prevented by getting the sub-contractors involved in the geotechnical work, or works associated with it, together soon after their appointments and discussing and agreeing interfaces and responsibilities. The contractor needs to appoint someone to manage these sub-contractors as they are likely to be relying on the services of others. Leaving sub-contractors working alone on a site is not acceptable, and disputes often arise when this happens. The list of issues is as follows:

- one sub-contractor is late, thereby delaying another sub-contractor;
- several sub-contractors need to use the same part of the site at the same time (including deliveries, use of tower cranes);
- one sub-contractor's design is not compatible with another's design;
- one sub-contractor's product is not compatible with another's product;
- when something goes wrong, all the sub-contractors blame each other and none can be proven to be responsible.

78.4.4 Preparation for construction

Specialist geotechnical works often require the use of specialist plant and the loadings these may apply to the ground may be considerable. The working platform is a key item of temporary works, and responsibility for its design, specification, construction, maintenance and repair must be clearly allocated. There have been fatal consequences from an inadequate working platform in the past. Guidance for piling platforms is provided in BRE report BR470 (Building Research Establishment, 2004).

Responsibility for identifying and removing obstructions in the ground needs careful consideration. Normally, shallow obstructions can be safely removed unless there are sensitive structures nearby. It may not be possible to remove deeper obstructions practically or safely, and hence these will have an impact on the design and construction of the geotechnical works. It may also not be possible to identify all obstructions in the ground, and the risks associated with these should be considered carefully.

Where obstructions have been removed from the ground, the geotechnical designer needs to take account of the methods used and any impact on their design or on adjacent structures both above and below ground. Backfilling of removed obstructions

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needs to be a controlled procedure using materials and methods that do not unduly affect other works.

Unexploded ordnance in the ground is often a real hazard and should be investigated on most sites. Expert advice is provided in CIRIA report C681 (Construction Industry Research and Information Association, 2009).

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It is recommended this chapter is read in conjunction with

- Chapter 93 *Quality assurance*
- Chapter 96 *Technical supervision of site works*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 79

Sequencing of geotechnical works

Mark Pennington Balfour Beatty Ground Engineering, Basingstoke, UK
Tony P. Suckling Balfour Beatty Ground Engineering, Basingstoke, UK

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To achieve the safety and technical requirements of geotechnical works, it is imperative that a sequence of working is agreed prior to commencement and is subsequently adhered to by all the organisations working on a project at the same time. When agreeing the sequencing, the role of the main/management contractor is crucial – they are likely to be the only party with an understanding of the overall construction sequence and the roles and responsibilities of each organisation on site. The main/management contractor needs to understand each organisation's ideal method of working to achieve their own particular safety and technical requirements. If, as is common, many organisations on a site interact with each other or are reliant on one another, the main/management contractor must help these organisations to modify their ideal method of working without compromising safety or technical requirements. Geotechnical design often requires a particular construction sequence to be followed or else the design is invalidated and the geotechnical works may not progress to plan; this would introduce another complication which would need to be managed by the main/management contractor.

79.1 Introduction

Every construction project will have a unique combination of particular contractors, sub-contractors, processes, products and materials on site. Geotechnical engineering is a specialist aspect of civil engineering and so is normally undertaken by specialist organisations. The geotechnical works need to be undertaken safely and achieve all the technical requirements. The safety of on-site personnel and the public is paramount, and the integrity of works by others (existing, new or proposed) must not be compromised.

It is common for the geotechnical works to be undertaken by one or more specialist organisations under the control of a main contractor or management contractor. This leads to two separate construction sequencing issues: the overall project construction sequence and the specific task construction sequence. To achieve the safety and technical requirements it is imperative that a sequence of working is agreed prior to commencement and then adhered to by all the organisations working on a project at the same time. When agreeing the sequencing, the role of the main contractor or management contractor is crucial – they will have responsibility for the overall project construction sequence. They are likely to be the only party on site with an understanding of the overall construction sequence and the roles and responsibilities of each organisation on the site. The main/management contractor needs to understand the ideal method of working for each of these organisations to achieve their own particular safety and technical requirements, and to investigate if any of the organisation's methods of working will impact on one another. If, as is common, many organisations on a site interact with each other or are reliant on one another, the main/management contractor must help these organisations to modify their ideal method of working without compromising safety or technical requirements.

Specific task construction sequences are generally better defined and controlled. In geotechnical works, each specific task sequence is generally the responsibility of one contractor who is experienced in the activity. Detailed method statements and risk assessments are carried out. It is hard to control the interaction and impact of specific task construction sequences on each other. A relatively minor change in the sequence of one task may have a significant impact on the overall process.

Geotechnical design often requires a particular construction sequence to be followed or else the design is invalidated and the geotechnical works may not progress to plan; this would introduce another complication which would need to be managed by the main/management contractor. The geotechnical designer needs to appreciate the constructability constraints in the design. If this is not the case and the contractor finds that the particular construction sequence cannot be followed due to practical or technical reasons, or due to safety concerns, then the work must cease and the sequence and design must be suitably modified before the works recommence.

79.2 Design construction sequence

Geotechnical design is dependent on a number of different factors; many of these are not exact and are based on the best available knowledge. Examples include assumptions about the ground conditions, the previous usage of the site, the groundwater regime and the type of construction methodology. It is important that all concerned understand any assumptions made in the design construction sequence, so that they are in a position to assess the impact of any changes during construction.

In order to accommodate unknowns or variables (due to the very nature of working with the ground), the geotechnical design would normally require one or more of the following to

be carried out on site: observations, monitoring, investigation, testing, and reported back to the designer to check the validity of the design. If the contractor chooses to deviate from the design construction sequence, either by choice or due to a lack of site control, it is likely that the design will be invalidated and the works will not progress to plan.

The geotechnical designer should have sufficient appreciation of the capabilities and limitations of the construction process, products and materials so that it is not unreasonable to expect the particular construction sequence assumed in the geotechnical design to be closely followed by the contractor. In reality, this can actually be difficult to achieve as many specialist geotechnical contractors use proprietary systems, and if the procurement process and contractual arrangements do not appoint the contractor before the design is finalised, then the design requirements may not be compatible with the contractor's capabilities or requirements. If the contractor finds that the particular construction sequence used in the design cannot be followed due to practical or technical reasons, or due to safety concerns, then the sequence and the design must be suitably modified before the works commence.

To best communicate the construction sequence between designer and contractor, a detailed drawing is recommended (see **Figure 79.1**). Geotechnical design is generally concerned with excavation into the ground or building on top of the ground. To this end, design construction sequences are generally two-dimensional drawings showing the maximum excavation or construction level for each phase. It is important not to overlook the plan constraints of the site works.

79.3 Site logistics

Site logistics are crucial in determining a successful construction project. There would be constraints on a very small site where plant movement is very limited, and on a large site where plant movement is a significant element in the programming of the project. The type of specialist plant available and how the plant can be moved may be an integral part of the overall solution. Contractors may organise work around available plant, which is difficult to plan during the design phase of contracts. However, it is important that allowance is made during design to cater for the most likely option. On particularly complex projects, advice should be taken from various specialist contractors to establish likely construction methodology.

Site logistics need to be planned to follow the specific design requirements of each element. For example, it is no use designing a complicated conveyor system to move spoil from a tunnel excavation if this does not fit around the propping system in the excavation at the tunnel portal.

Specific task sequences have to fit within the overall project sequence. The site logistics for specific activities are easier to plan than those for the overall project. Specific activities generally lie with one contractor, who is fully aware of their own plant requirements and limitations. For example, following the construction of a section of retaining wall, excavation may be required to proceed as quickly as possible in front of the wall. One aspect to consider in how quickly this can happen is the movement of plant around the site. One contractor may be responsible for the wall construction and another for the excavation. The full sequence of working can only be developed when the plant and site logistics of each element are understood. If these logistics require a change to the designed sequence, it would necessitate a redesign and may impact many activities.

Minimum working areas must be defined for each contractor working on the site. In addition, access to each work site area has to be clearly agreed between all parties. Construction sites are, by definition, constantly changing and developing. It is important that drawings showing the working space requirements for each contractor are prepared and agreed by *all* parties involved. These will have to be prepared on a regular basis to reflect the changing nature of the site (see **Figure 79.2**). It is common for site logistics to be drawn in plan but the vertical excavation or construction should not be ignored in these drawings.

79.4 Safe construction

Health and safety in geotechnical engineering is described in Chapter 8 *Health and safety in geotechnical engineering*.

Generic and specific risks and hazards need to be identified for both the specific and project construction sequences. Each contractor is responsible for establishing the hazards and risks for the activities they are undertaking, and also for the residual risks they find on arrival, and those they hand on to any follow-on contractor.

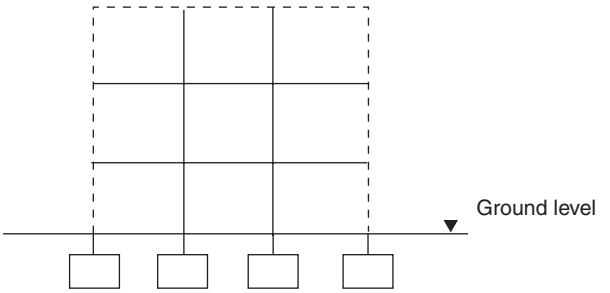
Geotechnical works are increasingly being carried out on brownfield sites which have had one or more previous uses. Any construction sequence undertaken needs to take into account this previous usage.

Every contractor needs to assess the generic hazards and their control or mitigation measures for the specific characteristics of the site, and also to identify site-specific hazards and carry out a risk assessment for these, see **Table 79.1**.

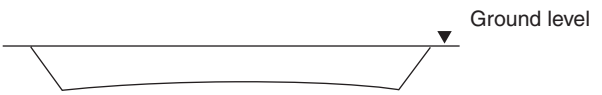
Part of the site-specific risk assessment process is to identify if the proposed construction works or their sequence causes a hazard to others. It is common for the geotechnical works to be undertaken by one or more specialist organisations under the control of a main contractor or management contractor. To achieve the safety requirements it is imperative that a sequence of working is agreed to by all prior to commencement on site, and then adhered to by everyone working on a project at any one time. When agreeing this sequencing, the role of the main contractor or management contractor is crucial; they are likely to be the only party on site with an understanding of the overall construction sequence and the roles and responsibilities of each organisation on the site. The main/management contractor needs to understand the ideal method of working for each of these organisations and then to investigate if any of the organisation's methods of working will impact on one another. If, as is common, many organisations on a site interact with each other or are reliant on one another, the main/management

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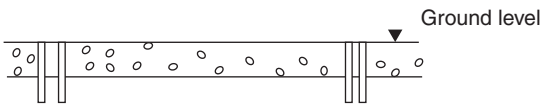
1. Demolish exisitng buildings



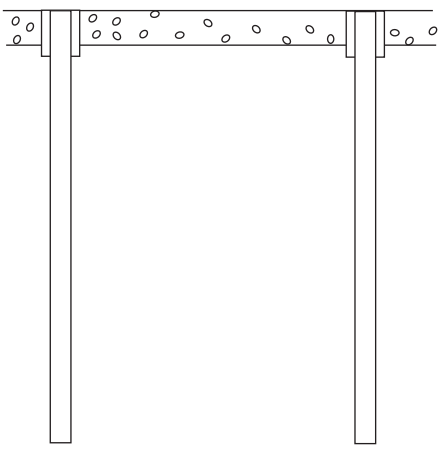
2. Remove existing foundations



3. Prepare site, i.e. probing, guide walls, working platform



4. Construct diaphragm walls



5. Excavate in stages, installing temporary propping and base slab
Maximum excavation levels prior to temporary or permanent propping is installed needs to be clearly specified

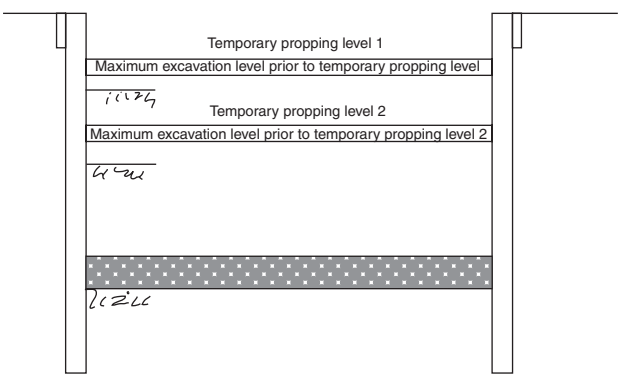


Figure 79.1 Example of a construction sequence for a Tunnel Boring Machine Launch Chamber

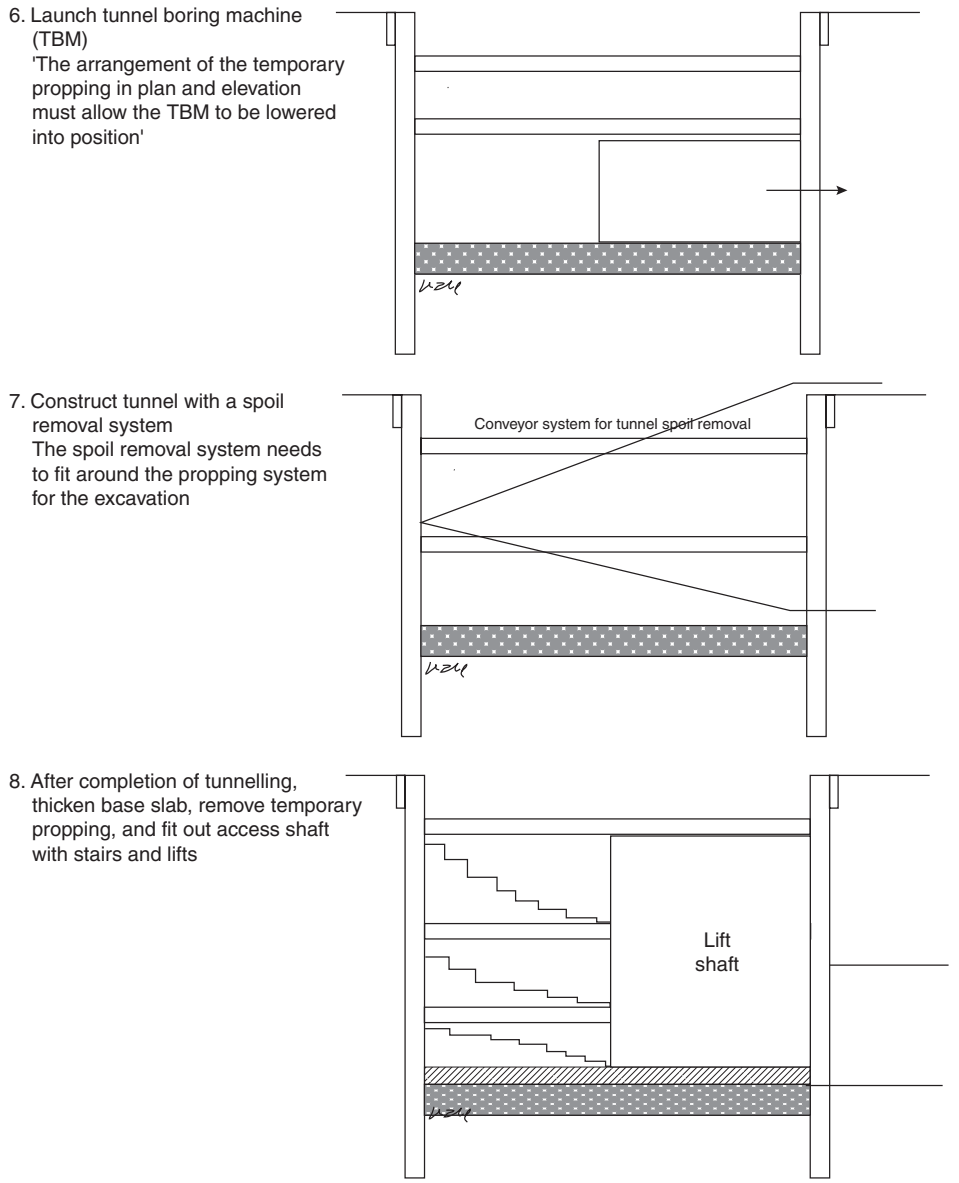


Figure 79.1 (Continued)

contractor must help these organisations to modify their ideal method of working to achieve a maximum level of safety.

79.5 Achieving the technical requirements

Geotechnical engineering is complex. The following technical requirements are often key components of the design and construction process:

- Installation method must not cause significant ground disturbance or movement and so invalidate the design (see **Figure 79.3**).
- Construction method must not be a nuisance, e.g. too noisy.
- Construction method must not cause damage to existing buildings or structures, either above or below ground, e.g. by causing too much vibration.
- Plant must not be too heavy or work in a way that could cause damage to underground structures or services, or to newly-constructed works.
- Sequencing of plant within temporary works needs to be carefully planned so that plant has sufficient and safe working areas, and temporary works continuously fulfill their function.
- Stockpiling of materials is done in a way that ground movements resulting from the applied surcharge to the ground do not damage existing buildings/structures or newly-constructed works.
- Construction method may cause softening or instability of the ground if work progresses too slowly.
- Construction method may cause premature failure of the ground if work progresses too fast.

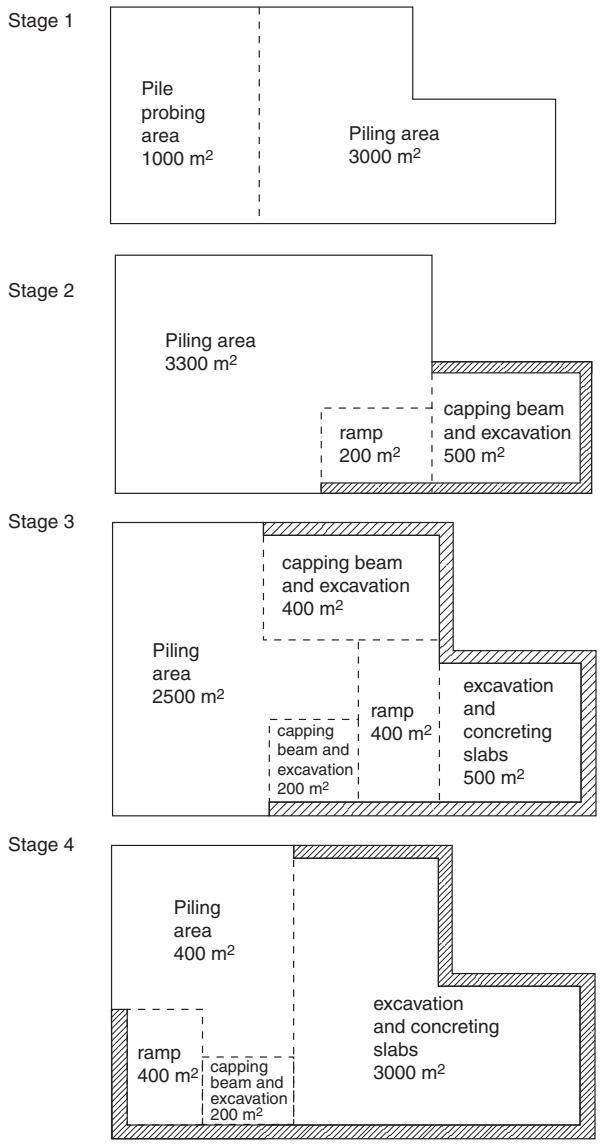


Figure 79.2 Example of typical working areas for a basement construction

- Follow-on works are planned in a way not to cause damage to the geotechnical works.

The relevance of each of these depends on the actual ground conditions, the geotechnical solution being adopted, the construction method being used and the design philosophy. Hence, geotechnical specialists are needed to assess each of these and communicate their findings to the professional team as an integral part of the design and construction process. The impact of one element of works on another, either generally or at interfaces, needs to be carefully considered.

79.5.1 Piling works

The following list is not exhaustive and there are many more examples of achieving the technical requirements for piles. Similar lists could be produced for all geotechnical processes.

- In ground conditions containing hard layers, the rotation of continuous flight augers (CFAs) can cause significant smearing of the bore, or induce ground movements invalidating the pile design.



Figure 79.3 Piles which have been affected by moving ground

Hazard	Likely consequences if hazard arises	Impact rating	Likelihood	Severity rating before mitigation (impact × likelihood)	Mitigation measures	Impact rating	Likelihood	Severity rating after mitigation (impact × likelihood)
	Risk of injury to either contractors' personnel	5	3	15		5	1	5
One contractor working in close proximity to another contractor	Risk of damage to plant	3	3	9	Provide barriers between work areas, exclusion zones within work areas and pedestrian access around the work areas	3	1	3
	Risk of damage to work done	3	3	9		3	1	3

Table 79.1 Example of how a generic risk assessment has been modified to take account of others on site concurrently. Ratings and likelihoods vary from 1 (low) to 5 (high)

- The driving of piles may be too noisy at the site boundary.
- Piles may be too close together: during construction of a new pile, a recently constructed one is damaged.
- Bearing pressures from piling plant on a working platform overlying a soft sub-grade may damage recently concreted piles.
- Piling plant working within temporary propping to basement walls has to be carefully designed and planned, with the props being removed and replaced in a controlled sequence.
- Stockpiling of fill material to form a piled embankment over soft clay is done in a limited way, so that ground movements resulting from the applied surcharge to the ground do not damage the piles.
- The time between boring and concreting a pile is to be minimised.
- Loading of a test pile in chalk should be undertaken in small increments.
- Trimming piles is to be carried out in a way so as not to cause damage to the piles.

79.6 Monitoring

It is often necessary for a geotechnical solution to be monitored during construction and maybe in the long term as well. The monitoring can be to confirm design assumptions or can be an integral part of the design, for example when filling for an embankment or when the observational method (see Chapter 100 *Observational method*) is being used.

The instrumentation should be installed prior to construction so that baseline readings can be established. The instruments and their cabling must be protected during construction, and the construction methods and sequencing should take this into consideration. Unfortunately this is often not the case.

79.7 Managing changes

Ground conditions are rarely homogeneous and are seldom investigated in enough detail prior to construction. Consequently it is common for the ground conditions to be slightly (or sometimes significantly) different to that expected. When this occurs, everyone involved comes under commercial pressure to provide an appropriate solution. In these circumstances, it is important to ensure that the changed sequencing of the geotechnical works still achieves the aforementioned safety and technical requirements.

It is common for an accident or technical problem to be related back to a change in circumstances on the site, which may be a change in ground conditions or in something else.

When on site, one should be aware that not everyone may actually follow the previously agreed method of working, either for good reason or for no reason at all.

There are always changes on site and it is difficult to ensure that all involved have reassessed their works and their impact on others. However, it is imperative to do so. A common problem occurs when a change is made to a specific task sequence, making it more efficient and cost effective. Checks may be made to show that the change in sequence has no impact on the design of that specific task. However, if this necessitates significant changes to the follow-on construction sequence (to ensure the design is valid) then the sequence should not be changed without the agreement of all the parties involved.

79.8 Common problems

Below are some specific problems that commonly occur on complex urban geotechnical projects.

(i) Poor quality material is used to backfill old foundations that have been removed, leading to excessive concrete overbreak on a new embedded wall installed along the line of the old foundations, which then has to be laboriously chiselled back to meet the requirements of the specification.

(ii) A ramp into an existing basement or excavation to allow a piling rig to access and egress to undertake piling from the base of the excavation normally needs to be at a slope of less than 1 in 10. Hence such a ramp needs to be over 100 m long for a 10 m deep excavation. It is often more practical to temporarily backfill an existing basement to allow new piling, but this may then require coring through any basement slab, or the breaking out of holes in the slab prior to backfilling.

(iii) Top-down basement construction can often be beneficial on congested sites, as one can then win back new laydown areas for materials and space for plant. For tall structures, top-down construction provides a shorter program; for smaller structures it may provide the only practical and safe method of construction.

It is recommended this chapter is read in conjunction with

- Chapter 8 *Health and safety in geotechnical engineering*
- Chapter 67 *Retaining walls as part of complete underground structure*
- Chapter 94 *Principles of geotechnical monitoring*
- Chapter 96 *Technical supervision of site works*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 80
Groundwater control

Martin Preene Golder Associates (UK) Ltd, Tadcaster, UK

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Groundwater control encompasses the range of temporary works techniques used to allow below-ground construction projects to be carried out in dry and stable conditions. Two principal approaches can be used: groundwater control by pumping (also known as construction dewatering), which involves pumping from an array of wells or sumps to lower groundwater levels in the vicinity of an excavation; or groundwater control by exclusion, which relies on low permeability cut-off walls around the excavation to prevent or reduce groundwater inflows.

80.1 Introduction

Groundwater control may be required as part of the temporary works for below-ground construction projects, in order to provide dry and stable working conditions. Groundwater control can be achieved by pumping (using a group of techniques known as construction dewatering). In this approach water is pumped from an array of wells or sumps, to lower groundwater levels in and around an excavation. Alternatively groundwater can be controlled by exclusion, where low permeability cut-off walls are installed around an excavation to prevent or reduce groundwater inflows.

80.2 Objectives of groundwater control

The occurrence and control of groundwater is fundamental to geotechnical engineering (see Chapter 16 *Groundwater flow*). Where groundwater is encountered during excavation, problems can occur either in terms of flooding of the excavation, or in the form of instability induced by its presence. Groundwater control can be defined as the range of activities used to allow construction work to proceed at depths below groundwater level.

The most obvious (but not necessarily the most important) objective of groundwater control is to prevent an excavation below the natural groundwater level from flooding. However, especially in fine-grained soils of low to moderate permeability (such as silts or silty sands) another objective is to avoid groundwater-induced instability of the excavation by controlling pore water pressures and hence effective stresses around the excavation. In these soils, the pore water pressures associated with even very small seepages into the excavation can cause serious instability. It is not always understood that in fine soils the objective of groundwater control is to control pore water pressures to ensure stability, and not to literally 'dewater' the soil.

Basic soil mechanics theory (see Chapter 15 *Groundwater profiles and effective stresses*) states that soil behaviour is controlled by the effective stress σ', which is related to total stress σ (due to external loads) and the pore water pressure u by Terzaghi's equation:

$$\sigma' = \sigma - u \tag{80.1}$$

The shear strength τ_f of a soil depends on the normal effective stress, according to the Mohr–Coulomb failure criterion:

$$\tau_f = \sigma' \tan \phi' \tag{80.2}$$

where τ' is the effective angle of soil friction. Equations (80.1) and (80.2) illustrate that a reduction in pore water pressure at constant effective stress (as may result from a drawdown in groundwater level) increases the normal effective stress and enhances the ability of the soil to resist shear, thus improving stability. Conversely, the positive pore water pressures associated with seepage into the excavation have a destabilising effect and result in slumping of side slopes and softening of the base as shown in **Figure 80.1(a)**. Base heave in an excavation can also occur due to unrelieved pore water pressures beneath very low-permeability layers below excavation formation level (**Figure 80.1(b)**). Both forms of instability can be avoided by the use of a suitable groundwater control system.

Groundwater control alone should not be expected to achieve a totally dry excavation. It can only do this in favourable ground conditions when used with an effective surface water control system. In other circumstances, groundwater control can give a stable and workably dry excavation, but some residual seepages may remain.

80.2.1 Permanent groundwater control

Although the predominant application of pumped groundwater control systems is as part of temporary works during construction, occasionally such systems are used on a 'permanent' basis for the entire design life of a structure. The most common application of permanent groundwater control is to reduce piezometric pressures below large underground structures. Their reduction will lessen the uplift buoyancy forces

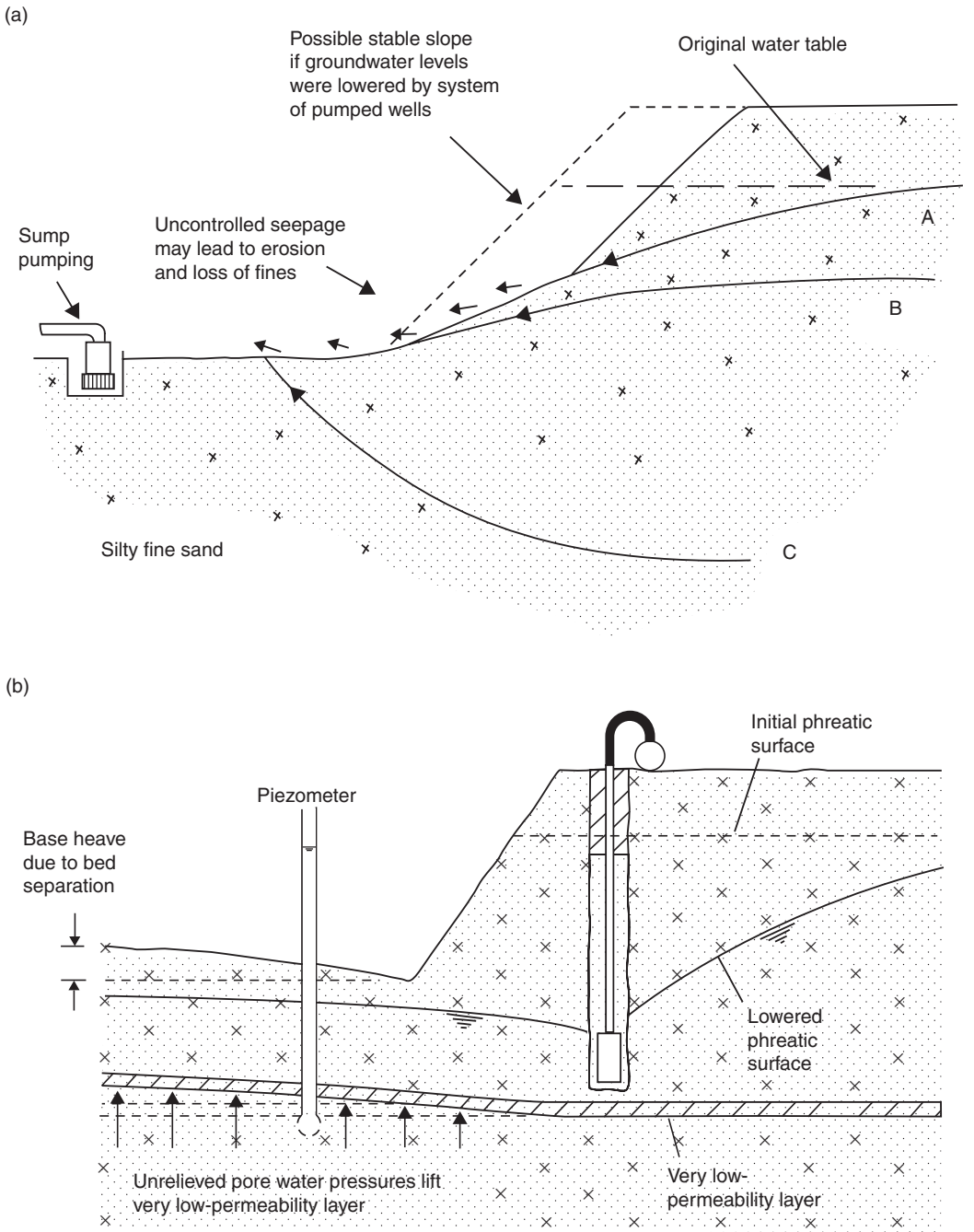


Figure 80.1 (a) Groundwater-induced instability of excavations; (b) Instability of base due to unrelieved pore water pressure
(a) Reproduced with permission from Cashman and Preene (2001) © Taylor & Francis Group; (b) Reproduced with permission from CIRIA C515, Preene *et al.* (2000), www.ciria.org

on the structure and may allow reduction in deadweight of the permanent structure. In such cases, the savings in capital cost of the structure may outweigh the costs of long-term operation of the pumping system (Whitaker, 2004).

80.2.2 Control of surface water

Groundwater control is separate from the control of surface water such as rainfall run-off. Many contractual disputes have arisen when a groundwater control system has been operating but puddles of water remain in the excavation. If layers of clay exist in the soil then these puddles may simply be ponded rainwater. Any excavation, including those above the water table, should have a system for surface water control, typically consisting of sump pumps and French drains. Surface water can come from a variety of sources, including rainfall, direct seepage from nearby rivers or lakes, leaking sewers and water mains

or the construction operations themselves. Whatever its source, surface water will need to be controlled to allow efficient construction operations. Guidance on surface water control can be found in Cashman and Preene (2001) and Powers *et al.* (2007).

80.3 Methods of groundwater control

The requirement to pump groundwater can be reduced or even avoided by installing a notionally impermeable physical cut-off wall or cofferdam around the excavation to exclude groundwater from it.

If an impermeable stratum exists at shallow depth beneath the excavation, then the wall may be able to penetrate down and into that stratum to create a full cut-off (**Figure 80.2(a)**). The only pumping requirement will be to drain the water trapped within the cut-off and to deal with leakages through the wall and through the impermeable stratum.

On the other hand, if an impermeable stratum does not exist at a convenient depth, only a partial cut-off (**Figure 80.2(b)**) can be formed. Groundwater will still enter the excavation, but the cut-off increases the seepage path length and reduces the flow rate compared to the case when there is no cut-off at all. The cut-off should be designed to be of adequate penetration to prevent piping failure of granular soils (see Chapter 16 *Groundwater flow*).

Alternatively, it may be possible to form a horizontal barrier or 'floor' to the cut-off structure to prevent vertical seepage (**Figure 80.2(c)**). The construction of horizontal barriers is relatively rare, but has been carried out using methods such as jet grouting (see Chapter 90 *Geotechnical grouting and soil mixing*) (Newman *et al*., 1994), and artificial ground-freezing techniques (Harris, 1995).

80.4 Groundwater control by exclusion

A wide range of methods can be used to exclude groundwater from excavations (Bell and Mitchell, 1986). Key attributes of more commonly used cut-off methods are described in **Table 80.1**. Some methods used to form groundwater cut-off barriers are described elsewhere in this book, including diaphragm walls, secant pile walls, sheet pile walls (Chapter 85 *Embedded walls*), permeation grouting, jet grouting (Chapter 90 *Geotechnical grouting and soil mixing*) and mix-in-place walls (Chapter 85 *Embedded walls* and Chapter 90 *Geotechnical grouting and soil mixing*).

Some of the methods are temporary. For example, the groundwater will thaw when artificial ground freezing is discontinued, or steel sheet piles can be extracted at the end of the job. These temporary methods should not have a significant effect on groundwater conditions at the site once the project is completed. However, methods which permanently affect soil permeability (e.g. grouting) can permanently alter groundwater flow regimes at the site – it is essential that the potential impact of this is assessed at design stage (see section 80.6.5).

Even when a cut-off is used, some pumping will be required to cope with:

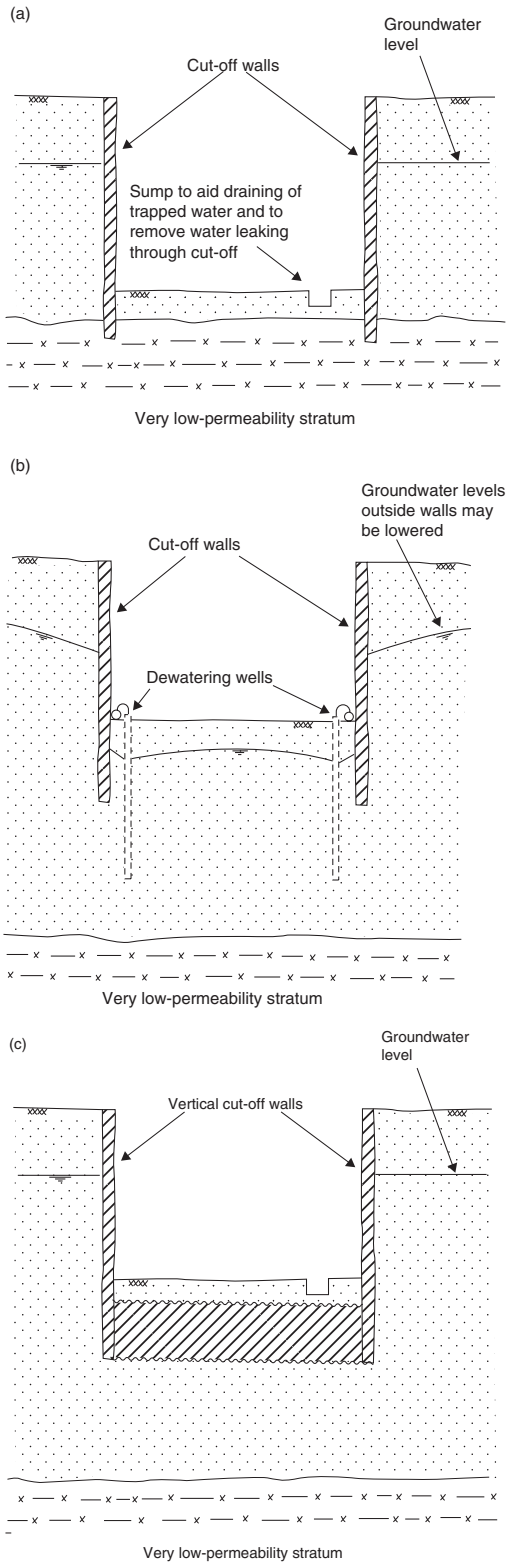


Figure 80.2 Physical cut-off walls (a) penetrating into very low permeability stratum; (b) used in combination with dewatering methods; (c) used with horizontal barrier to seal base

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Method	Typical applications	Comments
Displacement barriers		
(1) Steel sheet-piling	Open excavations in most soils, but obstructions such as boulders may impede installation	Temporary or long-term. Rapid installation. Can support the sides of the excavation with suitable propping. Vibration and noise of driving may be unacceptable on some sites, but 'silent' methods are available
(2) Vibrated beam wall	Open excavations in silts and sands. Will not support the soil	A vibrating H-pile is driven into the ground and then removed. As it is removed, grout is injected through nozzles at the toe of the pile to form a thin, low-permeability membrane. Relatively cheap
Excavated barriers		
(3) Slurry trench cut-off wall using bentonite or native clay	Open excavations in silts, sands and gravels up to a permeability of about 5×10^{-3} m/s	The slurry trench forms a low-permeability curtain wall around the excavation. Quickly installed and relatively cheap, but cost increases rapidly with depth
(4) Structural concrete diaphragm walls	Side walls of excavations and shafts in most soils and weak rocks	Support the sides of the excavation and often form the sidewalls of the finished construction. Minimum noise and vibration
(5) Bored pile walls (secant and contiguous bored piles)	As (4)	As (4), but more likely to be economical for temporary works use. Sealing between contiguous piles can be difficult
Injection barriers		
(6) Jet grouting	Open excavations in most soils and very weak rocks	Typically forms a series of overlapping columns of soil/grout mixture
(7) Permeation grouting using cementitious grouts	Tunnels and shafts in gravels and coarse sands, and fissured rocks	The grout fills the pore spaces, preventing the flow of water through the soil. Equipment is simple and can be used in confined spaces
(8) Permeation grouting using chemical and solution (acrylic) grouts	Tunnels and shafts in medium sands (chemical grouts), fine sands and silts (resin grouts)	Materials (chemicals and resin) can be expensive. Silty soils are difficult and treatment may be incomplete, particularly if more permeable laminations or lenses are present
Other types		
(9) Artificial ground freezing using brine or liquid nitrogen	Tunnels and shafts. Will not work if groundwater flow velocities are excessive (>1 m/day or 10^{-5} m/s)	Temporary. A 'wall' of frozen ground (a freezewall) is formed, which can support the side of the shaft as well as excluding groundwater. Plant costs are relatively high. Liquid nitrogen is expensive but quick; brine is cheaper but slower
(10) Compressed air	Confined chambers such as tunnels, sealed shafts and caissons	Temporary. Increased air pressure (up to 3.5 bar) raises pore water pressure in the soil around the chamber, reducing the hydraulic gradient and limiting groundwater inflow. High running and set-up costs; potential health hazards to workers

Table 80.1 Physical cut-off techniques for exclusion of groundwater
Data taken from Preene *et al.* (2000)

- groundwater trapped within the cut-off area;
- rainfall and precipitation;
- seepage through the wall and through the ground.

It is important to recognise that most cut-off walls will leak. For the common example of steel sheet piles, clutches may leak and declutching of piles may occur in difficult driving conditions. The presence of cobbles and boulders will have a detrimental effect on the integrity of many cut-off methods.

Despite the potential problems, groundwater control by exclusion is widely used both in the UK and overseas. Perhaps one of the reasons is that, if carried out effectively and if ground conditions are favourable, an exclusion system can minimise or even eliminate any groundwater lowering outside the site area. This can be a very attractive outcome on projects where settlement of nearby structures or detrimental effects on other water users are of concern (see section 80.6.5).

80.5 Groundwater control by pumping

Groundwater control by pumping involves pumping groundwater from an array of wells or sumps with the aim of temporarily lowering groundwater levels to allow civil engineering projects to be constructed in stable conditions. Groundwater control by pumping is also known as *groundwater lowering*, *construction dewatering* or simply *dewatering*. The amount of lowering of the groundwater level is known as drawdown.

Table 80.2 lists the various techniques of pumped well groundwater control methods available. However, in the UK

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Method	Typical applications	Comments
Drainage pipes or ditches (e.g. French drains)	Control of surface water and shallow groundwater (including overbleed and perched water)	May obstruct construction traffic, and will not control groundwater at depth. Unlikely to be effective in reducing pore water pressures in fine-grained soils
Sump pumping	Shallow excavations in clean coarse soils, for control of groundwater and surface water	Cheap and simple. May not give sufficient drawdown to prevent seepage from emerging on the cut face of a slope, possibly leading to instability
Wellpoints	Generally shallow, open excavations in sandy gravels down to fine sands and possibly silty sands. Deeper excavations (requiring >5–6 m drawdown) will require multiple stages of wellpoints to be installed	Relatively cheap and flexible. Quick and easy to install in sands. Difficult to install in ground containing cobbles or boulders. Maximum drawdown is ~ 6 m for a single stage in sandy gravels and fine sands, but may only be ~ 4 m in silty sands
Horizontal wellpoints (machine laid)	Generally shallow trench or pipeline excavations or large open excavations in sands and possibly silty sands	Horizontal wellpointing is suitable for trench excavations outside urban areas, where very rapid installation is possible
Deep wells with electric submersible pumps	Deep excavations in sandy gravels to fine sands and water-bearing fissured rocks	No limit on drawdown. Expensive to install, but fewer wells may be required compared with most other methods. Close control can be exercised over well screen and filter
Shallow bored wells with suction pumps	Shallow excavations in sandy gravels to silty fine sands and water-bearing fissured rocks	Particularly suitable for coarse, high-permeability materials where flowrates are likely to be high. Closer control can be exercised over the well filter than with wellpoints
Passive relief wells and sand drains	Relief of pore water pressure in confined aquifers or sand lenses below the floor of the excavation	Cheap and simple. Create a vertical flowpath for water into the excavation; water must then be directed to a sump and pumped away
Ejector system	Excavations in silty fine sands, silts or laminated clays in which pore water pressure control is required	In practice, drawdowns generally limited to 30–50 m. Low energy efficiency, but this is not a problem if flowrates are low. In sealed wells a vacuum is applied to the soil, promoting drainage
Deep wells with electric submersible pumps and vacuum	Deep excavations in silty fine sands, where drainage from the soil into the well may be slow	No limit on drawdown. More expensive than ordinary deep wells because of the separate vacuum system. Number of wells may be dictated by the requirement to achieve an adequate drawdown between wells, rather than the flowrate, and an ejector system may be more economical
Collector wells	High-permeability sands and gravels	Each collector well is expensive to install, but relatively few wells may produce large flow rates and be able to dewater large areas
Artificial recharge	High-permeability soils and highly fissured rocks	Recharge systems are complex to operate and maintain. Recharge wells often suffer from clogging and may require periodic backflushing and cleaning
Electro-osmosis	Very low-permeability soils, e.g. clays	Only generally used for pore water pressure control when considered as an alternative to ground freezing. Installation and running costs are comparatively high

Table 80.2 Summary of principal pumped well groundwater control methods
Data taken from Preene *et al.* (2000)

the vast majority of projects are carried out using just four main conventional dewatering techniques:

- sump pumping;
- wellpoints;
- deep wells;
- ejector wells.

Each of the main pumping techniques will be described briefly in the following sections. **Figure 80.3** is a diagram which shows the range of application of each method relative to the two key parameters: drawdown required and soil permeability (the shaded areas show zones where more than one technique may be suitable).

A variation on groundwater control by pumping is 'passive drainage' where groundwater flow from an excavation or slope is facilitated by gravity flow, i.e. without the requirement for direct pumping. Indirect pumping of the water, after it has emerged from the ground, may be necessary to prevent the water causing problems elsewhere on site. Relief wells (see section 80.5.5.1) are a commonly used method of passive drainage.

An important distinction within the pumping methods is between open pumping and pre-drainage methods. Open pumping, most commonly carried out by sump pumping,

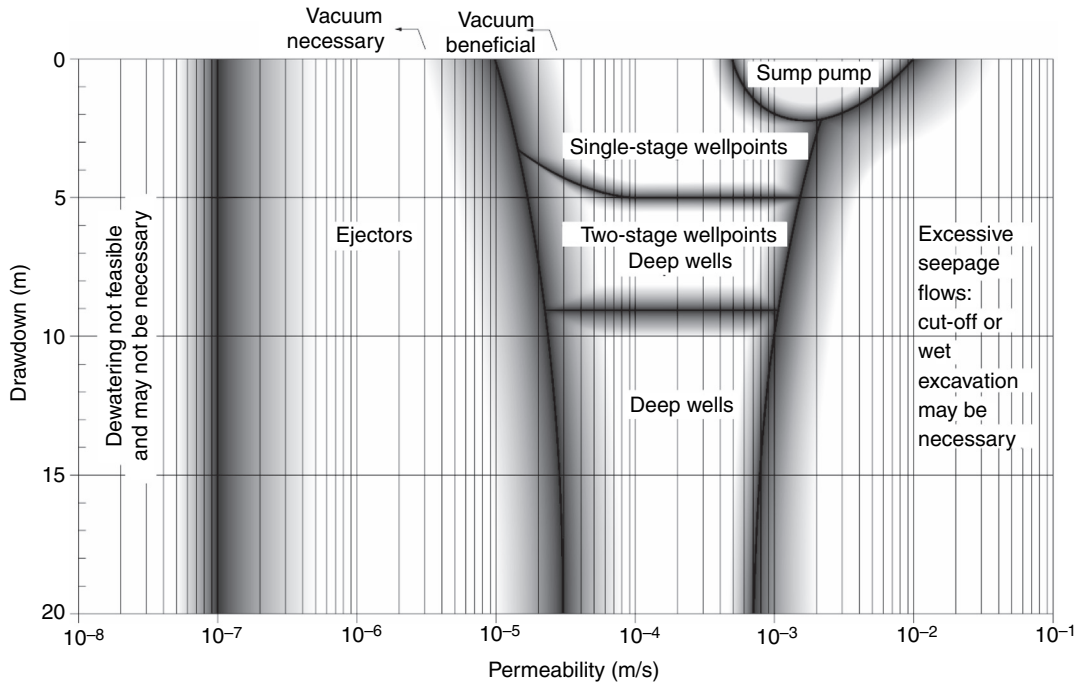


Figure 80.3 Range of application of pumped well groundwater control techniques
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involves removing water that has already entered the excavation by pumping from within it. While simple in practice, open pumping has the disadvantage that groundwater levels cannot be lowered in advance of excavation. Open pumping typically requires water to enter the excavation before it can be pumped away, and localised instability of the excavation may result.

In contrast, pre-drainage methods (which include wellpoints, deep wells and ejector wells) work on the principle of lowering groundwater levels in advance of excavation works. This group of methods has the advantage that groundwater can be managed so that water does not enter the excavation, reducing the risk of groundwater-induced instability.

80.5.1 Sump pumping

When sump pumping is implemented, water is allowed to enter the excavation where it is collected in drains and fed to special pits called sumps from where it is pumped away (**Figure 80.4**). The major limitations of sump pumping are that the sumps and collector drains take up space within the excavation, and it is not possible to lower groundwater levels much below formation level; the sump must be continually deepened as the excavation proceeds.

Under favourable conditions, sump-pumping systems can be a simple and cost-effective means of controlling groundwater inflows to an excavation. Under unfavourable conditions, a sump-pumping approach can result in delays, cost overruns and, occasionally, catastrophic failure. The primary limitation on sump pumping is the instability of the soil under the action of the seepage forces generated by the groundwater entering the excavation (**Figure 80.1**). This is commonly referred to as 'running sand conditions' or 'boiling' and can cause rapid loss of base and side slope stability, leading to a risk of undermining and settlement to adjacent structures.

The following attributes need to be taken into consideration when designing a sump:

- Depth: the sump should be deep enough to drain the excavation and drainage network, allowing for the pump intake level and some accumulation of sediment.
- Size: the sump should be substantially larger than the size of the pump to allow space for sediment and cleaning.
- Filter: the sump lining should be perforated or slotted, typically with a hole size or slot width of 10 to 15 mm. The sump should be surrounded with coarse gravel (20 to 40 mm).
- Access: good access is required to allow removal of the pumps for maintenance and cleaning of the sumps to remove any accumulation of sediment.

When carrying out sump-pumping operations, some of the sand and fines fraction in the soil will initially be removed in the immediate vicinity of the sump and drainage network. It is good practice to pass the discharge water through a settlement tank to allow the situation to be monitored and to remove the solids that settle readily prior to discharge. Settlement ponds or lagoons may be needed to remove any silt or clay fraction present to meet discharge consent requirements. If persistent movement of fines occurs, leading to ground loss and

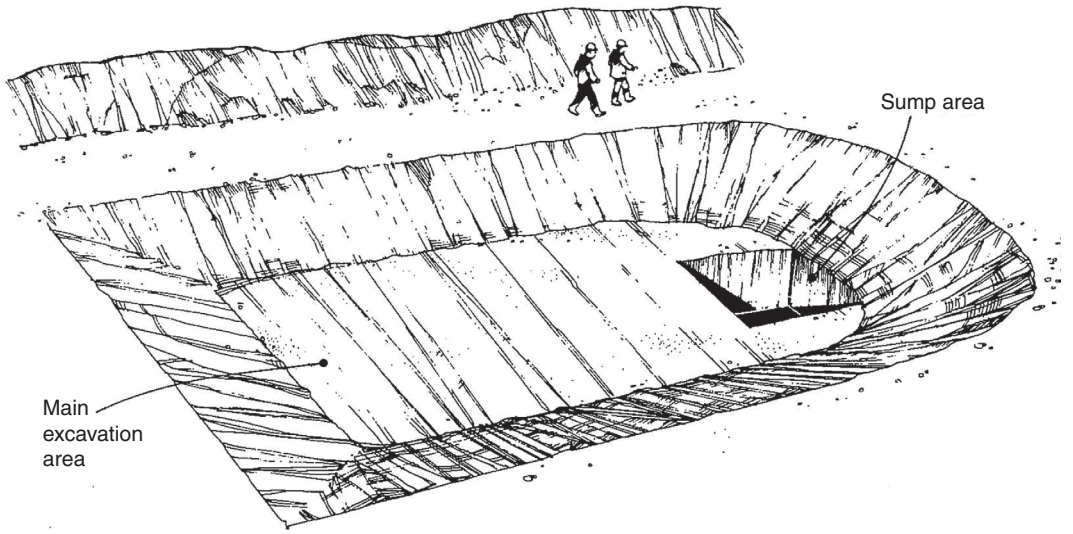


Figure 80.4 Sump within an excavation
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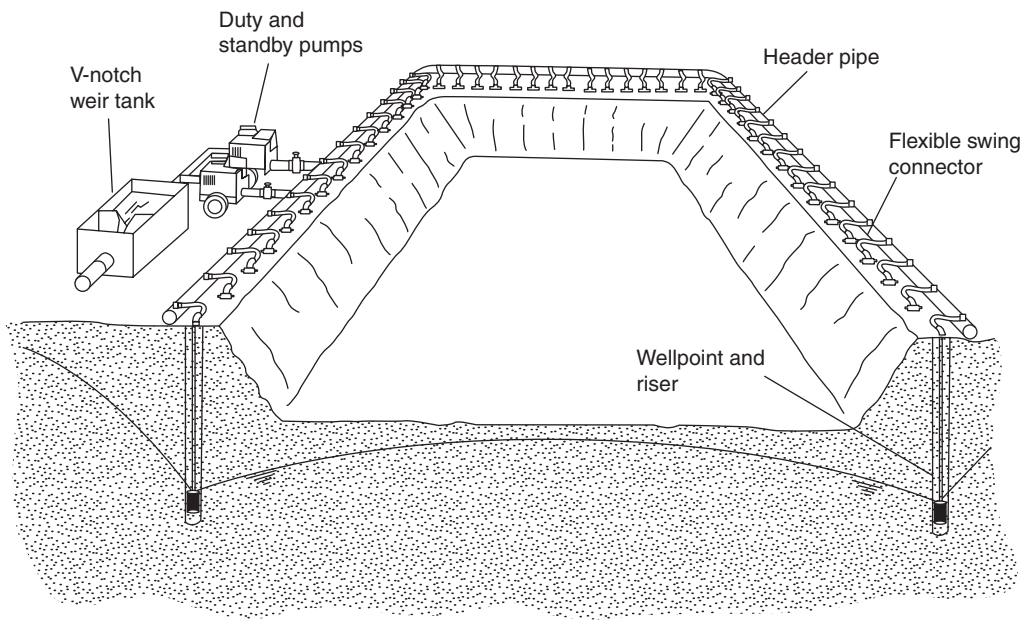


Figure 80.5 Wellpoint system
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settlement, or if an excavation shows signs of instability, sump pumping should be stopped and supplementary or other methods adopted. If the ground loss or instability is serious, it may be necessary to flood the excavation to maintain stability while the situation is reassessed.

80.5.2 Wellpoints

Wellpointing involves closely spaced, small-diameter shallow wells (known as wellpoints) in a ring around the excavation (**Figure 80.5**), or in lines alongside long trench excavations (**Figure 80.6**). The intention is to form a curtain of wellpoints around the excavation in order to intercept the groundwater. If a trench excavation is narrow and shallow, wellpoints may be required on one side of the trench only (single-sided system) but for wider or deeper trenches they will be required on both sides (double-sided system).

Wellpoints are typically installed at intervals of 1.5 to 3 m and linked via flexible 'swing' connectors to a 150 mm diameter header pipe laid alongside the excavation. The header acts as a manifold allowing one pump to act on many wellpoints

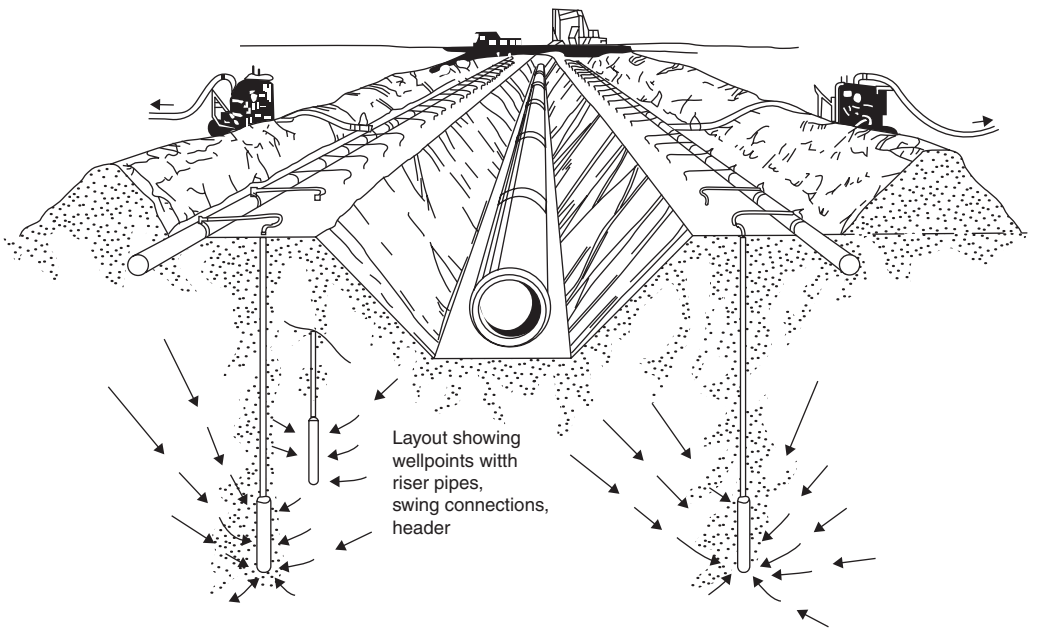


Figure 80.6 Progressive wellpoints for trench work
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(a 150 mm wellpoint pump can typically handle between 60 and 80 wellpoints). The wellpoint pump is located at ground level and applies a vacuum to the header pipe to draw groundwater up the wellpoints, along the header pipe to the pump from where the water is discharged. Pumps may be of a piston variety or may be a centrifugal pump with an ancillary vacuum pump.

Because the pump draws water up out of the wellpoints by suction, there is a practical limit to the drawdown that can be achieved using the wellpoint technique. The suction limit of the pump means that drawdowns of more than 5 or 6 m below the level of the pump and header pipe are rarely feasible. If greater drawdowns are required, a second stage of wellpoints can be installed at a lower level after the first stage has achieved its drawdown (**Figure 80.7**). Alternatively, the first-stage wellpoints and pump could be installed at a lower level after a site strip has been carried out. During operation, care must also be taken that the wellpoints are carefully adjusted or trimmed. Trimming is needed when wellpoints begin to pump excessive quantities of air when groundwater levels are drawn down to the top of the wellpoint screens. Trimming involves throttling the control valve on each wellpoint to reduce the air flow and avoid exceeding the pump's air-handling capacity. A poorly trimmed system will not achieve as much drawdown as a well-trimmed system.

Wellpoints typically consist of 38 mm diameter uPVC tubes with a 0.5 to 1 m long filter screen on the lower end. A steel jetting tube, washed into the ground by high-pressure water, is used to install the wellpoints. The hole around the wellpoint is backfilled with clean filter sand in a process known as sanding-in. The filter sand prevents fine particles being drawn into the wellpoints which could lead to clogging of the screens. Jetting is very quick in sands, but can be slow in gravels or

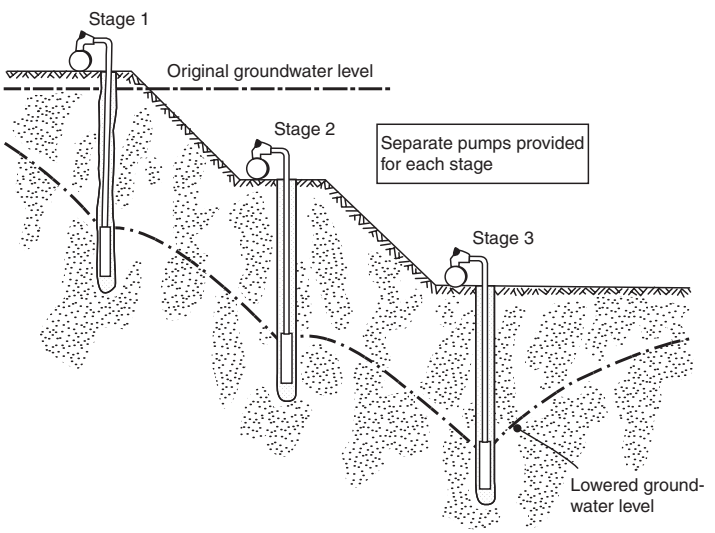


Figure 80.7 Multi-stage wellpoint system
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where cobbles are present. In addition to the high-pressure water supply, in difficult soils a 'hole puncher' with a hammer action can be used together with compressed air. Hydraulic augers are also sometimes used to penetrate stratums of stiff clay. Wellpoints are typically installed to 6 or 7 m below ground level, or occasionally deeper for pressure relief purposes.

For long, shallow trench excavations, horizontal perforated tubes are sometimes installed by a special trenching machine in a technique known as horizontal wellpoint dewatering (**Figure 80.8**). One end of the pipe is pumped on by a wellpoint pump every 100 m or so.

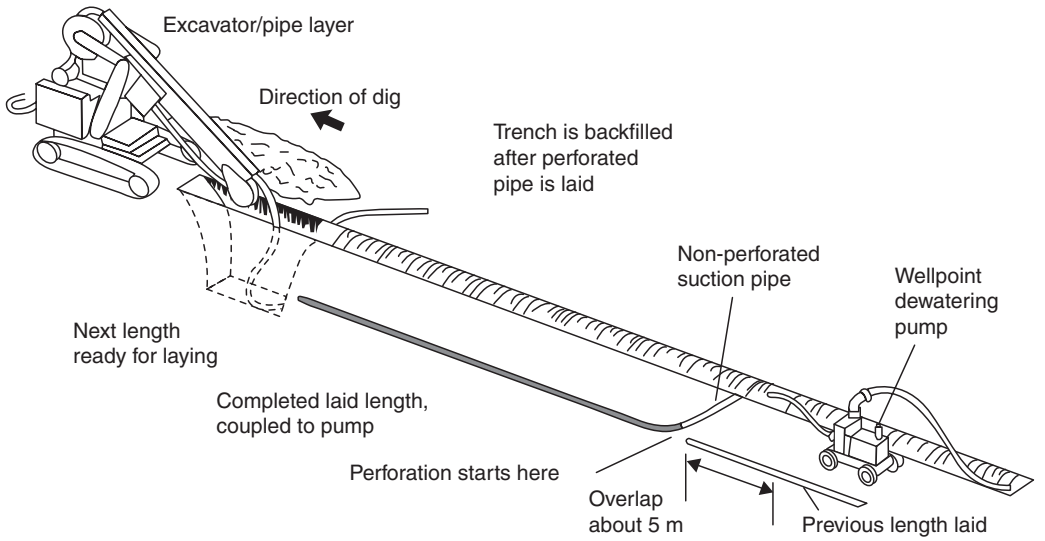


Figure 80.8 Horizontal wellpoint installation
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80.5.3 Deep wells with submersible pumps

A deep well system involves widely spaced deep wells around an excavation, each pumped by an electric submersible pump at depth. Because the submersible pump lifts the water from the base of the well there is no suction limit to the drawdown that can be achieved; theoretically, drawdown is limited only by the depth of the wells or by soil stratification. Unlike wellpointing, where the intention is to create a 'curtain' around the excavation, widely spaced deep wells rely on action at a distance. All the wells act together to cumulatively lower groundwater levels over a wide area.

A typical deep well (**Figure 80.9**) involves drilling a borehole (typically 250 to 350 mm diameter) to between 1.5 to 2 times the depth of the proposed excavation. A well liner (uPVC or HDPE tube) is then installed in the borehole. The lower section of the liner will be slotted or perforated to allow water to enter; this section is known as the well screen. The annulus between the borehole walls and the well screen is backfilled with filter gravel (known as the gravel pack). The annulus above the gravel pack may be filled with bentonite or grout to prevent surface water percolating down into the groundwater. The well itself may be drilled by a cable percussion rig or by a rotary drilling rig. Wells are typically installed at intervals of 15 to 60 m and are linked at ground level by electrical supply cables and water collection pipes.

For a deep well system, the most common layout is to space the wells evenly around the perimeter of the area where control of groundwater is required (**Figure 80.10**). The number of wells required for a scheme may be flexible. A few high-capacity wells or a greater number of smaller wells may give a similar extraction flow and drawdown. A few high-capacity wells may seem more cost-effective but, if there are uncertainties in the ground investigation information or a possibility of perched water, a larger number of smaller wells may result in better control of the groundwater. Also, a scheme with too few wells may be unacceptable if the stoppage of a single pump could cause flooding or even catastrophic failure. Standby electric power supply facilities can be readily provided, but standby pumping plant ready for immediate start-up is rarely supplied for economic reasons. Typically, the solution is to make sure that there is sufficient redundancy in the pumping capacity and that the system is not highly dependent on any one well. This can be a problem for schemes which comprise fewer than 3 or 4 wells. There have to be sufficient wells to draw the water table down. Keeping the groundwater level down may require a reduced flow rate compared with the initial period of pumping.

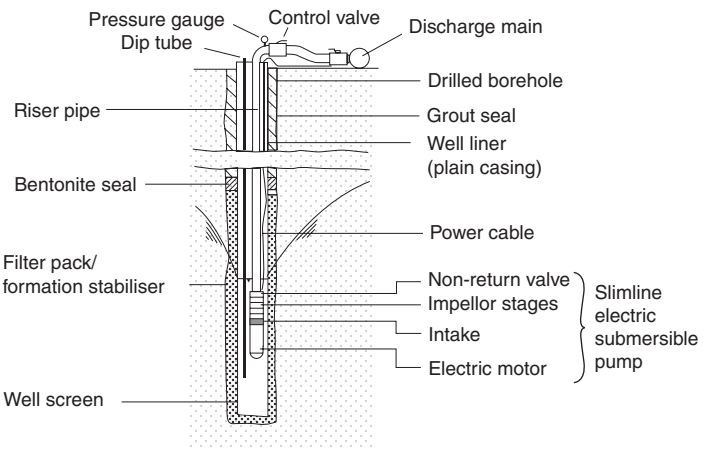


Figure 80.9 Typical deep well
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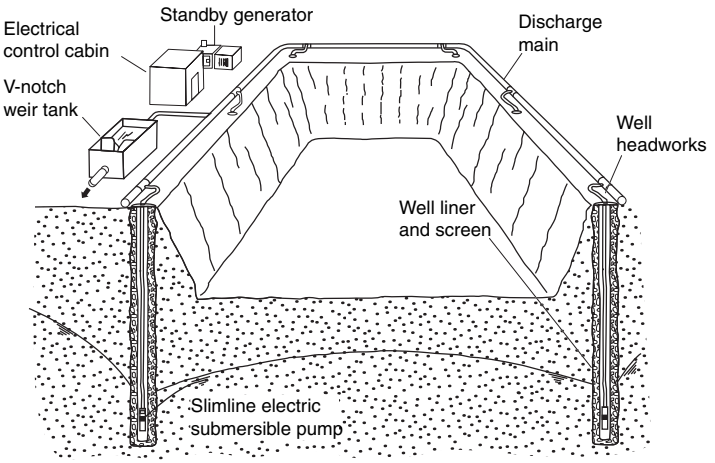


Figure 80.10 Deep well system
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One potential problem which can occur when deep wells are run for long periods is the build-up of a sticky red-brown gelatinous slime in the well and pump. This process is called biofouling and the slime is biomass produced by the iron-reducing aerobic bacteria *Gallionella*, which may occur naturally in groundwater. The lowering of the water level in the well provides the bacteria with an abundant source of oxygen. The *Gallionella* thrive in such conditions and adsorb soluble iron (Fe) present in the groundwater and excrete the resulting insoluble iron oxide and oxyhydroxide compounds. If the biomass is not removed, it can clog well screens and increase wear on pumps dramatically. Cleaning requires the pumps to be removed and washed down; the well can be cleaned by surging with compressed air. Other chemical and bacterial fouling problems can occur: calcium carbonate can leave a thick white paste in the wells, and anaerobic sulfate-reducing bacteria can create a black slime at the bottom of pumps, leading to acidic conditions which may eat into metal pump casings.

80.5.4 Ejector wells

Ejector well systems involve an array of wells around the excavation. Depending on ground conditions the ejector wells are sometimes closely spaced (like wellpoints) and sometimes more widely spaced (like deep wells). The principal distinguishing feature of ejector systems is the method of pumping. An ejector body is installed in each well containing a small-diameter nozzle and venturi. High-pressure water from supply pumps at ground level is supplied to the nozzle. The water passes through the nozzle at a high velocity (up to 30 m/s) creating a pressure drop and generating a vacuum of up to 0.95 bar at the ejector. This vacuum draws groundwater through the well screen to the ejector body, where it joins the supply stream of water and is piped back up to ground level. The water that comes out of the ejector is the supply water plus the groundwater; in simple terms you get more water out of an ejector than you put in. The water from the ejectors goes to a tank feeding the supply pumps and is recirculated to the ejectors (**Figure 80.11**). The excess water abstracted from the ground will start to fill up the tank and a V-notch weir is normally provided to allow the excess to overflow to a suitable disposal point. **Figure 80.11** shows that ejector systems have two header pipes: a supply header containing the high-pressure feed to each well, and a low-pressure return header to carry the recirculated water back to the supply pumps. The ejector body is fitted with a non-return valve which is intended to prevent high-pressure water being inadvertently injected into the ground.

Ejector wells are generally used only in lower-permeability soils such as silty sand or silts where groundwater flow rates are likely to be low. This is because commercially available ejector bodies have a relatively small pumping capacity. A typical ejector might be capable of pumping only 0.3 to 0.8 l/s of groundwater (the supply flow is in addition to this). Higher-capacity ejector bodies are theoretically possible, but are not available because of the low mechanical efficiency of ejectors. For every kilowatt of energy input into an ejector system, less than 25% is used to pump groundwater (the rest is used to circulate supply water and to overcome friction losses). This is a much lower efficiency than either deep wells or wellpoints, so for high groundwater flow rates, instead of designing larger ejectors, it is usually more economical to use deep wells or wellpoints.

A typical ejector well is similar to a deep well but, depending on the type of ejector used, can often be of smaller diameter. Single-pipe ejectors (where the supply and return flows are carried in concentric pipes) can fit down a 50 mm diameter well liner (**Figure 80.12(a)**). Twin-pipe ejectors (where the supply and return flows are carried in separate pipes) can generally fit down a 105 mm diameter liner (**Figure 80.12(b)**). In practice, for the supply pressures normally used of 100 to 160 psi (690 to 1 100 kPa), the drawdown achievable is limited to about 35 m below the level of the supply pump.

Like wellpoint and deep well systems, ejector wells are generally laid out in a ring configuration around the area to be dewatered. Spacing of ejector wells will be determined by the flow rate and capacity of the ejectors used. If the soil stratification indicates the possibility of perched water or overbleed seepage, the well spacing may have to be reduced. In practice, the intervals between ejector wells generally fall between those used for wellpoint systems (1.5 to 3 m) and those used for deep wells (10 m or more).

In a similar way to deep wells, ejector wells are prone to biofouling by bacterial action. Due to the smaller size of the flowpaths inside the ejector, where the biomass can build up, it is possible for an ejector body to become totally clogged up and cease functioning altogether. Regular monitoring of supply and return flow rates and drawdown levels should allow the problem to be identified at an early stage. The ejectors can be cleaned by compressed air surging in a similar way to deep wells.

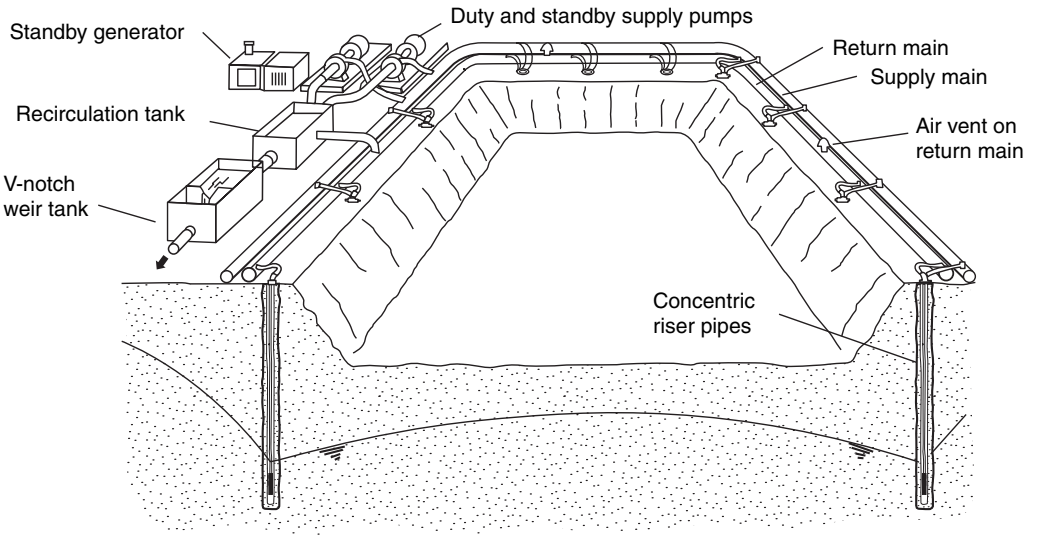


Figure 80.11 Ejector well system
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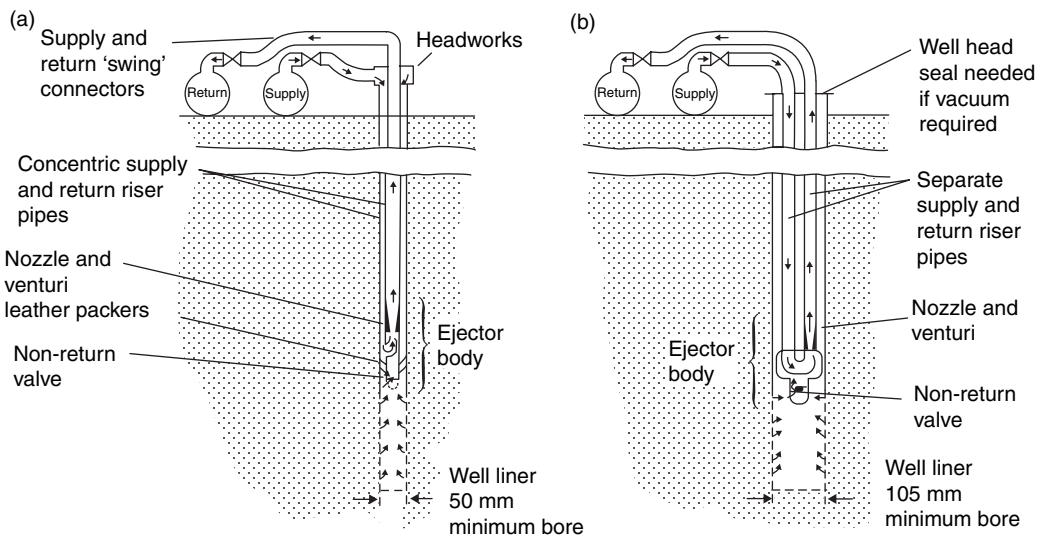


Figure 80.12 (a) Single-pipe ejector body; (b) twin-pipe ejector body
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80.5.5 Other groundwater pumping techniques

80.5.5.1 Relief wells

Relief wells (also known as pressure relief wells) can be used to prevent base heave of excavations (see Chapter 16 *Groundwater flow*), by allowing groundwater to flow upward from pressurised strata beneath an excavation (**Figure 80.13**). Relief wells are an alternative approach to pumped wells (typically located outside the excavation) and comprise a gravel-filled borehole (with or without a perforated liner) within the excavation. The relief well forms a vertical high-permeability pathway; as the excavation progresses groundwater will flow up the relief well into the excavation, thereby reducing pore water pressures below the base of the excavation, and improving base stability. The water entering the excavation is typically disposed of by sump pumping. A simple example of the application of relief wells is given in Ward (1957).

80.5.5.2 Collector wells

A collector well (sometimes known as a Ranney well after a proprietary system) typically comprises a vertical shaft (approximately 5 m in diameter), sunk as a caisson, from which small-diameter horizontal or sub-horizontal screened wells (termed

laterals) are drilled or jacked radially outward (**Figure 80.14**). Typically, a single duty pump is used to pump from the shaft, creating a pressure gradient along the laterals, causing them to discharge into the shaft.

The cost of installing collector wells can be large in comparison to other pumped groundwater control methods; for this reason, collector wells are rarely used in temporary groundwater control applications. Nevertheless, the technique has occasionally been used to lower groundwater levels (on a temporary or permanent basis) beneath areas where there is no surface access to drill conventional deep wells.

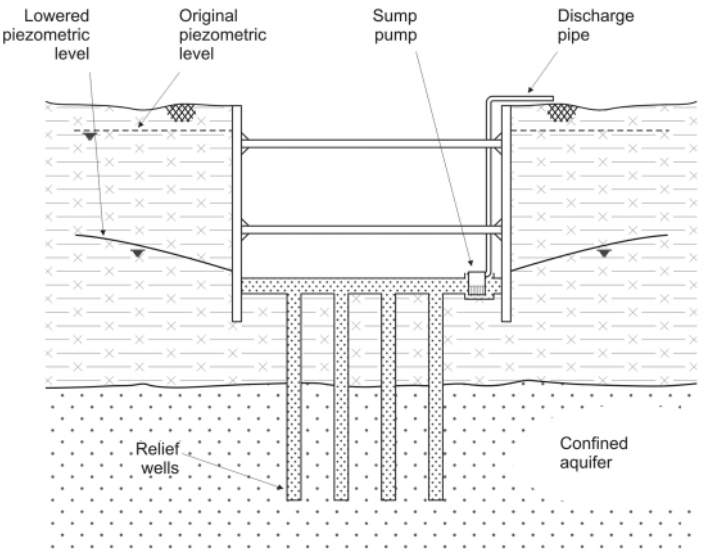


Figure 80.13 Relief well
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80.5.5.3 Artificial recharge systems

Artificial recharge involves injecting (or recharging) water into the ground in a controlled manner via recharge wells or recharge trenches. The intention is to mitigate the potential environmental impacts (including ground settlement) caused by pumping. Such systems need to be designed and operated with care as they must achieve their objective whilst retaining effective groundwater control in the excavation.

Water may be recharged via shallow recharge trenches, or via specially constructed recharge wells. Recharge trenches can be quick and cheap to install but, because of their shallow depth, are only suitable to recharge into unconfined aquifers with water tables near ground level. In contrast, recharge wells can be designed to inject water into specific aquifers, including (if appropriate) confined aquifers at depth beneath a site. A recharge well is essentially similar to a pumping well, but with the following features:

- The annulus around the well casing is typically sealed at the surface with a grout or concrete seal. This is to prevent injected water short-circuiting up to ground level via the filter pack.
- A downspout to prevent the recharge water from cascading into the well and becoming aerated. Aeration of the water may promote clogging processes in the well.
- Air vents to allow air to be purged from the system.
- Control valve and flow meter.

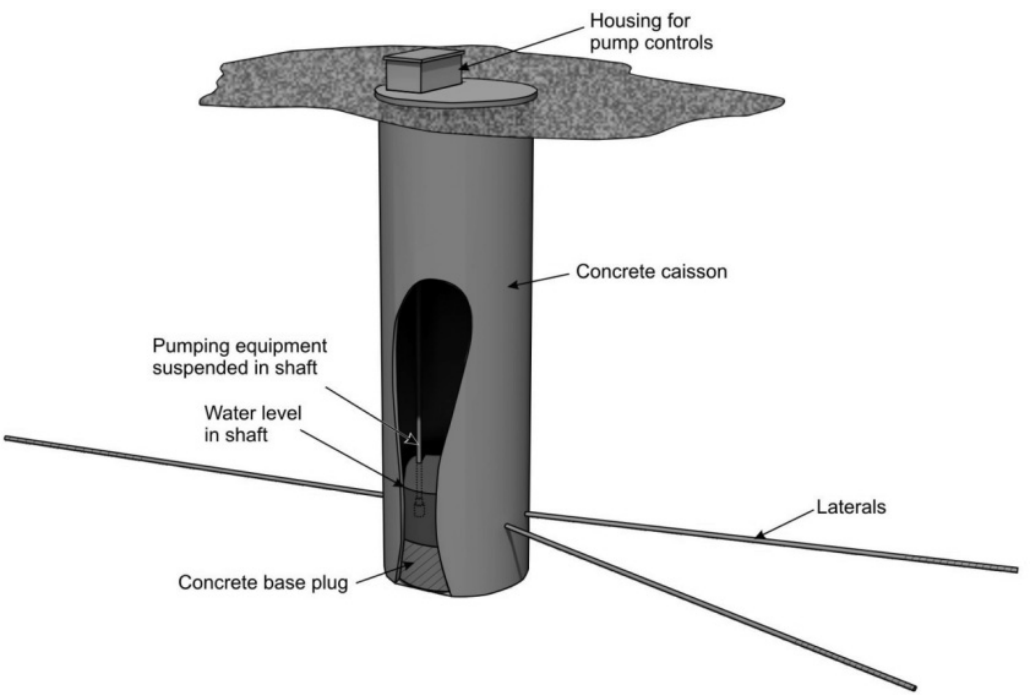


Figure 80.14 Collector well

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A fundamental issue with the operation of artificial recharge systems (wells and trenches) is that they are very vulnerable to loss of performance due to clogging. Clogging may be caused by low levels of suspended solids (sand, silt or clay particles) in the recharge water, or by chemical or biological processes (such as the build-up of iron deposits or bacteriological slimes). It is normal to allow for regular cleaning of recharge wells by backflushing, and to allow for additional recharge capacity – which is necessary due to the long-term deterioration in performance of the wells. Further information on artificial recharge to mitigate the impacts of pumping can be found in Powers (1985). An example of the application of a recharge system is given by Powrie and Roberts (1995). Examples of the use of recharge trenches are given in Cliff and Smart (1998).

80.5.5.4 Electro-osmosis

Pumped groundwater control systems are often ineffective in very low-permeability soils such as silts and clays. This is because groundwater movement under the influence of hydraulic gradients caused by pumping will be excessively slow. In such circumstances, groundwater flow may be more rapid under the influence of electrical potential gradients created by the technique of electro-osmosis.

Electro-osmosis is a highly specialised technique, whereby a direct current is passed through the soil between an array of anodes and cathodes (**Figure 80.15**). The electrical potential gradient causes positively charged ions and water molecules around the soil particles to migrate from the anode to the cathode. The small volumes of water generated can be pumped away from the cathode by wellpoints or ejectors.

The main application of electro-osmosis is for the stabilisation of excavations in very soft silts or clays. Further information can be found in Casagrande (1952) and Casagrande *et al*. (1981).

80.5.5.5 Pumping and exclusion methods used in combination

Pumped groundwater control methods are sometimes used in combination. Common examples include:

- Pumping to dewater cofferdams – even where the soil stratification allows a cut-off wall to form a complete cofferdam to exclude groundwater (**Figure 80.2(a)**), some residual groundwater will remain trapped in the excavation. Groundwater pumping from within the excavation will be necessary to prevent this water from interfering with construction operations.
- Cut-off walls used to reduce pumped flow rates – in very high-permeability soils, pumped flow rates from excavations will be very large. It may be appropriate to use cut-off walls penetrating below excavation level (**Figure 80.2(b)**). If the abstraction points (wells or sumps) are located within the cut-off walls, the pumped flow rate will be reduced to some degree, potentially reducing pumping costs and making the pumped discharge easier to dispose of.
- Groundwater lowering to reduce loading on cut-off structures – where the sides of an excavation are supported by cut-off walls such as sheet piles or diaphragm walls, a significant proportion of the lateral loading on the walls may be due to external groundwater pressures (see Chapter 63 *Principles of retaining wall design*). In some circumstances, pumped dewatering wells are located outside the excavation to lower groundwater levels and reduce loading on the retaining structure. This can allow overall cost savings to be made by reducing the necessary wall stiffness and propping requirements (Roscoe and Twine, 2001).

80.6 Design issues

The design process needs to address hydrogeological factors in relation to the calculation of pumped flow rates, environmental

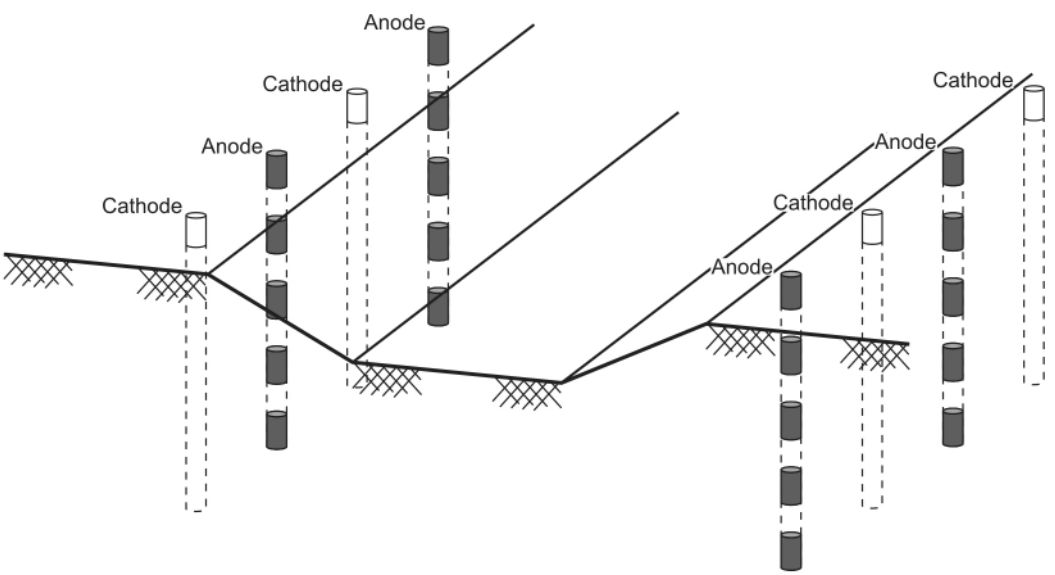


Figure 80.15 Electro-osmosis
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impacts, etc., but must also address the performance and selection of suitable technologies. In all but the simplest of groundwater control problems, the following steps should be followed as part of the design process:

(i) definition of problem and constraints;

(ii) development of hydrogeological conceptual model;

(iii) selection of method of groundwater control;

(iv) design calculations;

(v) assessment of environmental impacts;

(vi) review of design.

Each of these stages is briefly discussed in the following sections. On large and complex projects, the observational method is sometimes used (see Chapter 100 *Observational method*). Further details of design methods are given in Preene *et al.* (2000) and Cashman and Preene (2001).

80.6.1 Definition of problem and constraints

To allow a groundwater control system to be designed, three aspects must be addressed:

Define objectives of groundwater control The objectives must be established at the start. Is there a need to prevent inundation of excavations in permeable soils, or is pore water pressure control needed to ensure slope stability in fine-grained soils, or is there a need to reduce groundwater uplift pressures to prevent base heave in each of these? Such differing objectives may drive the design down different avenues.

Identify key constraints Groundwater control is not carried out in isolation; it must integrate with the rest of the construction project. Are there space and time constraints for the site works? Is the site in a sensitive environmental setting where prolonged pumping may cause problems (see section 80.6.5).

Gather and review available information It is important to collate information at an early stage to understand what is known about the site, and hence what is not known or is uncertain. Even if recent site investigations have been carried out, there may be uncertainty over the permeability of soil and rock at the site.

80.6.2 Hydrogeological conceptual model

A key stage in the design process is the development of the hydrogeological conceptual model relevant to the groundwater control problem, to provide the information necessary for design. If the conceptual model is a poor match for the actual conditions, any subsequent design work is likely to be of little value. If there is insufficient reliable data to formulate a convincing model, this could be a sign that further site investigation is needed. Some key factors in the conceptual model are outlined below.

80.6.2.1 Aquifer types and properties

The aquifer (water-bearing strata in which groundwater levels are to be lowered) must be defined and its nature identified. On major projects there may be a requirement to involve expert hydrogeologists to characterise groundwater conditions. Further details on the characteristics of aquifers can be found in hydrogeological texts such as Younger (2007).

80.6.2.2 System geometry

The depth and size of the excavation will have a key influence on the design of a groundwater control scheme. In particular, the depth of the deepest part of the excavation below groundwater level is a key parameter, which must be determined.

80.6.2.3 Soil permeability

The permeability of the aquifer is a critical parameter in the design of groundwater control systems, and must be assessed as part of the design process. Methods of estimation of permeability are described in Chapter 47 *Field geotechnical testing*. Permeability can be very difficult to determine accurately, either from laboratory or *in situ* tests; uncertainties in permeability can lead to major variations in calculated flow rates. It is often necessary to carry out sensitivity analyses to assess the impact of different permeability values on design.

80.6.2.4 Aquifer boundary conditions

Aquifers are rarely uniform and will typically have physical or hydraulic boundaries which influence their behaviour. Any significant boundary conditions need to be identified and accounted for in design. Examples of boundary conditions include low-permeability clay layers which may impede vertical water flow within sandy aquifers, or bodies of surface water (such as rivers, lakes or the sea) which may act as large local sources of water for an aquifer.

80.6.3 Selection of method of groundwater control

Correct selection of the method of groundwater control is essential for successful design. It is likely to be very expensive and disruptive to change methods and technologies at any stage after initial design.

A key step is to identify whether the most appropriate approach is groundwater control by pumping or by exclusion, or a combination of the two. This decision is likely to be influenced by a range of factors, including:

- technical issues such as order of magnitude of pumped flow rates, likely depth of cut-off to low-permeability layers (some scoping calculations may be needed at this stage);
- space or program issues for the main construction works;
- requirement to minimise external groundwater impacts (which would tend to favour exclusion options);
- availability of water disposal routes of adequate capacity (if no disposal routes are available, pumping options are unlikely to be feasible);
- cost and availability of experienced contractors for the various methods.

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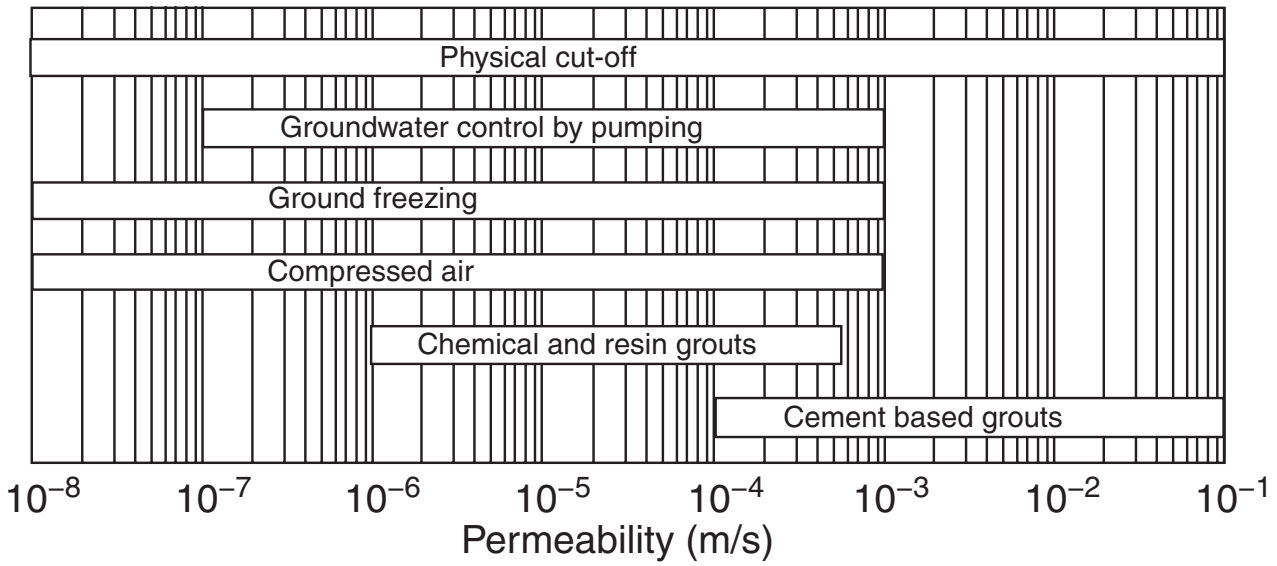


Figure 80.16 Approximate range of application of groundwater exclusion techniques in soils
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Once the method (exclusion or pumping) has been selected, the initial technology must be chosen. As the choice of technology will affect the design process, it must be selected early and then reviewed periodically during the design process to confirm it is appropriate. For groundwater control by pumping, initial technology selection can be made from **Figure 80.3**, which relates the range of application of pumped dewatering methods to the required drawdown and estimated soil or rock permeability. For groundwater control by exclusion, initial technology selection can be made from **Figure 80.16**, which relates cut-off methods to estimated soil or rock permeability.

80.6.4 Design calculations

Design calculations for pumped groundwater control systems are described in Preene *et al.* (2000) and Cashman and Preene (2001). In simple cases the design will be carried out using seepage calculations derived from classic Darcy's Law flow models; see Chapter 16 *Groundwater flow*, applied to appropriate boundary conditions. In more complex cases, numerical modelling may be used to model complex groundwater flow regimes. Occasionally the observational method (see Chapter 100 *Observational method*) may be used for groundwater control schemes. The observational method as applied to groundwater control by pumping is described in Roberts and Preene (1994).

80.6.5 Assessment of environmental impacts

Groundwater control has the potential to have measurable effects at considerable distances from the excavation which are dependent on the hydrogeological setting. These effects need to be assessed at design stage so that any necessary mitigation measures can be identified.

Pumping of groundwater can lower groundwater levels around a site. Occasionally, groundwater levels may be lowered by small amounts several hundred metres from the site. Cut-off walls installed as part of groundwater exclusion schemes may act as underground dams and may cause local changes in groundwater levels.

Table 80.3 summarises the range of potential impacts from groundwater control works. The more significant impacts include:

- reduction of yield from nearby water supply wells as a result of dewatering pumping;
- settlement of nearby structures as a result of effective stress increases caused by groundwater lowering (Preene, 2000);
- increased movement of contaminated or saline groundwater as a result of dewatering pumping;
- increases in groundwater level as a result of the installation of cut-off walls.

Guidance prepared by UK regulators is available (Boak *et al.*, 2007) on methodologies for hydrogeological impact appraisals (HIA) to be applied to dewatering abstractions. The HIA approach is often 'tiered' – with different levels of complexity of assessment, dependent on the nature of the project. Simple HIAs may be little more than a desk study to confirm there are no sensitive receptors nearby. However, a small number of large and complex projects may require extensive numerical modelling and field monitoring.

Further details of potential environmental impacts and associated mitigation measures related to groundwater control activities are given in Preene and Brassington (2003).

80.6.6 Review of design

It should be recognised that even when a thorough site investigation and design have been carried out, uncertainty may

Category	Potential impacts	Duration	Relevant construction activities
(1) Abstraction	Ground settlement Derogation of individual sources Effect on aquifer – groundwater levels Effect on aquifer – groundwater quality Depletion of groundwater-dependent features	Temporary	Dewatering of excavations and tunnels using wells, wellpoints and sumps Drainage of shallow excavations or waterlogged land by gravity flow
		Permanent	Permanent drainage of basements, tunnels, road and rail cuttings, both from pumping and from gravity flow
(2) Pathways for groundwater flow	Risk of pollution from near-surface activities Change in groundwater levels and quality	Temporary	Vertical pathways created by site investigation and dewatering boreholes, open excavations, trench drains, etc.
			Horizontal pathways created by trenches, tunnels and excavations
		Permanent	Vertical pathways created by inadequate backfilling, sealing of site investigation, dewatering boreholes, excavations, permanent foundations, piles and ground improvement processes
			Horizontal pathways created by trenches, tunnels and excavations
(3) Barriers to groundwater flow	Change in groundwater levels and quality	Temporary	Barriers created by temporary or removable physical cut-off walls such as sheet-piles or artificial ground freezing
		Permanent	Barriers created by permanent physical cut-off walls, or groups of piles forming part of the foundation or structure, or by linear constructions such as tunnels and pipelines
			Barriers created by reduction in aquifer hydraulic conductivity (e.g. by grouting or compaction)
(4) Discharge to groundwaters	Discharge of polluting substances from construction activities	Temporary	Leakage and run-off from construction activities (e.g. fuelling of plant)
			Artificial recharge (if used as part of the dewatering works)
		Permanent	Leakage and run-off from permanent structures Discharge via drainage soakaways
(5) Discharge to surface waters	Effect on surface waters due to discharge water chemistry, temperature or sediment load	Temporary	Discharge from dewatering systems
		Permanent	Discharge from permanent drainage systems

Table 80.3 Impacts on groundwater conditions from civil engineering works
Reproduced from Preene and Brassington (2003) from John Wiley & Sons Ltd

remain – perhaps in relation to values of soil permeability, or the magnitude of potential environmental impacts. It is important that a brief review is carried out at the end of the design process to consider if the remaining uncertainty is acceptable in the context of the proposed project. If necessary, the design may need to be revised, or additional ground investigation information gathered.

80.7 Regulatory issues

80.7.1 Abstraction licensing and discharge consents

In the UK, groundwater control by pumping is subject to specific Government legislation and is overseen by regulatory authorities. These authorities are: Environment Agency (EA, England and Wales); Scottish Environment Protection Agency (SEPA, Scotland); and Northern Ireland Environment Agency (NIEA, Northern Ireland). Publications are available which outline the regulatory approach (EA, 2008).

There are two facets to the legal requirements. The first deals with pumping of groundwater (termed abstraction) and is intended to make sure the regulators can control groundwater use so that groundwater control systems do not cause nearby groundwater users to lose their supplies. The second deals with disposal of groundwater (termed discharge) and is intended to ensure that the pumped water does not itself cause pollution. Up-to-date guidance on legal requirements can be found on the websites of the regulators.

80.7.2 Management of discharge water quality

Adequate management of the discharged water is essential for any groundwater control scheme. Discharge consents or permissions are required for all groundwater discharges.

Disposal options for discharge water include:

- To surface waters (e.g. river, water course, lake, sea). Consent is required from the regulators depending on the site location.

- To groundwater (e.g. via soakaways, recharge wells, or recharge trenches). Consent is required from the regulators depending on the site location.
- To an existing sewerage network. Permission is required from the sewer authority (e.g. water utilities or their agents).

Silt pollution caused by poorly managed pumped discharges from groundwater control operations is a common problem, typically resulting from poorly planned and executed sump pumping operations. It can harm the aquatic environment in several ways, including:

- injuring fish by its abrasive action;
- clogging the gills of fish, causing them to die by suffocation;
- destroying spawning sites and insect habitats on the river bed, removing the source of food for fish;
- coating the leaves of aquatic plants, limiting their growth.

The ideal way to manage suspended solids in discharges is to tackle the cause of the problem and to design and specify the groundwater control system with adequate filters to minimise sediment in the discharge water. Provided that suitable filters are installed, wellpoint, deep well and ejector well systems do not normally produce discharges with high sediment content, except during the initial periods of pumping and development when dirty water may be produced for short periods.

The method which most commonly produces sediment-laden water is sump pumping (section 80.5.1). Installation of adequate filters around sumps can be difficult and, as a result, clay, silt and sand-size particles can be drawn to the pump and entrained in the discharge water. Whenever sump pumping is carried out, arrangements should be made before final discharge to remove any suspended solids to below the maximum levels set in the discharge consent (by using, for example, appropriately designed and operated settlement tanks or ponds). If this is not possible, it may be necessary to change to another groundwater control method, such as wellpoints, with adequate filters.

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80.8.1 Useful websites

Construction Industry Research and Information Association; www.ciria.org
Environment Agency; www.environment-agency.gov.uk
Northern Ireland Environment Agency; www.ni-environment.gov.uk
Scottish Environment Protection Agency; www.sepa.org.uk

It is recommended this chapter is read in conjunction with

- Chapter 15 *Groundwater profiles and effective stresses*
- Chapter 16 *Groundwater flow*
- Chapter 46 *Ground exploration*
- Chapter 100 *Observational method*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 81

Types of bearing piles

Steve Wade Skanska UK Plc, Rickmansworth, UK
Bob Handley Aarsleff Piling, Newark, UK
James Martin Byland Engineering, York, UK

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Piles are slender, columnar foundation elements. They are most commonly designed, as discrete bearing piles, to transfer part or all of the vertical loading imposed by a structure through weak and compressible soils to competent and stable soils, or rock. This chapter describes the most commonly available types of bearing pile and discusses the methods of construction and the advantages and constraints of individual techniques. Other applications of piling are discussed in Chapters 83 *Underpinning* and 85 *Embedded walls*. Section 81.2 describes the plant and a range of methods used in the construction of bored piles, including rotary cast-in-place, continuous flight auger and displacement auger techniques. Brief discussion of some specialised techniques including under-reaming, base grouting, embedded columns and geothermal piles completes this section. Driven piling techniques including timber, segmental precast, prestressed precast and driven cast *in situ* concrete are described in section 81.3, along with steel H and box section piles and steel tube piles. A discussion of modern driven piling plant is set in a historical context. The impact of ground conditions, environmental aspects and particular construction issues such as drivability, installation control and the effects of pile driving on the ground are also discussed. Finally section 81.4 discusses the wide-ranging application of micro-piling techniques. Practical aspects of various construction methods are discussed, covering rotary case and auger, rotary duplex, rotary down-the-hole-hammer, rotary percussive, hollow-segmental-auger and driven micro-piling. The section is completed with discussions of the grouts used in micro-piling and the particular environmental, quality assurance and health and safety issues that apply.

81.1 Introduction

The use of piles as a foundation solution is in itself a key decision in the design process (see Section 5 *Design of foundations* of this manual). Once the use of piles has been identified, it is important to consider which forms of pile and methods of construction are most appropriate to the project. The selection of appropriate pile types should take place early in the foundation design if time is not to be lost pursuing uneconomical or impracticable solutions. The constraints of a particular project may dictate that only a narrow range of pile types is applicable, or may leave room for a wider choice of solution, which can then be based on more environmentally sustainable and economic grounds. The factors affecting the selection of pile type are many and varied, and are often interrelated. They include:

Performance	Environmental	Site constraints
• Bearing capacity	• Noise	• Restricted access
• Uplift capacity	• Vibration	• Restricted headroom
• Lateral load capacity	• Spoil disposal	• Restricted working hours
• Durability	• Contamination	
	• Carbon efficiency	• Existing assets and structures

Safety	Geotechnical
• Work next to operational railway	• Very weak strata (e.g. peat)
• Airside work at airports	• Deep unstable strata
• Work adjacent to sensitive assets	• Water-bearing strata
• Work on sloping sites	• Obstructions
	• Rock

These factors are commonly in conflict and a degree of engineering judgement is required to identify the most appropriate solution. A commercial and residential tower development, for instance, may call for relatively few piles with correspondingly high axial and lateral loading. On a site with reasonably open access large diameter bored piles (see section 81.2.1) would be suitable. On the other hand, if the site is subject to severe access, constraints and headroom limits, the installation of large diameter piles may be uneconomical or even impracticable. This may therefore dictate that a micro-piled solution (see section 81.4) is adopted. Such conflicts can have profound implications for the design of the sub-structure, and in some cases, the feasibility of development overall.

In another case, a relatively open site may be underlain by a significant volume of contaminated soil. Here the use of displacement piles (which avoid the need for off-site disposal of spoil) would present itself as an attractive solution. On a site where vibration is not an issue this might be most economically achieved with a driven form of pile (see section 81.3). However, adjacent to a vibration-sensitive installation, or in a residential location, a vibration-free displacement auger pile solution (see section 81.2.4) may be more appropriate.

In practice it is found that, however similar, no two projects are quite the same in regard to the founding solution. The piling solution for an extensive residential development, for example, may deploy a driven pile solution in the first instance. As the development grows and some sectors are populated, even though the performance requirements and ground conditions remain the same, later adjacent plots may require an

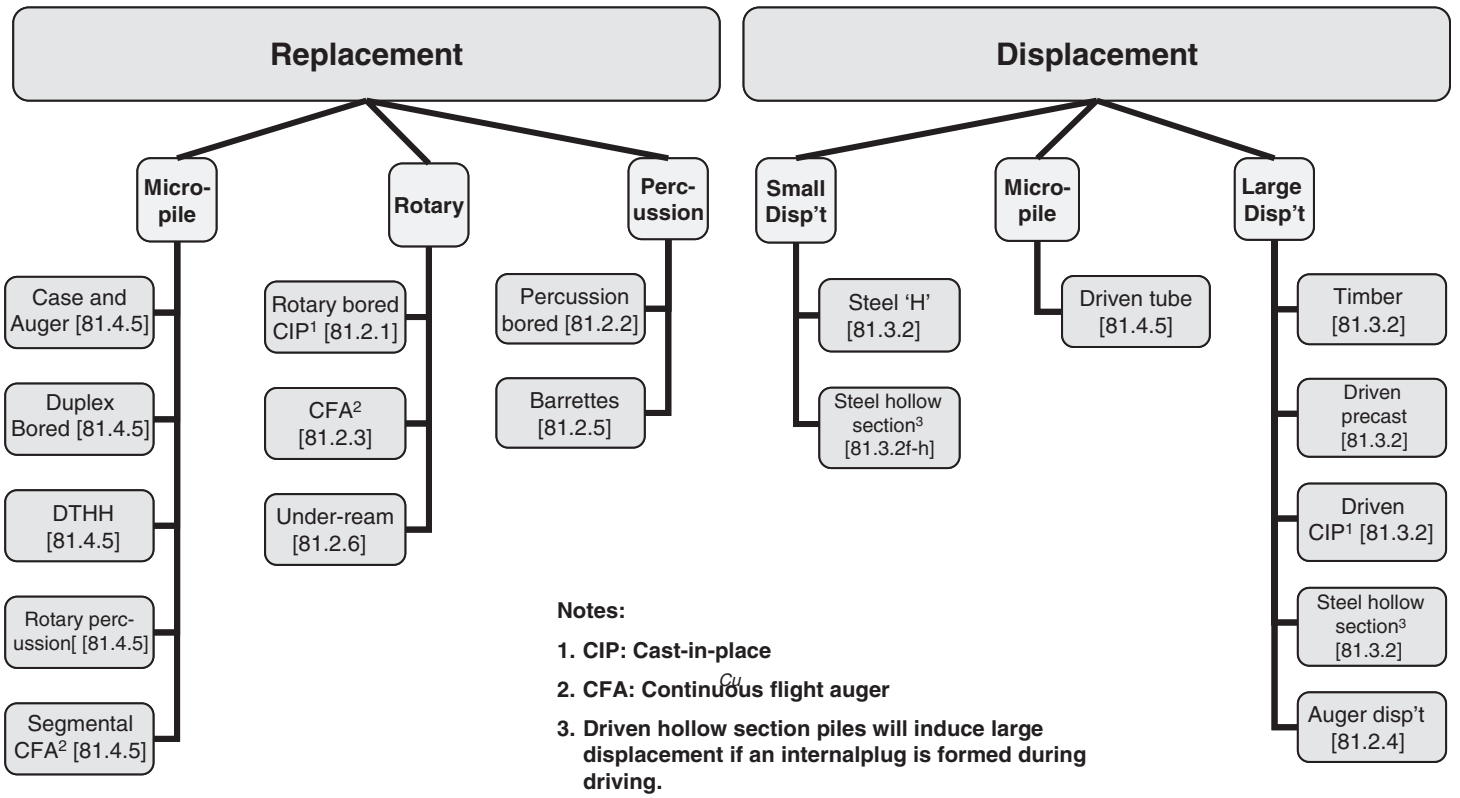


Figure 81.1 Broad classification of common bearing pile types (with reference to relevant sections of this chapter)

alternative solution such as CFA piling (see section 81.2.3) in order to keep vibration or noise to acceptable levels.

Figure 81.1 presents a simple classification of pile types, categorising each according to whether installation involves replacement or displacement of the soil (as originally proposed in BS 8004:1986 – now superseded). The figure includes references to the relevant sections in this chapter, where each pile type is briefly discussed.

81.2 Bored piles

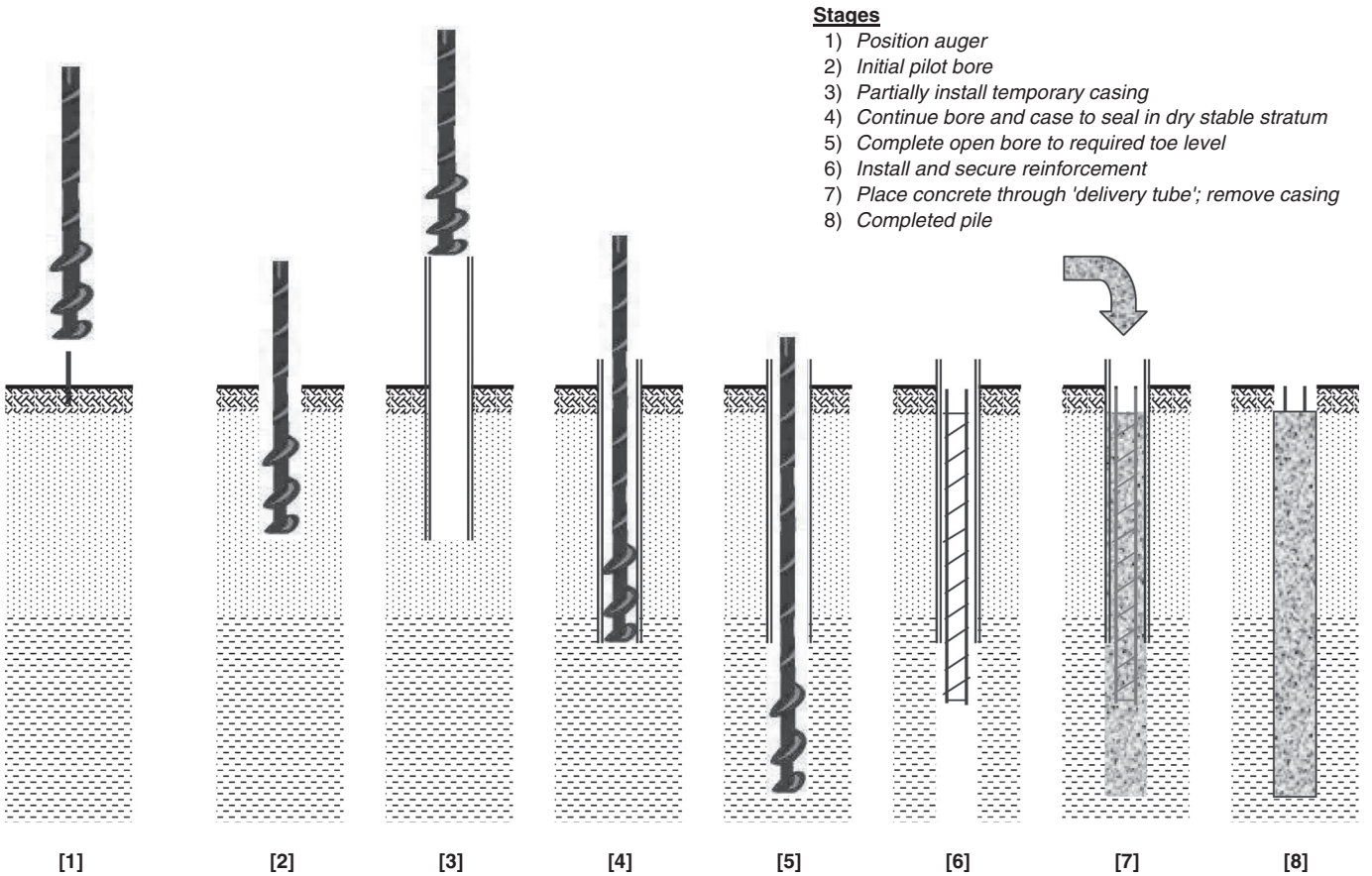
Bored pile construction in the UK is generally carried out in accordance with the *Specification for piling and embedded retaining walls* (SPERW) (Institution of Civil Engineers, 2007) and is subject to the requirements of the European Execution Code, BS EN1536 (British Standards Institution, 1999). A bored pile is defined in BS EN1536 as 'a pile formed, with or without casing, by excavating or boring a hole in the ground and filling with plain or reinforced concrete'. This section describes the processes involved in constructing bored piles and gives illustrations of the plant most commonly used in the UK for this purpose. Some more specialised methods of construction are also discussed. Summaries of typical construction tolerances and the most commonly available pile sizes and depth ranges are given. The particular case of bored piles installed with micro-piling equipment is described in section 81.4.

Bearing piles are most commonly used to transfer structural loads through weaker near-surface soils, to more competent strata at depth. Unsupported bores and excavations through weak and unstable strata, such as very soft clays, highly organic soils, highly fractured rock or non-cohesive soils (silts, sands, gravel, especially if they are water-bearing), can result in uncontrolled inflows of soil and water. This may create unstable cavities outside of the pile shaft, undermine adjacent footings and pose a risk to the ultimate integrity of the concreted shaft (see Chapter 81 *Piling problems*). In unstable soils and beneath the water table it is therefore necessary provide a means of temporary or permanent support. The means of support may take the form of:

- temporary or permanent casings;
- drilling/stabilising fluid;
- soil-filled auger flights.

These stabilisation methods, coupled with the types of tools and processes used to excavate the piles, provide the key distinctions between the different forms of bored piles that are described below.

The majority of bored piles installed in the UK are constructed using either rotary bored and cased methods or continuous flight auger (CFA) techniques. Variations of these methods, such as tripod bored piles, circular clamshell grabs and down-the-hole hammer (DTHH) techniques, are also



Notes:
i Temporary casing may be installed in a single length (as shown) or segmentally as the bore proceeds.
ii Reinforcement generally extends beyond casing to prevent cage uplift during casing extraction.
iii Concrete 'delivery tube' should be long enough to direct concrete and avoid segregation by impact on steel (ICE SPERW B3.5.2.3).
iv Concrete should be cast to a level that ensures that a sound concrete connection can be made after trimming to the specified level (ICE SPERW Table B1.5).

Figure 81.2 Basic construction sequence for a rotary bored cast-in-place pile

sometimes used to suit particular sub-soil and environmental conditions. Increasing use is also being made of cast-in-place displacement piles, particularly to support relatively light structural loads. These distinct bored piling methods are discussed below. Heavy vertical loads, and particularly high lateral loads, may justify the use of barrettes, constructed using rectangular clamshell grabs and diaphragm walling techniques (see Chapter 85 *Embedded walls*). Chapters 79 *Sequencing of geotechnical works* and 82 *Piling problems* include discussion of the need for stable working platforms, the sequencing of pile installation and the problems that can arise during and following pile construction.

81.2.1 Rotary bored cast-in-place piles

The basic process of constructing a rotary bored cast-in-place pile is shown in **Figure 81.2**.

After setting the rig up over the pile location [1] an initial pilot hole is bored [2] using a short, flighted auger, mounted on a kelly bar. A temporary casing is then inserted [3] and the bore and casing are progressed together until a seal into a stable, dry stratum is achieved [4]. The bore is then completed to the required depth [5], the auger is withdrawn, and the reinforcement cage is lowered into position and secured [6]. Concrete is placed in the bore to the required level [7] and the temporary casing is withdrawn. Other types of bored cast-in-place pile generally follow this basic process but with variations to one or more aspects of construction to meet particular project-specific constraints.

The particular case of CFA piling is discussed in section 81.2.3 below.

81.2.1.1 Bored piling rigs

Bored piling rigs have developed considerably over the past 30 years in response to the changing and increasing demands made by the construction industry. The crane-mounted rigs that dominated rotary piling until the 1980s (**Figure 81.3(a)**) have



Figure 81.3 Rotary bored cast-in-place piling rigs: (a) crane-mounted rotary piling rig; (b) light–medium duty hydraulic rig; (c) heavy duty hydraulic rig
Photos courtesy of Cementation Skanska Ltd

Pile type	Rig characteristic	Range of diameters (mm)[1],[2]	Maximum depth (m)[1],[2]	Typical axial load range[3]
Rotary bored	Light rotary (< 35 t)	300–1200	18–35	< 5000 kN
	Medium rotary (40–80 t)	< 1800	40–60	< 20 MN
	Heavy rotary (> 100 t)	> 2400	< 90	> 30 MN
Percussion bored	Tripod	300–600	< 30	< 600 kN
CFA	Light CFA (< 35 t)	300–750	18–24	< 3000 kN
	Medium CFA (40–80 t)	450–1200	24–28	< 7500 kN
	Heavy CFA (> 100 t)	450–1200	< 32	< 10 MN
Cased-CFA	Twin-drive (> 100 t)	600–1200	casing: 18 m; bore: 25 m	< 10 MN
Auger displacement	Rotary (> 60 t)	400–600	25	< 2000 kN

Notes:
The above figures are given as a preliminary guide only. The advice of specialists should be sought before selecting the particular pile type, plant type and method of construction.
[1] Maximum achievable pile diameters and depths vary according to the plant manufacturer. Figures shown are for guidance only.
[2] In addition to the rig size and power, the available pile diameters and depths may also be limited by the ground conditions.
[3] Irrespective of pile diameter and depth, the achievable axial loads are dependent on ground conditions.

Table 81.1 Typical bored pile characteristics

been steadily replaced by purpose-built, self-erecting, vertically masted hydraulic units (**Figure 81.3(b)**). Simultaneously, as the demands of the commercial and infrastructure sectors have risen, increasingly powerful rigs have been developed, capable of installing large diameter piles to depths in excess of 60 m (**Figure 81.3(c)**). Tomlinson and Woodward (2008) provide a useful summary of the characteristics of self-erecting hydraulic rigs from various suppliers. Commonly available pile diameters and depths are listed in **Table 81.1**.

81.2.1.2 Excavation tools

A wide range of tools are available for use with rotary piling rigs to excavate pile shafts through the variable ground conditions found beneath UK sites (**Figure 81.4**). One or more of these tools may be used during the construction of an individual pile.

Augers are generally used to excavate soft ground (sands, clays, gravels, etc.) and some weak rocks (including chalk and marl). In hard ground a variety of chisels may also be deployed to break up the material at the base of the bore before removal

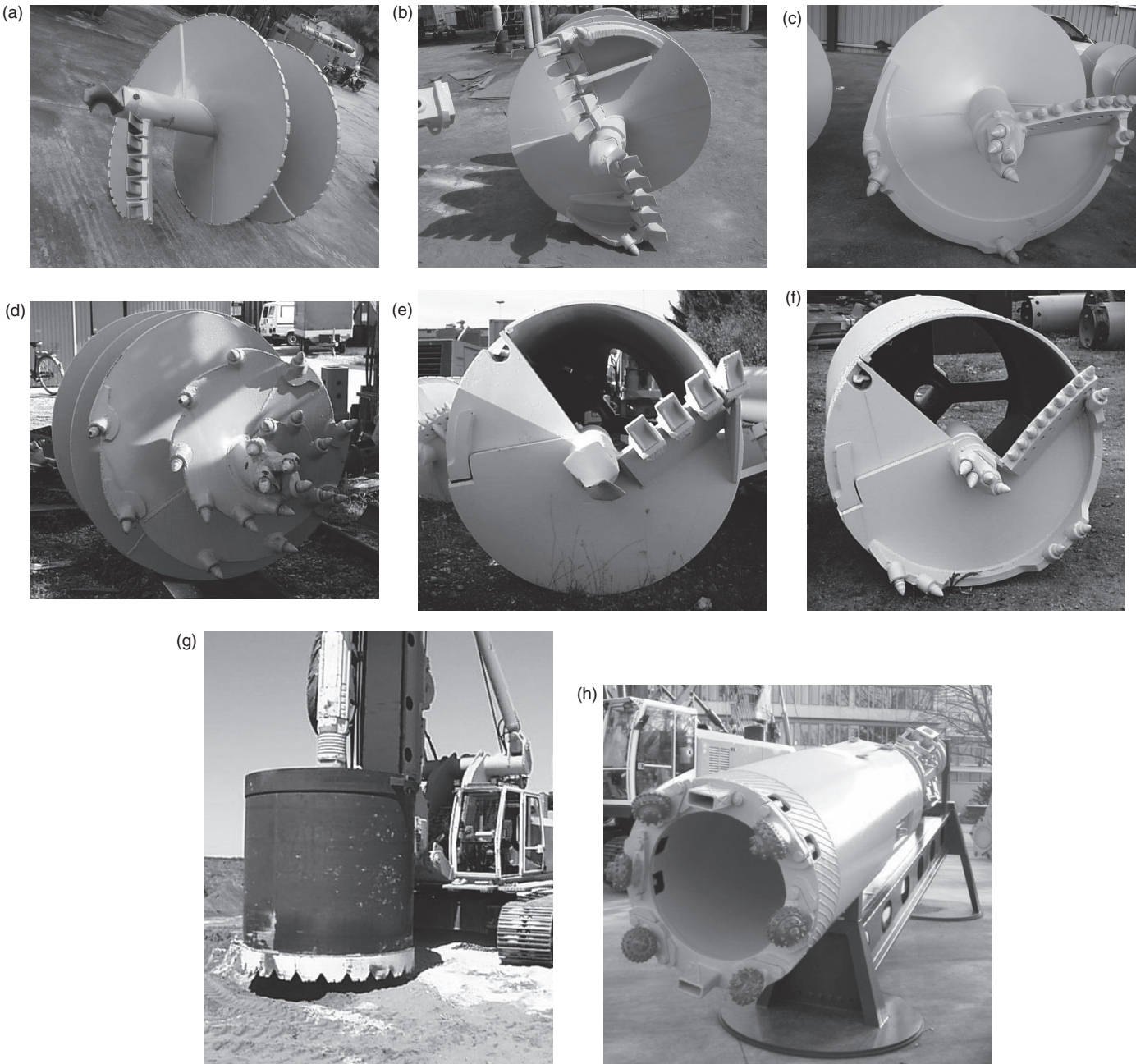


Figure 81.4 Rotary bored piling – excavation tools: (a) Single start soil auger; (b) double start soil auger; (c) single start rock auger; (d) single start progressive rock auger; (e) single cut digging bucket; (f) single start rock digging bucket; (g) large diameter coring barrel; and (h) heavy duty coring barrel with air flush

Photos courtesy of Bauer Equipment UK Ltd

by auger. Specially designed toothed augers can also be used to excavate stronger rocks, although production rates fall significantly at strengths above 10–15 MN/m^2, depending on the type of rig and tool design being used.

Drilling buckets are designed to recover materials that are not easily retained on an auger during retrieval from the bore. This can arise in water-bearing soils and fractured rocks. Specially designed buckets may be used when drilling under stabilising fluid, to minimise turbulence during insertion and retrieval of the tool, and avoid the resulting shaft and base instability. Where appropriate a drilling bucket may also be used to clean the base of the pile before concreting. Some caution is required in cohesive soils, however, since this can create a softened layer at the pile toe thereby diminishing the end-bearing response of the pile.

An alternative means of excavating pile shafts, not commonly used in the UK, is to employ a circular-section, clamshell grab or hammer grab. These tools are rope-suspended

from a tracked crane and can be combined with segmental casing techniques (see below) to excavate through most soils, and form sockets in weak rocks (using chisel attachments).

Harder and more massive rocks call for alternative methods:

- coring barrel;
- rock roller assemblies coupled with air flush or reverse-circulation systems to remove the cuttings (described in greater detail by Tomlinson and Woodward, 2008).

81.2.1.3 Casings

Temporary casings most commonly take the form of single-wall steel tubes, installed in one piece as illustrated in **Figure 81.5(a)**. These may be installed and removed from the bore using the piling rig or an attendant crane, with or without the assistance of a casing vibrator.

Where closer control of the boring process is required segmental casings may be used. These are stiff double-walled casings, typically 40–60 mm thick, mechanically jointed in 1–6 m lengths, with a cutting head at the leading edge (**Figures 81.5(b)–(c)**). Segmental casings can be installed to significant depths using the rotary drive of medium and heavy duty hydraulic rigs. Where greater installation/extraction capability is required, a casing oscillator (**Figure 81.5(e)**) or specially designed casing extractor may be used.

Segmental casings, installed as the bore proceeds, offer greater control over pile verticality and enable virtually

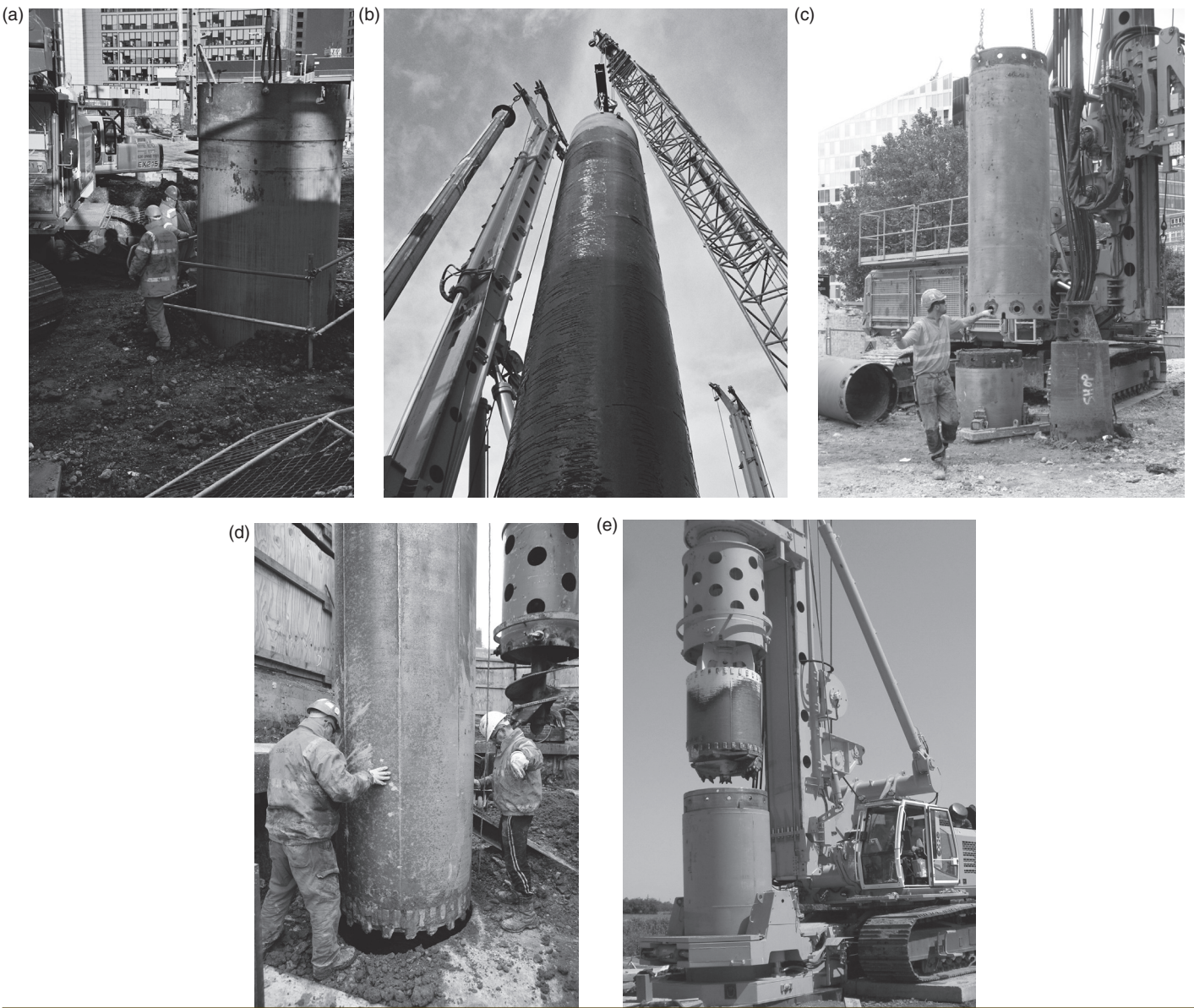


Figure 81.5 Rotary pile casings: (a) single wall temporary casing; (b) slip coated permanent lining; (c) segmental casing; (d) cutting head; (e) casing oscillator
Photos (a) to (d) courtesy of Cementation Skanska Ltd; (e) Courtesy of Bauer Equipment UK Ltd

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continuous support of the bore through unstable deposits. With the right combination of power and cutting head, they can also be used in conjunction with heavy duty coring tools to overcome substantial obstructions (**Figure 81.6**), including, in some cases, old piles.

Drill casings can be left in place as permanent linings. These linings may be used to prevent washout of concrete fines by flowing groundwater or to support the concreted shaft in very soft soils. They may also be slip-coated to reduce negative skin friction or to reduce the load shed by the pile in the vicinity of sensitive structures such as tunnels and basement walls (**Figure 81.5(b)**). Lightweight permanent linings may alternatively be installed and cast within the pile bore, allowing recovery of the more expensive drill casings for reuse.

81.2.1.4 Stabilising fluids

Stabilising fluids are used to support pile bores where they pass through unstable soils at depths beyond the economical use of temporary casing. A typical pile construction sequence using stabilising fluid is shown in **Figure 81.7**. Stabilising fluids take the form of a bentonite clay suspension or a polymer fluid. The composition, properties and operation of bentonite and the various polymer fluids are distinct from one another and call for understanding and experience on the part of the practitioner. A few practical observations are made here:

- Drilling with fluid support is only feasible from a working level at least 1.5–2.0 m above the effective water table, since support of the bore wall is provided by maintaining a positive pressure differential between the fluid and the surrounding groundwater.
- Polymer fluids generally hold less detritus in suspension, for less time, than bentonite, making base cleaning more critical with polymer fluids.
- Polymer fluids suit smaller sites since they require less space for the mixing and treatment plant.
- Waste bentonite must be disposed of by tanker to licensed tips. Polymer fluids may be broken down to water and polymer, reducing the difficulty and cost of disposal.

More detailed accounts of the use of bentonite and polymer stabilising fluids may be found in Fleming *et al*. (2009).

(a)

(b)

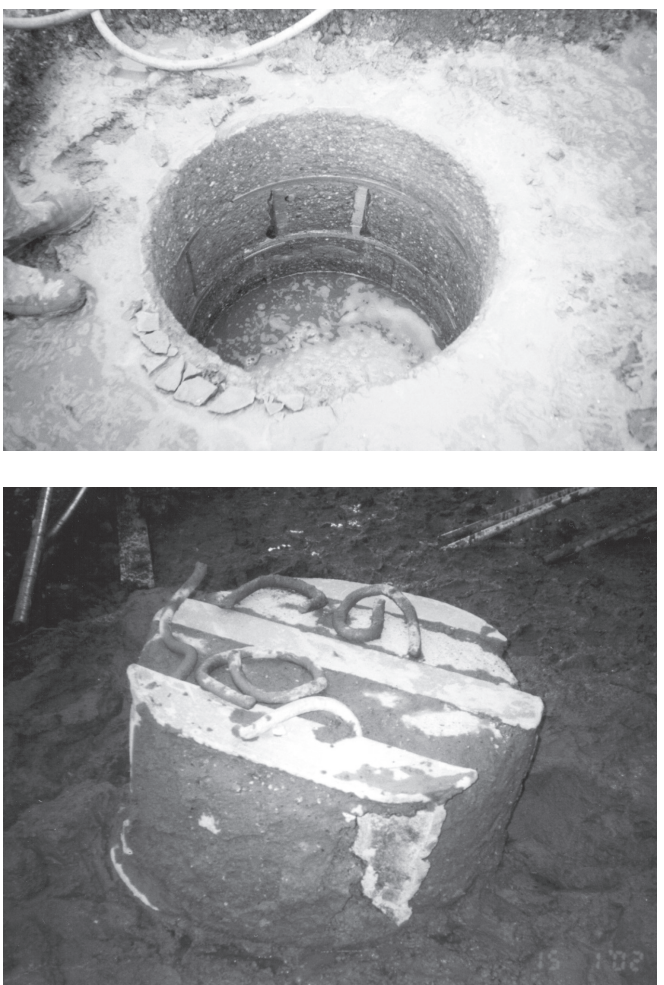


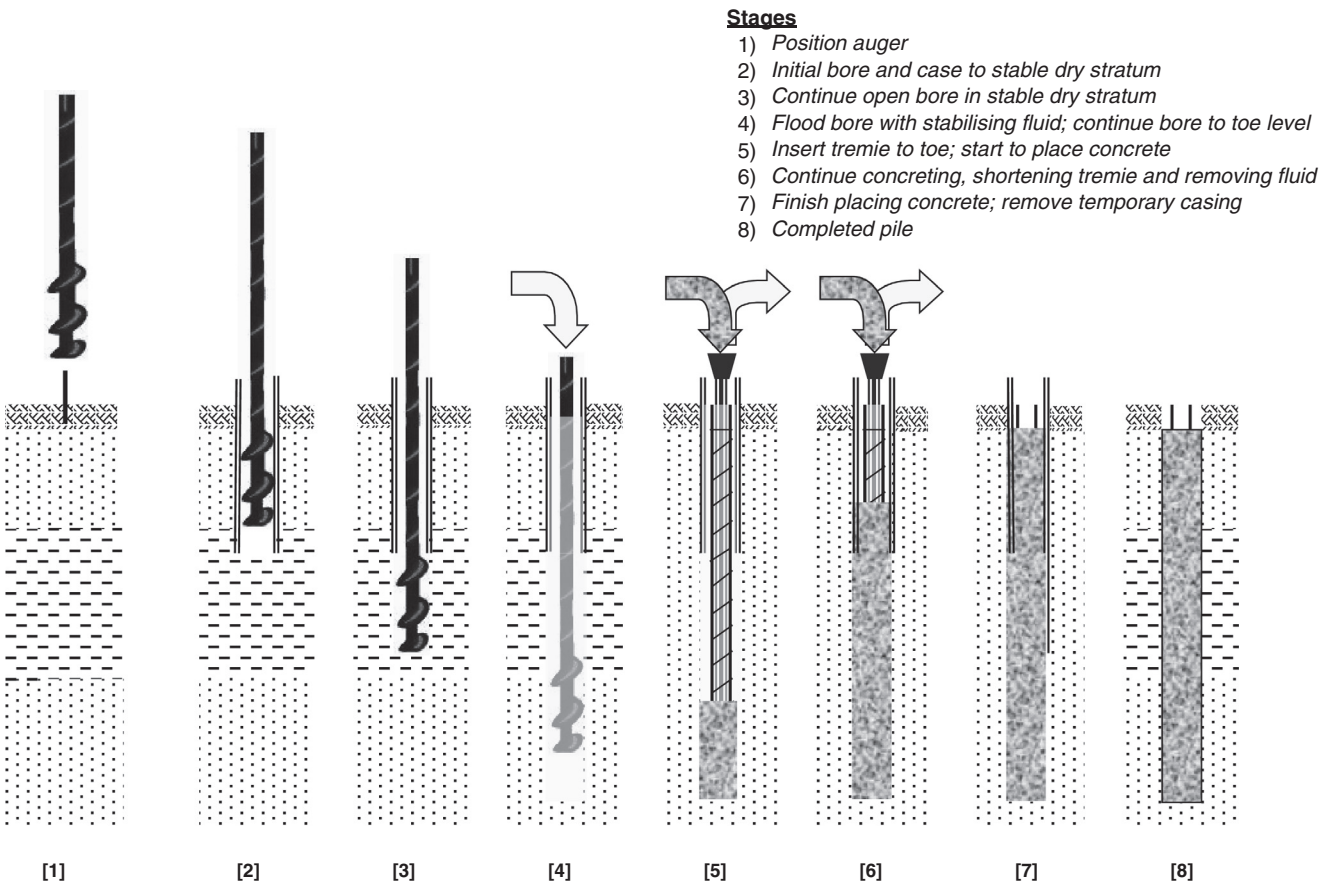
Figure 81.6 Rotary piling – obstruction removal: (a) core hole through concrete; (b) removal of reinforced concrete obstruction
Photos courtesy of Cementation Skanska Ltd

81.2.1.5 Placing concrete

In a dry bore, SPERW (Institution of Civil Engineers, 2007) requires that concrete is placed, in a single continuous operation, via a hopper and sufficient rigid delivery tube to avoid a fall of more than 10 m within a reinforcement cage. This limits segregation of the mix caused by impact on the reinforcement during the fall. In a wet bore (filled with water or stabilising fluid) concrete must be placed with a full-length watertight tremie tube (**Figure 81.7**). The initial charge of concrete is separated from the fluid by a disposable plug to avoid segregation or contamination of the mix, and delivered to the pile toe via the tremie tip. This charge displaces the fluid upward, covering the pile base and engulfing the tip of the tremie. Further concrete is delivered through the tremie, which is progressively shortened as the concrete level rises, until the required concrete level is reached. The displaced fluid is pumped away at the pile head. Throughout this operation the fluid and concrete levels must be maintained to counteract the effects of any groundwater pressures along the pile shaft. The tremie tip must remain embedded between 3 m and 6 m into the rising concrete at all times.

81.2.2 Percussion bored cast-in-place piling

Tripod-mounted cable percussive boring equipment continues to be available for piling in the UK, although it is increasingly replaced by the growing capabilities of hydraulic micro-piling rigs. The method involves much manual handling of heavy tools and casings, and achieves relatively slow production. This makes it less attractive than other methods on both health-and-safety and commercial grounds. The boring tools, including clay cutters, 'shells' for sand and gravel, and chisels, are suspended by rope from the rig and sequentially lifted and dropped in the



Notes:
i Temporary casing may be installed in a single length (as shown) or segmentally as the bore proceeds.
ii Reinforcement generally extends beyond casing to prevent cage uplift during casing extraction.
iii Concrete should be delivered by tremie in a manner to fully displace, and avoid mixing with, the stabilising fluid (ICE SPERW - B3.5.2.4).
iv Concrete should be cast to a level that ensures that a sound concrete connection can be made after trimming to the specified level (ICE SPERW Table B1.5).

Figure 81.7 Typical construction sequence for bored cast-in-place pile under stabilising fluid

bore to cut or pump soils up from the base of the bore (**Figure 81.8**). The tool is raised and spoil is cut away or emptied out on the working platform for disposal. Considerable experience is needed to ensure that this process does not significantly soften or loosen the surrounding soils. Temporary casings, which are hammered or surged into place, are handled in short threaded sections, screwed together and installed as the bore proceeds. The reinforcement and concrete processes are similar to rotary piling except that concrete is commonly delivered by dumper, via a chute at the pile head, or by pump.

Tripod piling rigs are light and easily transported, and have been used for installing piles where access and headroom are limited (within basements and in the vaults beneath existing buildings, benched into sloping sites, etc.). Diameters up to 600 mm are possible and depths in excess of 30 m have been successfully achieved.

Earlier percussive processes, such as the Benoto method, Puller (2003), have been used to install larger diameter piles. These methods have died out in the UK, in the face of more economical techniques. In recent years however, down-the-hole hammers, more commonly used for small diameter mining applications, have become available in sizes suited to bored piling. Single-bit hammers up to 750 mm diameter are available. Larger diameters can be drilled using cluster drills (multiple hammer units). These units can be used in combination with cased rotary piling to form deep rock sockets (Fleming *et al.*, 2009). The technique requires large quantities of compressed air and casing for borehole stability. Hydraulically driven hammers are also becoming available. These can reduce the need for temporary casing.

81.2.3 Continuous flight auger (CFA)

The CFA process (**Figure 81.9**) allows economical and rapid pile construction in water-bearing strata, without the need for casing or stabilising fluid. It is suitable for constructing piles through sands, gravels, silts and clays, and for limited

(a)



(b)



Figure 81.8 Percussion bored piles – tripod piling using: (a) 'shell' for boring in granular soils; (b) 'clay cutter' for boring in cohesive soils
Photos courtesy of Cementation Skanska Ltd

penetration of weak rocks. In comparison to rotary piling, greater power is required to drill in and extract the auger in a single pass. It is particularly important that the CFA piling equipment is sufficiently powerful to achieve the required pile depth without significant flighting of material from the bore (note: 'flighting' is the upward displacement of spoil along the auger flights to ground surface). The available depth and diameter range, and the ability to penetrate hard strata with CFA piling, are therefore more limited than with conventional rotary piling (**Table 81.1**). CFA piling is not appropriate in ground containing frequent and substantial obstructions, including old foundations and boulders, without due consideration in the foundation design.

After setting the rig up over the pile location [1] the hollow-stemmed, continuously flighted auger is drilled in a single stroke to the full pile depth [2]. The disturbed spoil remains on the flights at this stage [3]. Concrete, delivered by pump, is injected through the hollow stem of the auger to the base of the pile [4]. The auger, which is still rotating slowly, is lifted a short distance (< 100 mm) to allow entry of the concrete into the bore so that the initial charge engulfs the tip, and is then returned to the full bore depth. The spoil-laden auger, still rotating slowly, is then gradually retracted as further concrete is injected to form a continuous shaft [5]. Once the shaft is fully concreted to ground level the auger is withdrawn, the pile head is scraped clean and the reinforcement cage is inserted into the concrete [6] leaving the top of the steel at ground level [7].

81.2.3.1 CFA piling rigs

CFA piling became established in the UK piling market during the 1980s, coinciding with the development of purpose-built hydraulic piling rigs (**Figure 81.10**). Many hydraulic rigs are capable of modification to work in either rotary or CFA mode. The distinguishing features of CFA rigs are the single 'string' of continuously flighted auger, and the location of the drive table at the top of the auger.

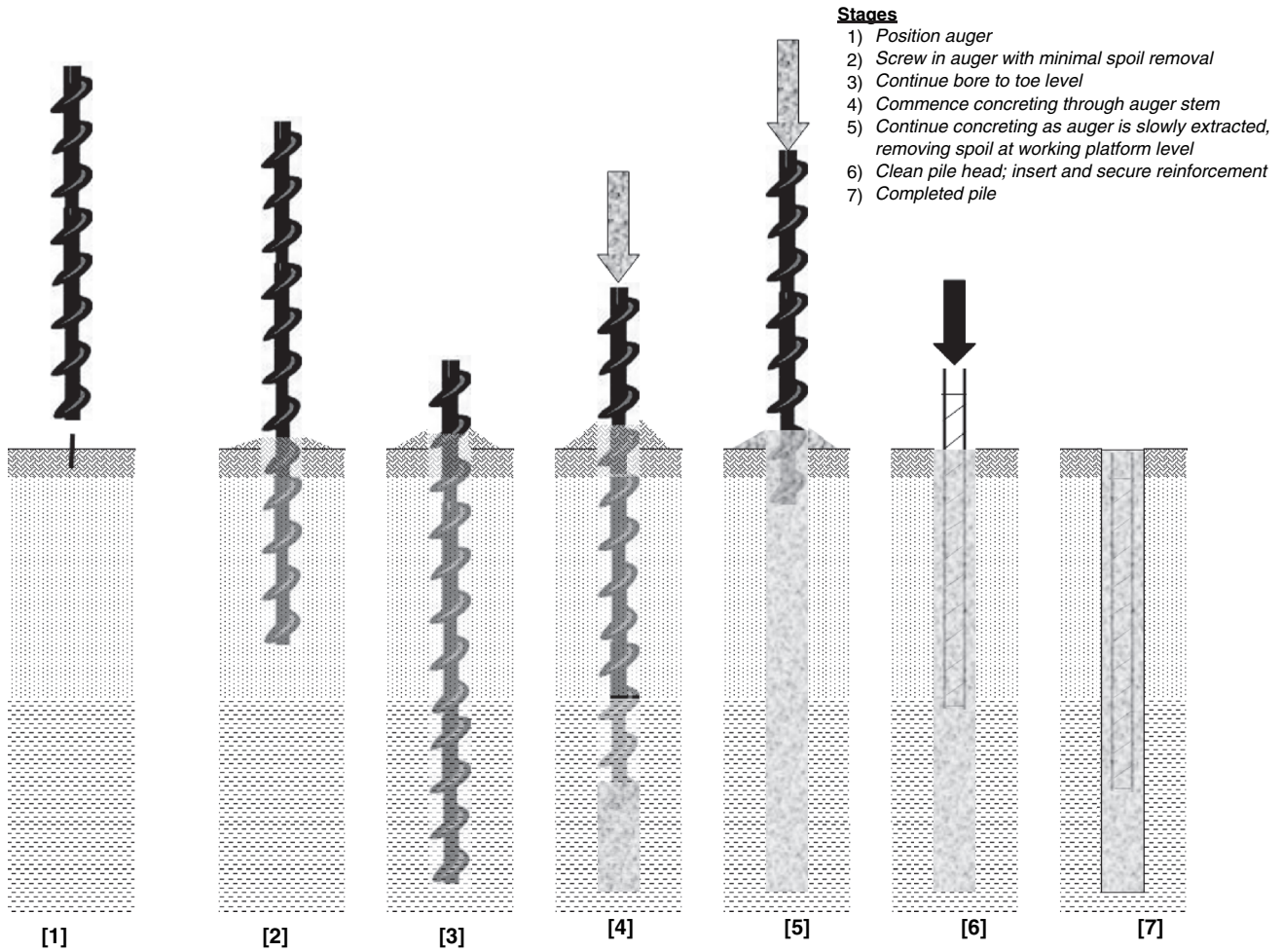
Many CFA piling rigs are fitted with a winch-driven or hydraulic 'crowd' facility designed to increase the downward thrust of the auger whilst drilling through harder strata. Extraction capability is enhanced by providing a 'foot' at the base of the mast which bears on the working platform to stabilise the rig during withdrawal of the auger. Mechanical auger cleaners are fitted to CFA rigs to prevent spoil from rising with the auger string during extraction. The rising spoil would otherwise present a hazard to operatives working below, and potentially destabilise the rig.

81.2.3.2 Excavation tools

As the name implies the excavation tools are restricted to augers. In common with rotary piling, the lead augers are mounted with a wide range of cutting tools designed and arranged to suit the expected ground conditions (**Figure 81.11**). The various configurations of flat teeth, picks, bullet teeth, etc. are suitable for boring through most soils and into weak rocks. CFA piling is not generally capable of penetrating stronger rocks or hard obstructions.

81.2.3.3 Placing concrete

CFA piling relies on a steady delivery of concrete to the pile. Concrete is therefore often delivered to site by readymix wagon and held in an agitator ready for delivery by pump to the piling rig. Once the auger has been drilled to depth the pump lines and hollow stem are pre-charged with concrete. The auger, still rotating slowly, is lifted a short distance (< 100 mm) to allow entry of the concrete into the bore (via the hollow auger stem and an end



Notes:
i Pile boring and concreting stages should generally be subject to automatic real-time monitoring (ICE SPERW B4.4.8).
ii Concrete should be cast to the commencing surface level to allow removal of contamination before installing reinforcement.
iii Reinforcement cage must be robust enough to resist the stresses imposed by insertion through fresh concrete (ICE SPERW B4.4.7).

Figure 81.9 Basic construction sequence for a continuous flight auger pile

or side entry valve at the auger tip). The auger is returned to the full bore depth, so that the initial charge engulfs the tip, and then steadily extracted with the spoil as more concrete is delivered to fill the resulting pile shaft. It is essential that a positive pressure of concrete is maintained at the auger tip throughout the concreting process and that the auger tip remains embedded within the rising column of concrete. Failure to control and coordinate the delivery of concrete and extraction of auger can lead to pile integrity problems (see Chapter 81 *Piling problems*).

81.2.3.4 Rig instrumentation

During the early development of the CFA technique in the UK a number of projects encountered serious pile integrity problems; others suffered ground loss and an unexpected variability in load-bearing performance. These problems arose in part because the CFA process is largely hidden from the eye and cannot be directly verified. Fleming (1995) discussed these issues and highlighted the need for close control of the boring and concreting phases of pile construction. It is now a standard requirement (Institution of Civil Engineers, 2007) that automated instrumentation is provided to monitor critical parameters throughout pile construction.

Data from built-in instrumentation is displayed in real-time on a computer, mounted in the operator's cab. The data include critical boring and concreting parameters including auger depth and rates of rotation and penetration, applied torque, concrete pressure and rate of delivery (**Figure 81.12**). This enables the operator to control flighting of material and over-rotation during boring, and to prevent over/undersupply of concrete during auger extraction. The data also provide a permanent construction record for each pile. These detailed records, coupled with integrity test results (Chapter 97 *Pile integrity testing*), provide



Figure 81.10 CFA piling rig
Photo courtesy of Cementation Skanska Ltd

essential tools with which to assure the quality of the CFA operation.

81.2.3.5 Placing reinforcement

It is necessary to cast the pile to commencing level to allow removal of contaminated concrete before insertion of the reinforcement cage. Cages must be robust enough to tolerate insertion into wet concrete to the required depth without distortion. Insertion depths up to 12 m are common. Greater depths are feasible (depths over 20 m have been reported) but commonly require additional measures which can include careful cage detailing, special mix designs and using a cage vibrator. Particular caution is required when piling in dry, fine-grained silts and sands. These soils can draw moisture from the concrete, causing an early set, and thus prevent full insertion of the cage.

81.2.3.6 Cased CFA piling

Cased CFA piling combines the advantages of CFA piling with the additional capabilities of a cased system. The process is



Figure 81.11 CFA piling: typical lead auger and cleaner configuration
Photo courtesy of Cementation Skanska Ltd

the same as the CFA method except that a single length of thick-wall temporary casing is inserted and removed simultaneously with the auger. The casing has a cutting head and is counter-rotated against the auger, creating a higher torque to enable penetration into harder strata. The stiff temporary casing helps to prevent flighting of superficial deposits when drilling in hard strata, and provides greater verticality control. The system requires a very high-powered rig with either a single drive head coupled with a torque-multiplier or two independent rotary drive heads (**Figure 81.13**). The former system has a typical depth limit of 16–18 m, whilst with the latter, respective cased and bored depths of 21 m and 27 m are claimed. When using the twin-drive system it may be feasible to cast the piles with a cut-off level below commencing level, in the same way as cased rotary piles.

The simultaneous extraction of auger and casing results in spoil being discharged from a point high above the working platform. The additional weight of spoil and equipment at height can have a destabilising effect on the rig. Particular attention is therefore required in the design of the working platform and in assessing the rig stability during operation. The safe discharge of the spoil to platform level must be controlled by purpose-built chutes (**Figure 81.13**).

81.2.4 Cast-in-place displacement auger piling

Cast-in-place displacement auger piles are installed using high-torque rotary equipment with drilling tools designed to form the bore by displacement rather than removal of the soil.

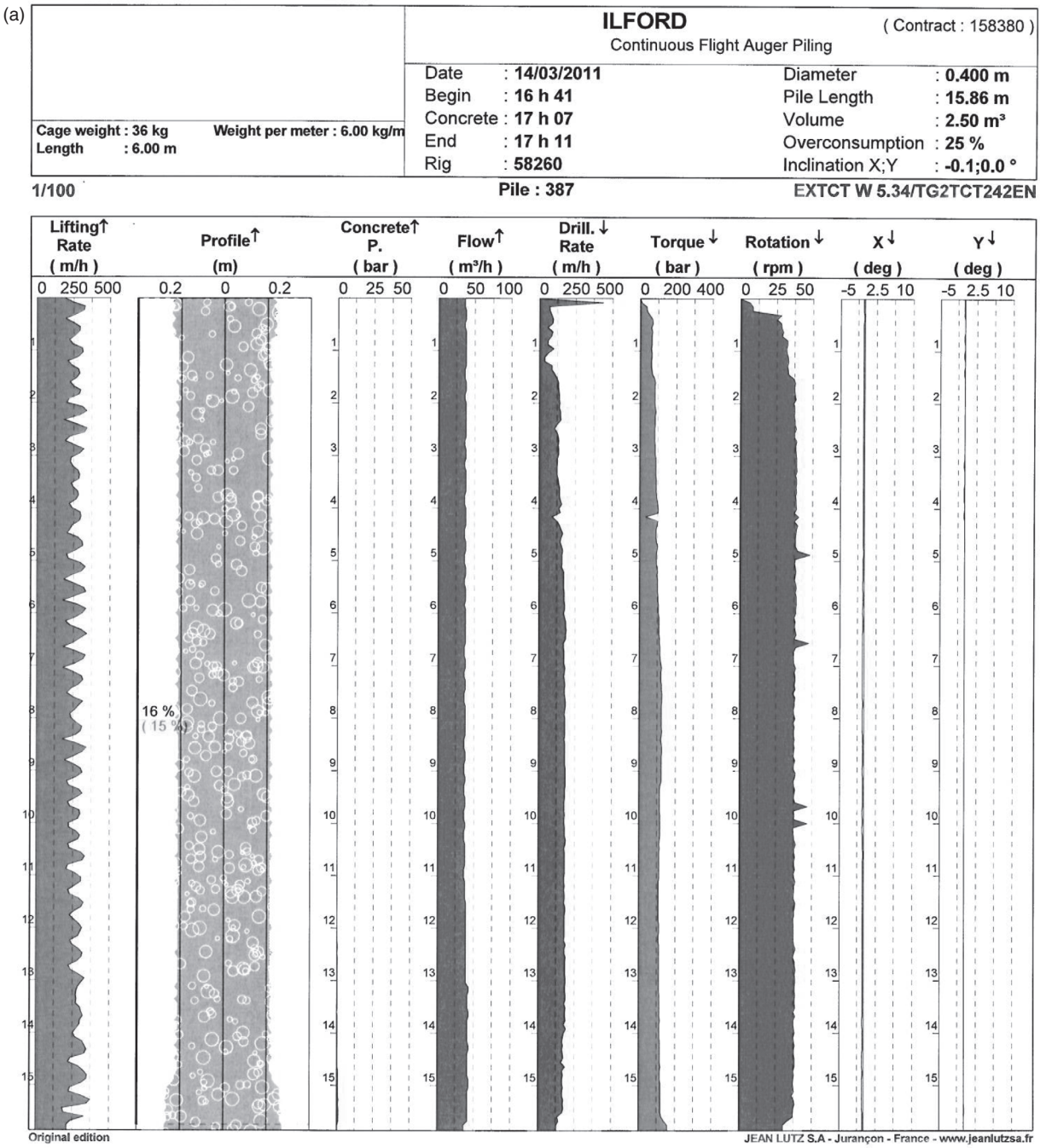


Figure 81.12 CFA piling rig instrumentation
Photo courtesy of Cementation Skanska Ltd

The system is relatively quiet and vibration-free, minimises the volume of spoil handling and concrete supply, and can improve the compaction of granular deposits. In the simplest form (e.g. the 'Omega' pile, Fundex system) a tapered or 'bullet'-headed auger is screwed into the ground, displacing the soil, and extracted as concrete is placed through the hollow stem, to leave a straight-shafted pile. A typical construction sequence is illustrated in **Figure 81.14**.

Alternative methods (e.g. Atlas pile, continuous helical displacement (CHD) pile and the 'Screwsol' system) employ a short, flighted, bullet-headed auger, screwed into the ground in one pass and carefully back-screwed along the same trajectory during concreting, to leave a screw-form pile (**Figure 81.15**). The construction of screw piles is subject to the requirements of the European Execution Code, BS EN12699:2001 (British Standard Institution, 2001).

(b)



Figure 81.12 (Continued)



Figure 81.13 Cased-CFA piling rig
Photo courtesy of Cementation Skanska Ltd

All of these methods require comprehensive instrumentation to confirm successful construction of the pile at the required diameter. It is vital to confirm that each pile has reached the design bearing stratum and, in the case of screw-form piles, that the flights are robust enough to prevent premature failure. As with CFA piling the reinforcement is installed after completion of the concrete column to ground level.

When considering the use of this technique, it should be borne in mind that, in common with driven displacement piling, cast-in-place displacement piling is likely to cause heave of the surrounding ground.

81.2.5 Barrettes

Barrettes may be used to carry exceptionally large vertical loads and high lateral loads. They are constructed using diaphragm walling methods and equipment (Chapter 85 *Embedded walls*). They are most commonly rectangular but may take a variety of forms (**Figure 81.16**).

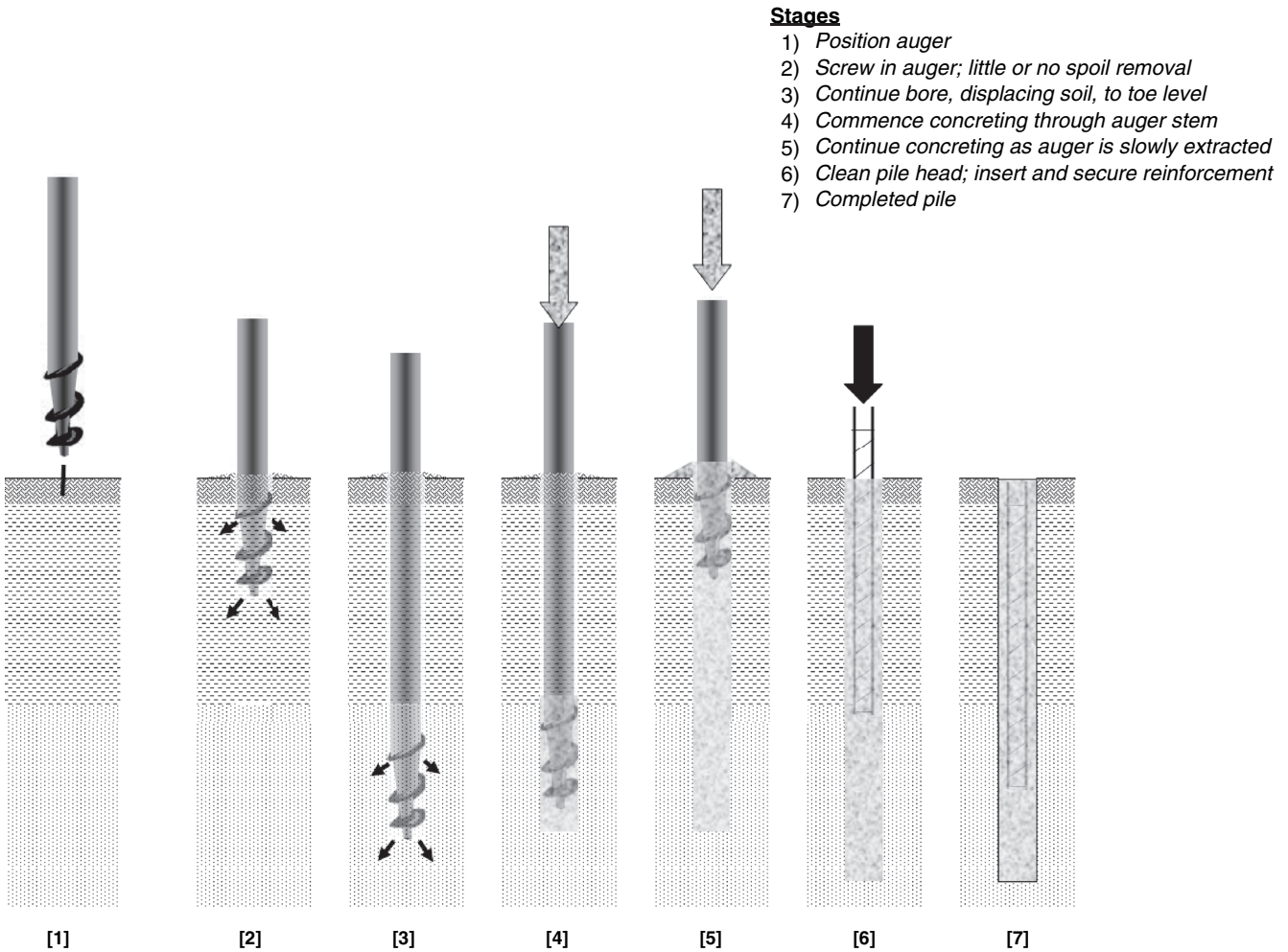
In soft ground, and some weak rocks, barrettes may be excavated using rope-suspended grabs. In stronger rocks, rock cutters or milling machines may be required, although deployment of such equipment is only occasionally economical compared to rotary piling.

81.2.6 Specialised techniques

81.2.6.1 Under-reaming

Under-reaming is an extension of the rotary piling technique which can significantly enhance the capacity of relatively slender and short piles. It is only practicable in stable and dry strata, typically stiff clays. After forming a straight-shafted bore to the required level a specialised under-reaming tool (**Figure 81.17**) is lowered in the closed position to the toe of the bore. As the tool is rotated the reaming arms are slowly opened to form the under-ream. Cuttings are removed by a collecting bucket at the bottom of the tool.

Under-reaming tools are designed to form a maximum under-ream diameter of approximately three times the pile shaft diameter, with a roof angle commonly limited to about 35° to vertical (Institution of Civil Engineers, 2007). Given the dependence of such piles on end-bearing it is essential that the drilling process is precisely controlled to ensure that no remoulded material or crumbs of soil remain on the base of the ream. This is usually proven by the use of CCTV, and testing or sampling apparatus mounted on the tool (for safety reasons physical descent and inspection is no longer an accepted process).



Notes: i Pile boring and concreting stages should generally be subject to automatic real-time monitoring (ICE SPERW B4.4.8).
ii Concrete should be cast to the commencing surface level to allow removal of contamination before installing reinforcement.
iii Reinforcement cage must be robust enough to resist the stresses imposed by insertion through fresh concrete (ICE SPERW B4.4.7).

Figure 81.14 Displacement auger piling – typical construction sequence.

81.2.6.2 Base grouting

Base grouting is a secondary process applied to rotary piles. Its purpose is to mobilise very high base capacities in dense sands without excessive pile settlement. The effect is two-fold:

- consolidation of sand loosened by the pile bore;
- pre-compression of the shaft, reversing the direction of applied shaft friction.

This gives the pile a very stiff response under load, a necessary requirement for particularly tall structures, and can result in economies in the pile design (see Chapter 54 *Single piles*). Base grouting is generally only economical in large diameter piles and requires the pile to have sufficient shaft capacity to resist the uplift caused by grouting.

It is achieved by casting grout circuits into the pile. Each circuit comprises injection and exit tubes linked by a 'tube-à-manchette' (Chapter 86 *Soil reinforcement construction*) across the pile toe. The number of circuits depends on the pile diameter (**Figure 81.18**). After casting, and before the pile concrete gains significant strength, each circuit is pressurised with water to 'crack' the grout valves and the base concrete, in preparation for grouting. At a later stage cementitious grout is injected under pressure at the pile base in a controlled process that upwardly compresses the pile shaft. The effect of the grouting is confirmed by monitoring the pile head for uplift (a minimum of 1–2 mm is typically specified) or by instrumentation installed close to the toe of deeper piles, where the pile head may not respond.

81.2.6.3 Embedded columns

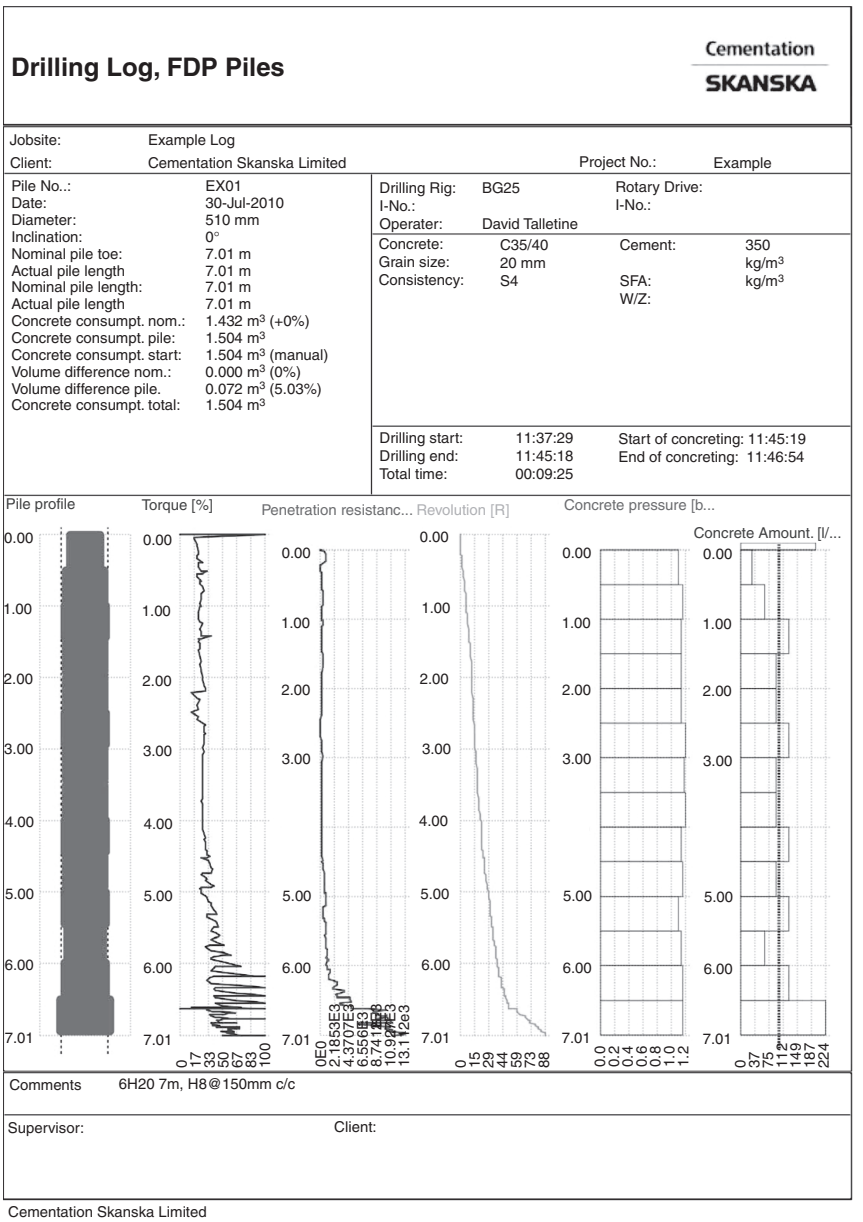
Embedded (plunge) column technology has been developed to enable accurate installation of permanent columns within

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(a)



(b)



Drilling Log, FDP Piles

Cementation SKANSKA

Jobsite:	Example Log		
Client:	Cementation Skanska Limited	Project No.:	Example
Pile No..:	EX01	Drilling Rig:	BG25
Date:	30-Jul-2010	I-No.:	
Diameter:	510 mm	Operater:	David Talletine
Inclination:	0°	Rotary Drive:	
Nominal pile toe:	7.01 m	I-No.:	
Actual pile length	7.01 m		
Nominal pile length:	7.01 m		
Actual pile length	7.01 m		
Concrete consumpt. nom.:	1.432 m³ (+0%)		
Concrete consumpt. pile:	1.504 m³		
Concrete consumpt. start:	1.504 m³ (manual)		
Volume difference nom.:	0.000 m³ (0%)		
Volume difference pile.	0.072 m³ (5.03%)		
Concrete consumpt. total:	1.504 m³		
Concrete:	C35/40	Cement:	350 kg/m³
Grain size:	20 mm		
Consistency:	S4	SFA:	kg/m³
		W/Z:	
Drilling start:	11:37:29	Start of concreting:	11:45:19
Drilling end:	11:45:18	End of concreting:	11:46:54
Total time:	00:09:25		

Pile profile | Torque [%] | Penetration resistanc... | Revolution [R] | Concrete pressure [b... | Concrete Amount. [l/...

Comments: 6H20 7m, H8@150mm c/c

Supervisor: Client:

Cementation Skanska Limited

Figure 81.15 Displacement auger piling: (a) typical displacement auger; (b) typical displacement auger pile log
Photos courtesy of Cementation Skanska Ltd

basements constructed using top-down techniques. The columns are embedded in large diameter rotary cast-in-place piles using specially designed placing equipment. These may be simple mechanically adjustable guides, or more sophisticated hydraulically operated frames that can install columns to tolerances consistent with BS EN1993 (British Standards Institution, 2005a) in dry or wet bores. **Figure 81.19** illustrates the typical embedded column installation sequence. The columns are generally rolled steel column sections, although other forms such as pre-cast concrete may be accommodated. After installation and removal of temporary casings the open bore around the column must be temporarily backfilled to stabilise the column and the surrounding ground and to prevent accidents.

81.2.6.4 Geothermal piles

Piled foundations are increasingly being used to exploit geothermal energy exchange between the supported structure and the ground. Coupled with heat exchanger technology, geothermal piles can be used to heat and cool buildings in an environmentally friendly manner. Flexible plastic pipework is incorporated in the reinforcement cages (**Figure 81.20**) of rotary cast-in-place piles or plunged into CFA piles. These pipes are connected via a manifold in the sub-structure to a heat pump,

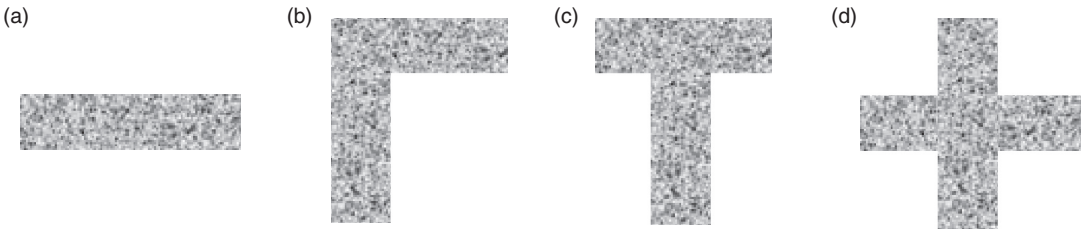


Figure 81.16 Typical forms of barrette



Figure 81.17 Under-reaming tool
Photo courtesy of Cementation Skanska Ltd

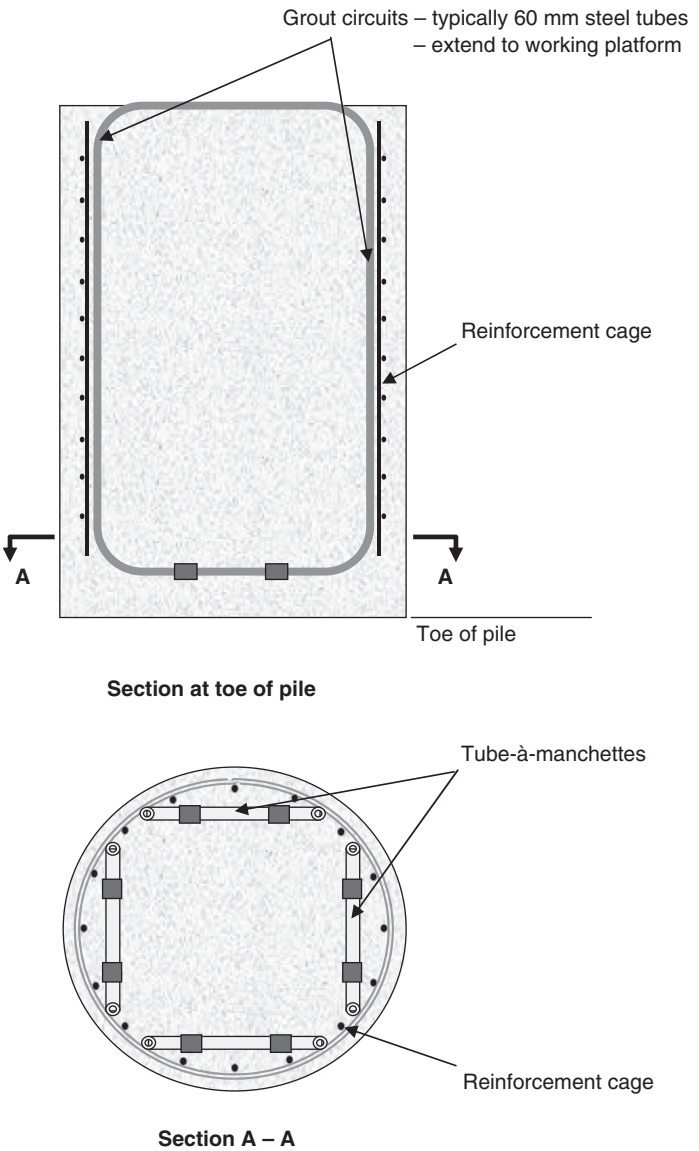


Figure 81.18 Typical base grouting circuit configuration

which allows a reversible exchange of energy between the structure and the ground (Bourne Webb *et al.*, 2009).

81.3 Driven piles

81.3.1 Introduction

Driven piles have historically been described by the means of installation or construction, driven into the ground using a piling hammer. A better term would be displacement piles, describing the manner in which the pile interacts with the soil during driving. The soil is displaced, predominantly in a lateral direction, rather than replaced as with a bored pile. A driven

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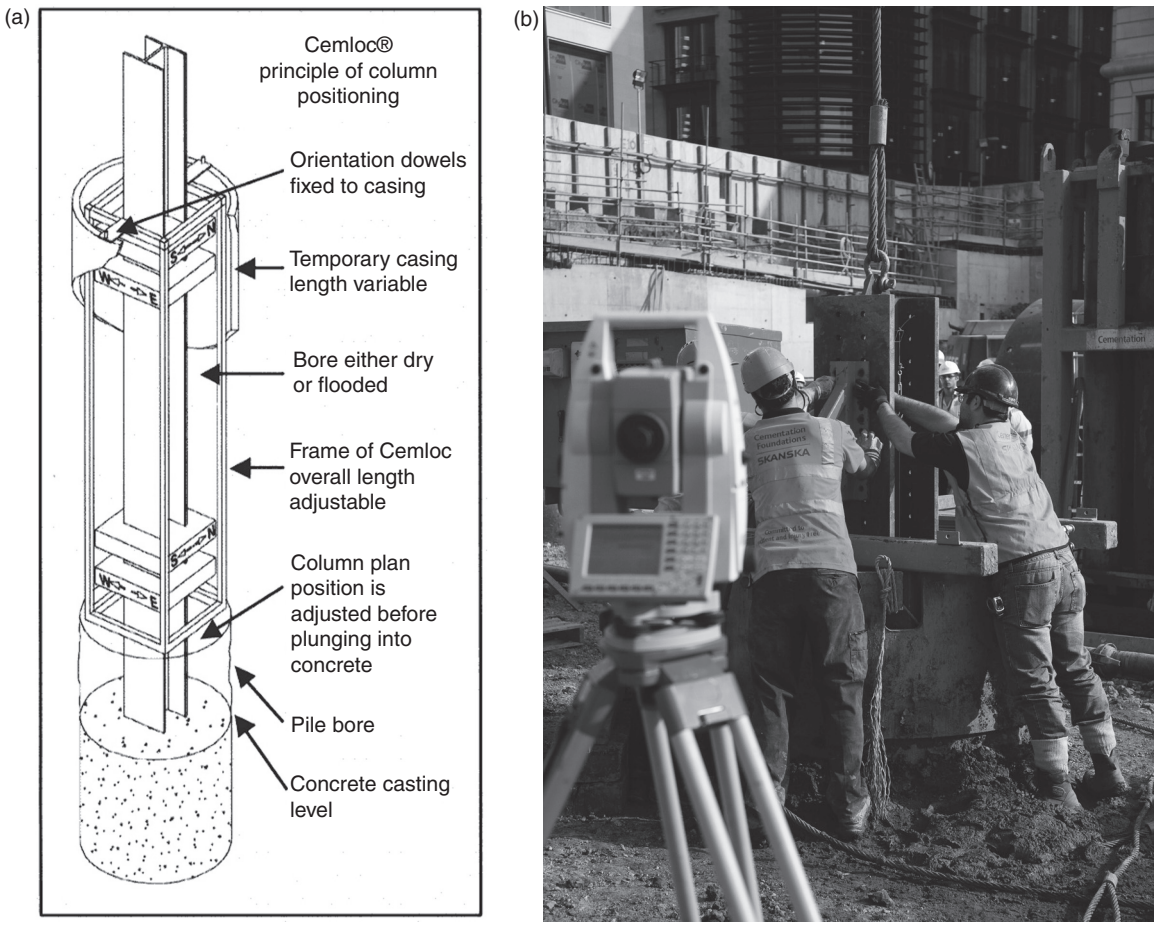


Figure 81.19 Plunge column piles: (a) plunge column installation; (b) precise installation of plunge column
Photos courtesy of Cementation Skanska Ltd

pile may be pre-formed or cast *in place*, and may be a large or small displacement type, as explained below.

In the UK driven piles are generally installed or constructed in accordance with the (Institution of Civil Engineers, 2007) SPERW and the process is subject to the requirements of the execution code for displacement piles BS EN12699:2001 (British Standards Institution, 2001; under systematic review by CEN TC288 at the time of publication).

81.3.2 Types of displacement pile used in the UK

(a) The concept of driving a pile for construction purposes is credited to a Neolithic tribe called the 'Swiss Lake Dwellers' who lived in what is now Switzerland about 6000 years ago. They used timber piles, not for supporting heavy loads as we do today, but for elevation to protect against attack from wild animals. The Romans built the first bridge across London's River Thames in the year AD 60 using timber piles. The modern age of wood-preserving began in England in 1832 with the concept of pressure injecting chemicals into wood.

In more recent times timber piles have been used almost exclusively in sea defence works (groynes), installed as the vertical posts supporting timber planks which act together to reduce longshore drift on beaches. Considerations of durability and drivability require that they are invariably produced from hardwood species, with section sizes of between 225 and 350 mm square.

It is necessary to protect the lower end of the timber pile with a pointed cast iron shoe, as shown in **Figure 81.21**, to aid penetration through the granular beach deposits without undue damage or deviation from plan position or vertical alignment. In order to prevent 'brooming' or splitting of the pile head a steel band is fitted (see **Figure 81.22**).

(b) Segmental pre-cast concrete piles were introduced in the 1970s into the UK from Scandinavia. These large displacement piles are now universally square in section and proprietary section sizes range from 150 mm to 400 mm square, with working loads usually in the range 200 kN to 1400 kN, although higher loads are achievable in favourable soil conditions.

Individual segment lengths normally range from 3 to 15 m, although some manufacturers are able to supply up to 17 m in single lengths. Where soil conditions require it the pile can be extended almost indefinitely (the longest to date in the UK is in excess of 70 m) using pile joints. BS EN12794 (British

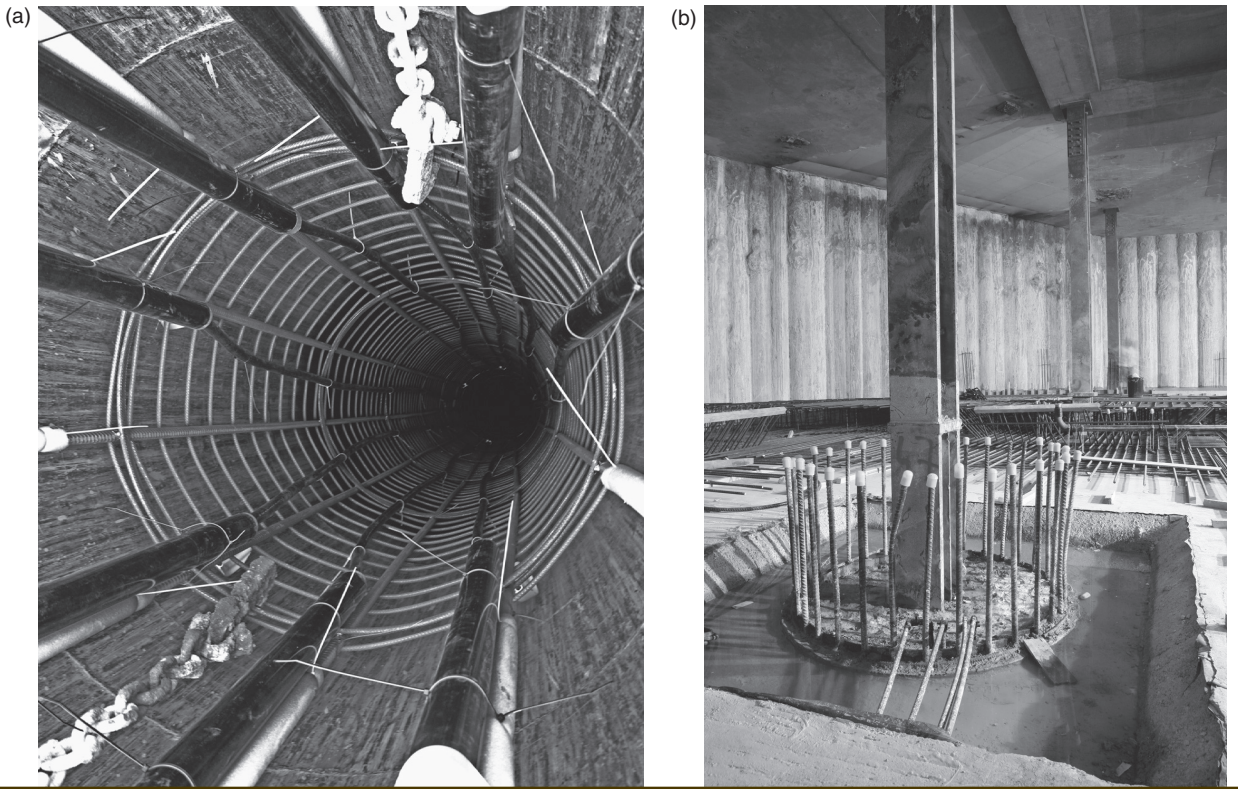


Figure 81.20 Geothermal piles: (a) typical pipe installation; (b) completed geothermal pile with plunge column
Photo courtesy of Cementation Skanska Ltd



Figure 81.21 Timber piles with cast iron shoes
Photo courtesy of Aarsleff Piling



Figure 81.22 Timber pile head protection
Photo courtesy of Aarsleff Piling

Standards Institution, 2007) is the regulatory standard for both pile segments and pile joints, which are classified according to structural capability and robustness.

Piles are cast in moulds and removed to a storage area for curing the day after casting (see **Figure 81.23**). Once the required characteristic concrete strength has been reached, usually 8 to 10 days after casting, the piles can be delivered to site for installation.

All pile segments should be labelled to provide traceability (see **Figure 81.24**).

(c) Whilst segmental pre-cast concrete piles are generally reinforced with straight high-tensile main bars and a square helical binding, pre-stressed concrete piles manufactured using strand reinforcement have been used on sites where the

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Figure 81.23 Pre-cast concrete piles being removed from casting moulds
Photo courtesy of Aarsleff Piling



Figure 81.24 'CE' marked pre-cast concrete piles being installed on a power station project in Kent
Photo courtesy of Aarsleff Piling

quantity of piles justified setting up a casting facility on site. This type of pile permits longer segment lengths and is more popular in countries like the Netherlands where transporting long piles by road is easier than in the UK.

Where the piles are installed to a predetermined length in easy driving conditions, normal reinforcement can be incorporated into the head of the pile to provide bond steel into the pile cap (see **Figure 81.25**). Largely overtaken by the segmental pre-cast concrete pile, pre-stressed piles are now rarely encountered in the UK.

(d) Driven cast-in-place (DCIP) concrete piles are, as the term suggests, a large displacement pile formed in the ground using a temporary steel casing. The lower end of the casing is closed off during driving with an expendable steel shoe, and the casing is then driven to the design toe level, embedment or required resistance, generally with a hydraulic drop hammer striking the head of the casing.

Prior to placing high slump concrete, a steel reinforcing cage bound with circular helical wire is introduced into the casing. The casing is then withdrawn using either a vibrator or rapid blows of the piling hammer to assist with extraction and concrete compaction. The sequence of construction is illustrated in **Figure 81.26**.



Figure 81.25 Pre-stressed concrete piles with starter bars cast into the pile head

At all times it is necessary to maintain sufficient head of concrete in the casing to minimise the risk of 'necking' of the shaft, and piles must be cast above the water table and cut-off level, generally to piling platform level. These precautions are designed to prevent loss of pile section and/or washing out of fines from the concrete.

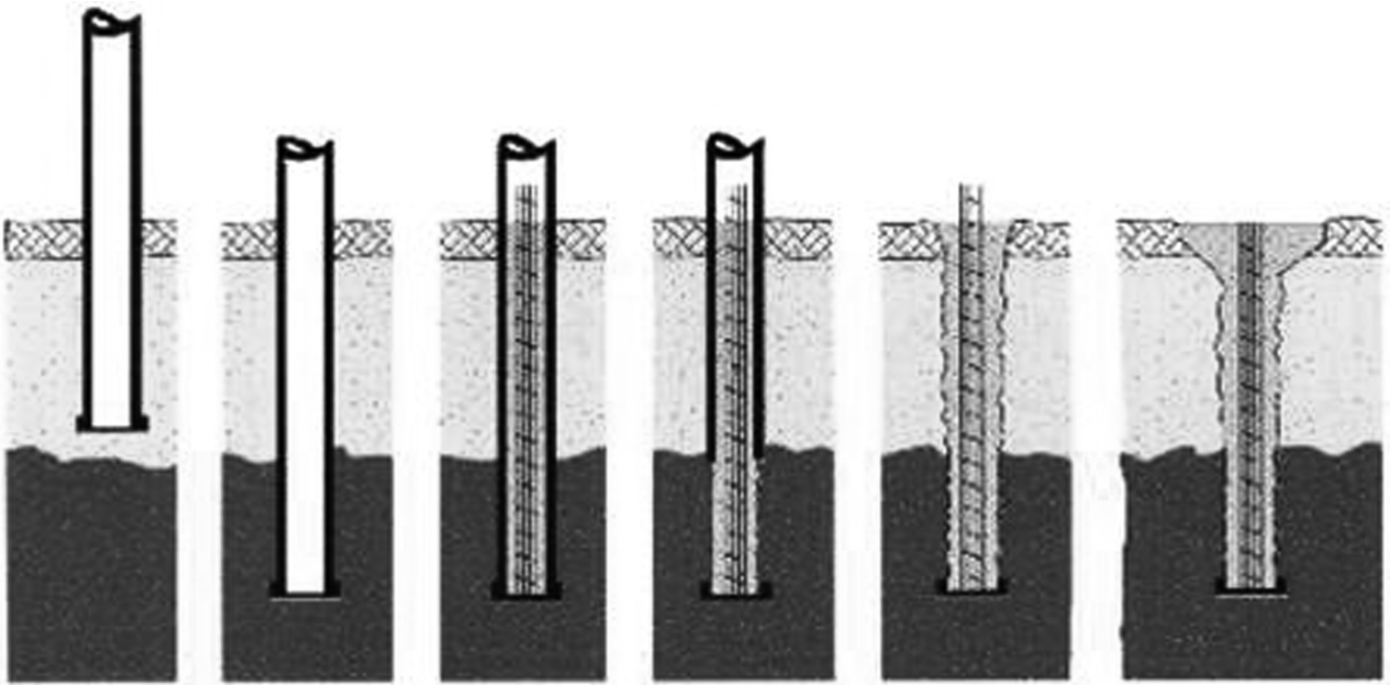


Figure 81.26 DCIP construction sequence (flared head shown at the right side of the schematic)
Photo courtesy of Cementation Skanska Ltd

DCIP piles are readily adaptable to changes in founding level and can incorporate flared heads as an extension of the shaft when used under concrete floor slabs or embankments.

Proprietary sizes range from 340 mm to 600 mm in diameter with safe working loads between 600 kN and 1500 kN depending on the ground conditions.

(e) Steel H piles take their name from the cross-section of the profile used. These may be universal column sections or universal bearing piles where the web and flanges are of the same dimensions and thickness. H piles are small displacement piles and are well suited to hard driving through ground where less robust piles might be damaged by artificial or natural obstructions.

They are able to resist higher levels of combined axial and lateral loads than the equivalent sizes of concrete pile and can be installed at a rake to enhance this capability further. This high combined load-bearing capacity, together with their low soil displacement characteristics, mean that they have been used frequently on highways projects, an example of which can be seen in **Figure 81.27**.

H piles can be delivered to site in single lengths of up to 30 metres, but this would require the use of very large rigs. Alternatively piles can be extended *in situ* by butt-welding, although this is time consuming and expensive.

Section sizes vary from 203 mm × 203 mm × 45 kg/m to 356 mm × 368 mm × 174 kg/m, capable of supporting maximum axial loads ranging from 600 kN to 2300 kN in compression when manufactured from high yield strength steel.



Figure 81.27 Steel H piles being installed to support a major road crossing a railway line
Photo courtesy of Aarsleff Piling

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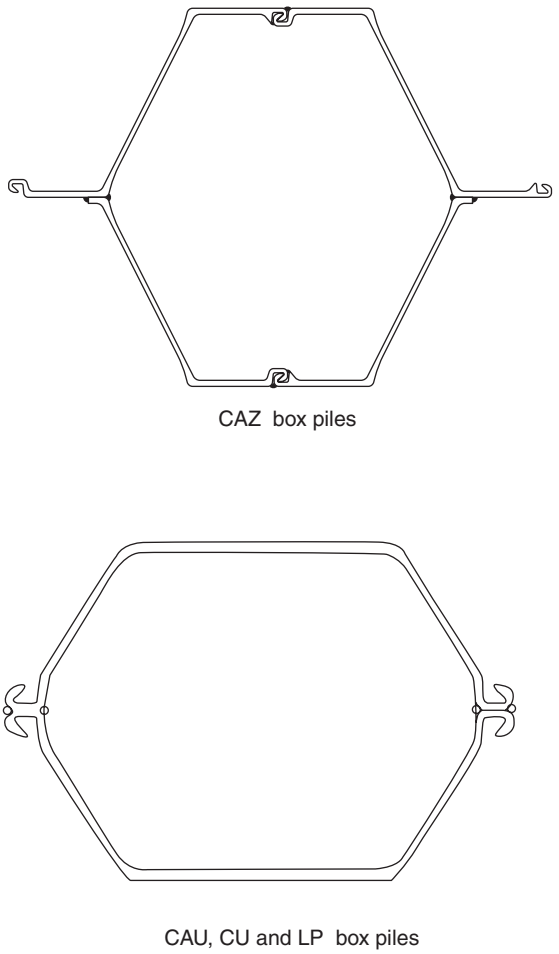


Figure 81.28 CAZ and CU steel box piles
Reproduced with permission from Arcelor Mittal; all rights reserved

(f) Steel box piles are formed by welding two U-section sheet piles or two pairs of Z-section sheet piles back to back, forming a symmetrical box profile (**Figure 81.28**). The clutches of the piles can be used to introduce the box piles into a line of sheet piling where high vertical and horizontal loads can be resisted without adversely affecting the appearance of the wall.

They can also be used as individual bearing piles in open jetty or marine dolphin construction as they have geometrical properties that permit long lengths with little or no lateral support.

They are generally driven open-ended and are, therefore, small displacement piles unless they plug with soil during driving (see the following section on large diameter steel tube piles for an explanation of plugging). They can be supplied in similar lengths to H piles and extended in the same way.

(g) Large diameter steel tube piles are commonly used in marine piling such as jetties, piers and mooring dolphins and utilise their high buckling strength as columns and energy-absorbing capacity in bending to resist loads where the piles project some distance above the sea-bed (**Figure 81.29**). Tubes



Figure 81.29 Pairs of 23 m long 610 mm diameter raking steel tube piles installed for a quay wall in Glasgow
Photo courtesy of Aarsleff Piling

with wall thicknesses greater than 10 mm are suitable for driving open-ended and so can be considered small displacement piles unless they plug with soil during driving.

Plugging is likely to occur once the cumulative shaft friction from the trapped soil on the inside of the tube wall exceeds the gross end-bearing resistance of the tube when considered as closed.

Large diameter steel tubes can also be driven with closed ends. These piles range in size from 273 mm to 2020 mm in diameter.

(h) Small diameter steel casings are an alternative to pre-cast concrete piles in situations where artificial or natural obstructions could deflect or damage the concrete pile. Commonly known as Aberdeen casings, they are a recycled product from the offshore oil industry, originally manufactured for use as drill casing.

Available as non-prime or second-hand material, they are manufactured to an American Petroleum Institute (API) specification with steel having yield strength of the order 550 to 600 N/mm^2. However, being uncertified material they are usually treated as being at least equivalent to grade S355JH. Independent batch testing of larger quantities can be arranged if deemed appropriate to particular project requirements.

Driven open-ended (and so qualifying as small displacement piles) their robust nature and drivability means that safe working loads of 400 kN to 1200 kN are achievable using the same installation equipment as is required for pre-cast concrete piling (see **Figure 81.30**).

Diameters range from 178 to 339 mm and stock lengths are in the range 11–14 m. The piles can be extended by driving the plain end of a second tube into the collar at the head of the first length driven (see **Figure 81.31** for detail of collar).



Figure 81.30 339 mm diameter Aberdeen casings to support a wind turbine foundation near Inverness
Photo courtesy of Aarsleff Piling



Figure 81.31 244 mm diameter Aberdeen casing collar
Photo courtesy of Aarsleff Piling

81.3.3 Types of displacement pile used outside the UK

(a) Within mainland Europe driven piling has generally been restricted to Scandinavia, the Netherlands and Belgium, together with Germany and Spain.

As stated in section 81.3.2(b) segmental pre-cast concrete piles originated in Scandinavia and are still widely used in those countries. They are often used as the upper part of a composite pile, with a timber shaft below the water table where the risk of timber decay is minimal. Segmental pre-cast concrete piles are also common in northern Germany and are now being regularly used in Poland and some former Eastern bloc countries.

In addition to installing (generally) pre-stressed concrete piles, contractors in the Netherlands and Belgium employ many variations on the DCIP piling technique, known locally as vibro-piling. In this context a Combi-pile, as the name suggests, involves the introduction of a pre-formed element as the shaft of the vibro-pile. In areas of deep compressible soil the annulus can be filled with bentonite to reduce downdrag, or otherwise filled with grout to improve positive shaft friction.

The term 'Combi-pile' is also used to describe tubular steel piles, usually driven open-ended to form the primary elements of an embedded steel pile wall. Clutches welded on either side of the tubular piles allow one or more sheet pile sections to form the secondary elements between the tubular piles.

In Belgium and Germany the Frankipile is still used regularly, having become practically obsolete in the UK. DCIP piles, including the Frankipile, are occasionally used in France and Italy but to a far lesser extent than bored or CFA piles.

In Spain driven pre-cast concrete piles, both segmental and pre-stressed, will be encountered on projects in the southern regions of the country.

In Germany and Switzerland small diameter ductile cast iron driven piles, installed with high-frequency hydraulic hammers, have established themselves as a low-vibration displacement piling system.

In a wider geographical context, small diameter steel 'pipe piles' are common in North America, along with proprietary systems such as the 'Tapertube' pile. Pre-cast concrete piles are also used but section sizes are larger than those generally encountered in the UK. In many areas of the Far East where deposits of soft compressible materials are encountered, pre-stressed spun concrete 'pipe piles' are commonplace.

81.3.4 Practically extinct types of driven pile (UK only)

Towards the end of the twentieth century the UK piling industry witnessed the decline and/or demise of two versatile driven piles, which lost their significant market share to faster and/or less labour-intensive methods. Improvements in integrity testing of piles and rig instrumentation, which boosted confidence in DCIP and continuous flight auger piles also contributed to the demise of the first of these to be discussed here.

(a) Developed and patented by Russian marine engineer Alexander Rottinoff, the West's shell pile was constructed using pre-cast, fibre-reinforced concrete tubes, 900 mm long, threaded on to a steel mandrel and driven into the ground with a solid concrete shoe placed at the bottom of the mandrel. The shells were held together with circular steel bands to prevent ingress of groundwater or silt and driven into the ground using a rope-operated drop hammer rig as shown in **Figure 81.32**.

Once the specified depth or resistance had been achieved the mandrel was withdrawn and concrete placed in the core, with a reinforcing cage to suit the diameter and loading conditions. Diameters ranged from 380 mm to 600 mm.

The same company also produced the West's segmental pile, an early form of solid pre-cast concrete pile 280 mm in diameter with 900 mm long segments being joined together during driving with a simple spigot and socket joint. On completion

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Figure 81.32 Westpile 30RB based shell piling rig

of driving, a full-length single reinforcing bar was grouted into a central longitudinal hole. This system was only suitable for light loads in soft soil conditions and became obsolete on the introduction of West's Hardrive pre-cast concrete pile.

(b) The Frankipile, which is still used world wide, was first developed by the Belgian engineer Edgar Frankignoul, who registered an international patent for the system in 1902. It is a cast-in-place concrete pile with an enlarged base and a cylindrical shaft which can penetrate stiff soils and readily reach depths of 20 m.

The pile is constructed using a temporary steel casing and an internal drop hammer, the sequence being as illustrated in **Figure 81.33**. A plug of gravel is placed in the bottom of this tube and compacted by the hammer, which then enables the casing to penetrate the ground by virtue of the friction between the inner wall and the gravel plug.

On reaching the design depth, the casing is lifted slightly and the gravel plug broken out. Dry concrete is then introduced and expelled by the hammer to form an expanded base, thereby significantly improving the original bearing capacity of the founding stratum.

After inserting a reinforcing cage the pile shaft was traditionally formed in the same way using a semi-dry concrete, thus

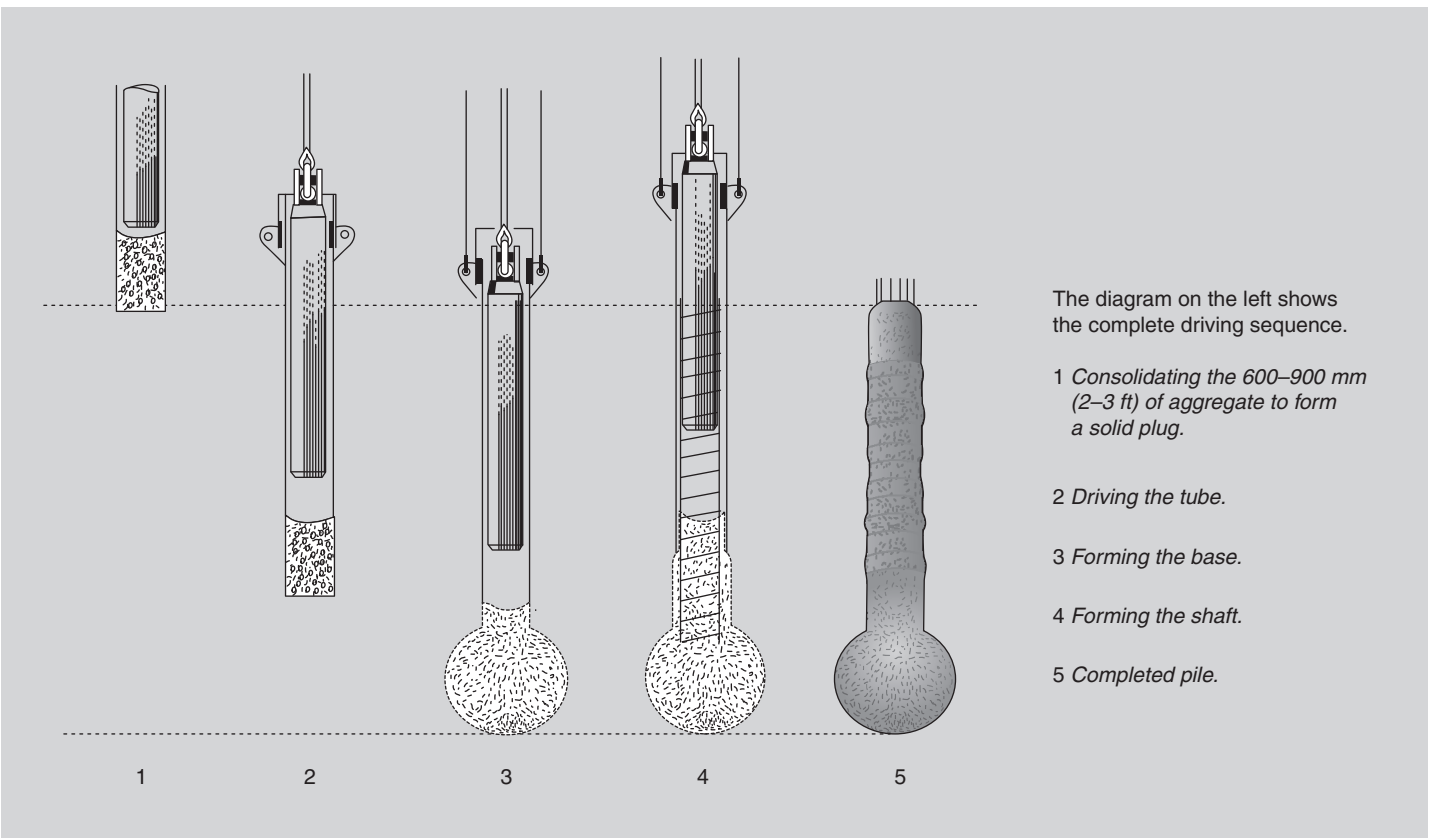


Figure 81.33 Frankipile construction sequence
Reproduced from FRANKI® pile, courtesy of Cementation Skanska Ltd

producing a pile with excellent end-bearing resistance and improved shaft resistance due to the ribbed profile caused by successive lifting of the tube and hammering out of the concrete.

Such a construction process is slow and labour-intensive, with production rates being only 20–25% of alternative driven cast-in-place or driven pre-cast concrete piles. The technique was modified in the mid 1970s by using high slump wet concrete to form the pile shaft, and where particular circumstances such as aggressive ground conditions require a protective coating, a pre-cast concrete pile can be used to form the shaft.

The application of the Franki system in the UK has decreased dramatically due to cost considerations, although this system is still an option when site conditions are suitable.

81.3.5 Driven piling plant

Driven piling frames provide a guide for the alignment of the pile and hammer during driving and usually have at least two winches. Early piling frames comprised open platforms on which were mounted the winches (often steam powered with a large boiler) and a vertical lattice or tubular mast that supported the piling leader used to guide the pile and hammer, as shown in **Figure 81.34**.

On land these frames usually moved on railway lines or by using a system of rollers; some Frankipile machines were able to 'walk' using an ingenious system of hydraulic jacks. These large frames provide the rigidity and stability needed to control the alignment of very large piles, and more recent versions can still be found working in marine applications. However, conditions on most onshore projects favour the use of smaller equipment.

Initially the move away from large fixed piling frames involved the use of hanging leaders suspended from the jib of a crawler crane. This type of rig was successfully used by West's Piling and Construction (later Westpile) for many years to install shell piles using a rope-operated drop hammer (see **Figure 81.32**) and the same type of equipment, modified by the addition of an adjustable foot boom and foot pad, was marketed by BSP International Foundations for the installation of steel piles using their own diesel hammers. At about the same time piling contractors such as Expanded Piling, Keller (then GKN) and Cementation adapted the BSP style of equipment to suit the rigours of DCIP piling, predominantly by strengthening the leaders. Typical examples of these largely obsolete piling rigs can be seen in **Figure 81.35**.

The arrival of pre-cast concrete piles in the 1970s from Scandinavia brought with it the compact fixed leader rig mounted on an Akermann hydraulic excavator, manufactured by Banut in Sweden. Although hanging leader rigs were used for some considerable time to install pre-cast concrete piles in the UK, the Banut rig allowed the development of a hydraulic piling hammer where the drop weight is lifted using a hydraulic ram instead of a rope and winch, the result of which is a better-controlled driving operation. Banut and Junttan (from Finland) have developed the fixed leader rigs to a level where they are suitable for all forms of driven piling, with only a few examples of hanging leader machines remaining in service. Junttan have further developed a hydraulic hammer with an accelerated fall which increases the efficiency rating of the hammer blow by up to 35%. A modern Junttan PM20 rig is illustrated in **Figure 81.36**. A Banut 700 rig is featured in **Figure 81.30** whilst a smaller Banut 500 rig can be seen in **Figure 81.37**.

Box piles and steel tube piles are generally installed using a combination of hydraulic/electric vibrator and/or hydraulic drop hammer suspended from a crawler crane, although fixed leader rigs can be used given satisfactory access and working space. Thin-walled steel tubes can be bottom-driven using an internal hammer falling on a plug of semi-dry concrete in the same manner as a Frankipile.

81.3.6 Suitable and unsuitable ground conditions

Driven concrete piles can be used in a wide range of ground conditions. Often considered to be only suited to end-bearing



Figure 81.34 Rail-mounted Frankipile rig *c.*1930



Figure 81.35 Historical photograph of crane-based driven cast in place drop hammer rigs

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Figure 81.36 Junttan PM20 piling rig with 5 tonne accelerated hydraulic hammer
Photo courtesy of Aarsleff Piling



Figure 81.37 Banut 500 rig with shrouded 3 tonne hydraulic hammer unloading piles on a housing site
Photo courtesy of Aarsleff Piling

conditions in granular soils and weak rock, they can be used in cohesive soils provided that the limitations of driving stresses (for pre-cast concrete piles) and casing extraction forces (for DCIP piles) are considered. DCIP piles are not normally suitable for pile depths much in excess of 20 m because of the effort required to extract the casing. Boulders and massive obstructions will preclude the use of driven piles unless they can be economically removed prior to piling. Steel tube and H piles are particularly suited to hard driving situations or where small obstructions are anticipated. In the latter case it would be prudent to use Aberdeen casings as these are more readily available than ex-mill materials if additional piles are required. Durability in aggressive ground conditions will influence the choice of pile type and material. Steel tube piles can incorporate a sacrificial element of wall thickness, and both pre-cast concrete and steel piles can be given a protective coating to improve resistance to corrosion.

81.3.7 Environmental considerations

A common reaction to a proposal to use driven piling is that it will create undesirable levels of noise and vibration.

However, most construction processes create detectable levels of vibration and noise, and tolerance is greatly increased when the origin, likely levels and duration of noise or vibration are made known in advance and those affected are reassured that consideration has been given to these factors.

It may not be possible, for technical reasons, to replace driven piling by a 'quieter' piling technique. Even if it is feasible, the operation of the alternative method may prolong the piling duration, so that the overall disturbance to the community may not actually be reduced. Piling hammers are available that are completely shrouded (see **Figure 81.37**), and most hydraulic driven piling rigs can be adapted to prebore pile positions prior to driving to reduce the level of vibration (see **Figure 81.38**).

Factors affecting the acceptability of site noise include site location and layout, pre-existing ambient noise levels and the daily hours of work. In the case of both noise and vibration, measured levels will reduce with increasing distance from the source. Annex F of BS 5228-1 (British Standards Institution, 2009) gives guidance on predicting the expected levels of noise from construction sites and Annex E of BS 5228-2 (British Standards Institution, 2009) contains empirical expressions derived by Hiller and Crabb (TRL Report 429: 2000) for the prediction of vibration levels which take into account driving energy, soil conditions and distance between the source and the point of measurement.

Further considerations which play a part in the overall environmental impact of a piling scheme are traffic movements in and out of the site, the efficient use of natural materials, and the embodied energy/CO_2 emissions for the complete process.

A positive environmental consideration is that driven piling produces no spoil, making it particularly suitable for sites with contaminated Made Ground that would otherwise have to be disposed of safely and at great cost.

Extensive research at the University of Sheffield, the results of which have been endorsed by the Environment Agency, has concluded that solid square or circular piles pose no additional threat to an underlying aquifer when they are driven through contaminated ground that is separated from the aquifer by a clay layer.



Figure 81.38 Junttan PM20 rig fitted with CFA auger string and rotary head for pre-boring
Photo courtesy of Aarsleff Piling

81.3.8 Drivability

Given a set of foundation loads and a ground investigation report it is not that difficult to produce a piling layout with numbers, loads, section sizes, diameters and lengths. However, that piled foundation must be buildable, and in the case of driven piles, the lengths of piles calculated must be achievable using readily available equipment and without damaging the piles or driving equipment in the process.

This drivability assessment can be made based on experience, and a responsible piling specialist will always be prepared to comment on the viability of a scheme at an early stage of the design.

In addition to experience, computer software is available that can model the soil, pile and driving system to predict resistance to driving and stresses in the pile during installation.

81.3.9 Installation control

A driven pile provides valuable feedback on the strength of the ground through which it is being installed. This resistance to penetration will usually be recorded on the first piles driven (and thereafter at regular intervals) as the number of blows per unit of penetration (usually 250 mm in UK) for a given hammer weight and drop.

'Sets' will be recorded at varying intervals and at the end of drive, three consecutive sets will usually be recorded. The precise definition of set is the permanent downward displacement of the pile through the soil per hammer blow; however, measurements of set are generally recorded as the total movement for 10 successive blows. Set calculations provide a useful guide to the resistance that should be encountered for a given load-bearing capacity but should not be used in isolation. The relevant ground investigation information must also be reviewed to verify that the level at which the set is being achieved is reasonable.

Additional measurements of temporary compression (sometimes referred to as 'quake') of the pile and soil under individual hammer blows will also provide an indication of whether the pile toe is in competent material or is, for instance, sitting on a thin hard band underlain by weaker soil. Driven piles can be designed and installed to an embedded length in cohesive soils, in which case the set achieved is not necessarily indicative of bearing capacity, although comparing penetration records will highlight any potential variation in soil strengths and pile performance.

In weak rocks the depth of embedment will be as relevant as the set achieved, and in those materials prone to relaxation (such as Coal Measure Mudstones) it will generally be necessary to overdrive the pile to achieve a satisfactory outcome. Driven pre-formed piles have the advantage of permitting restrike sets to be measured at some time after installation to determine whether the ground is relaxing or conversely setting up and improving. These restrike measurements can be combined with dynamic load testing where appropriate.

81.3.10 Ground-related aspects of pile installation

As stated in section 81.3.1, driven piles displace soil as they are installed. Although the displacement is predominantly in a lateral direction, this can result in ground heaving upwards in closely spaced groups of piles. In the case of short piles end-bearing on rock, this can be enough to unseat the piles, but should be detectable by routine restrikes on pre-formed piles, and any risen piles can then be reseated by redriving. Ground heave can also affect the accuracy of setting out in large pile groups by moving marker pins. In the case of longer DCIP piles heave forces can compromise the integrity of freshly placed concrete if piles are installed consecutively too close together. In confined working areas such as inside coffer-dams or basements, ground displacement can have adverse effects on adjacent temporary or permanent structures.

Closely spaced DCIP piles should not be installed consecutively even where ground heave is not an issue. The driving of the casing can jeopardise the integrity of an adjacent pile shaft in which the concrete is still workable or not set. The minimum permissible spacing between adjacent piles formed in the same day will be a function of casing diameter and the soil strength. The sequencing of operations should, therefore, take account

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of this risk, and plant movements should also be planned to avoid tracking over constructed piles.

Silty soils that are prone to dilation can give rise to 'false sets' where apparently high dynamic resistance does not result in the anticipated static resistance under a load test.

Driven piles installed through soft clays and silts can generate excess pore water pressures during driving, which will dissipate with time. As a result the ground will eventually reconsolidate (the same can happen if ground heave occurs) and induce an additional load on the shaft of the pile which should be accounted for in the pile design.

When pre-cast concrete piles are installed firstly through soft or loose materials and then require prolonged driving through more competent soil the pile will experience tensile forces caused by the stress wave travelling from the head of the pile to the toe and back to the head. In extreme cases these forces can be large enough to cause yielding of the main reinforcement in the pile. The computer software referred to in section 81.3.8 can be used to predict the tensile forces in the pile, and the hammer weight, fall height and main reinforcement in the pile can be adjusted to suit the anticipated conditions. Dynamic test measurements taken during installation can subsequently be used to verify the actual driving stresses in the pile.

Driven piles installed in cohesive soils will usually exhibit a degree of 'set-up' after a period of time has elapsed after installation, this effect being caused by the clay remoulding around the shaft as the excess pore water pressures created in the soil during driving dissipate. The same effect will also be found to occur in some grades of chalk. With pre-formed piles this effect can be demonstrated by the tightening of the set at restrike compared to that measured at the end of drive. In the case of DCIP piles restrike sets are not possible and in both cases load testing the pile too soon after installation will result in the pile underperforming compared to its long-term capability. However, as the ground may take several weeks to recover to a state where the pile performance can be optimised, considerations of programme rarely allow for this benefit to be maximised.

81.4 Micro-piles

81.4.1 Definition

Bored micro-piles are defined as bored piles with a shaft diameter no greater than 300 mm, and driven micro-piles are defined as driven thin-walled steel piles with a shaft diameter no greater than 150 mm.

81.4.2 History and introduction

Bored micro-piles were originally developed by Fernando Lizzi and Fondedile during the 1950s in Italy. Lizzi employed the Pali Radice ('root pile') micro-pile system for strengthening existing foundations which were settling or required upgrading (see **Figure 81.39**) (Lizzi, 1982). Bored micro-piles have become increasingly popular throughout Europe, North America and the rest of the world over the past five decades and they are now used in many different applications. Driven micro-piles were developed during the 1980s and they are primarily used to provide a low-cost foundation solution for the domestic building market. There are other types of micro-pile such as jacked, jet grouted and post-grouted micro-piles; however, they are less common and they are not covered here. The installation of micro-piles is labour-intensive, expensive and time consuming and they should only be used when the more conventional piling techniques have been discounted. However, when used correctly and in the right circumstances, they can bring real value to a project. There are a multitude of micro-pile types and this chapter can only provide a brief introduction to them. Specialist micro-pile advice should be sought when the preliminary micro-pile scheme has been decided.

81.4.3 Where to use bored micro-piles

Bored micro-piles are extremely versatile and they can be used in a wide variety of compression and tension loading applications:

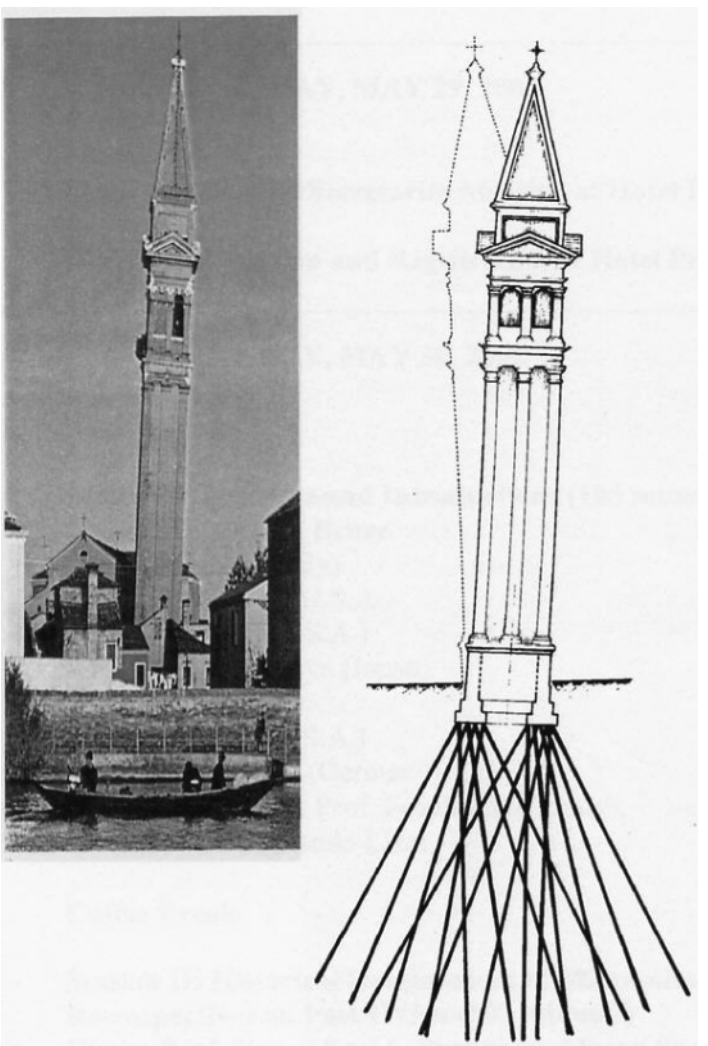


Figure 81.39 Burano Tower, Venice
Reproduced from Lizzi (1982)

- **Foundation upgrading and strengthening** – Pali Radice ('root pile') micro-piles are formed through existing foundations and develop an intimate bond with the structure; they found in competent ground at depth, thus providing the foundation with 'roots', hence their name.
- **Restricted access and working room** – There are a wide range of micro-pile drill rigs, from small 1 tonne rigs which can drive through doorways and work within 2 m of headroom (see **Figure 81.40**), through to larger 15 tonne rigs with 9 m+ high masts (see **Figure 81.41**). The capabilities, production rates and costs of these micro-pile rigs vary considerably.
- **Difficult ground and hard rock** – Bored micro-piles can be installed using an extensive range of drilling techniques to penetrate almost any ground conditions and/or obstructions, except possibly steel and strong timber, where high-speed coring techniques may need to be employed.



Figure 81.40 1.5 tonne drill rig (auger)



Figure 81.41 15 tonne drill rig (DTHH)

- **Permanent casings** – Bored micro-piles can install permanent steel casings through very soft ground or voids (e.g. caves or solution features) as part of the construction process.
- **Seismic loadings** – An increasingly important international use of bored micro-piles is their ability to be retrofitted to existing structures to upgrade their foundations to withstand seismic loadings.
- **Basement restraint** – Bored antiflotation tension micro-piles (ATMs) provide an efficient system of restraining large deep basements against hydrostatic uplift forces where there is reasonably competent ground at shallow depth. In addition, they can help reduce bending moments in basement slabs and thus reduce their thickness.

81.4.4 Where to use driven micro-piles

Driven micro-piles are less versatile and are generally used on new and existing (underpinning) housing projects where there is poor ground over firmer strata. Pile loadings are in the range of 100 kN to 300 kN. Driven micro-piles can be installed in restricted-access and low-headroom working locations using small drill rigs or even a hand-held grundomat hammer (see **Figure 81.42**).



Figure 81.42 Grundomat driven micro-pile

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81.4.5 Types of micro-pile and drilling systems

There are a large range of micro-pile types and drilling systems to accommodate the wide range of working environments and ground conditions around the UK and the rest of the world:

- **Bored micro-piles** – There are four main types of bored micro-pile drilling systems (see **Figure 81.43**). These range in diameter from 100 mm to 300 mm with SWLs between 100 kN and 1500 kN (normally 300 kN to 700 kN). High-capacity micro-piles often require a full-depth large diameter central reinforcement bar to ensure adequate structural capacity. This central rebar is usually fully threaded (e.g. Dywidag GEWI or Macalloy Mac 500) to enable it to be connected together in sections using full-strength couplers for low headroom or deep pile applications. In addition, a steel head plate may also be required to facilitate an efficient force transference mechanism into the pile cap. If a steel head plate is used then this should be located in the middle of the pile cap and secured above and below with full-strength nuts. Bored micro-piles generate their geotechnical restraint primarily from shaft friction because the shaft area is considerably larger than the end-bearing area. They can provide high tension capacities as well as high compression capacities when they are founded deep into strong bedrock and incorporate a full-depth large diameter central rebar (e.g. 50 mm to 75 mm diameter). Micro-piles are bored using 0.5 m to 1.0 m lengths of drill rods/augers, depending upon the pile diameter and the headroom available, and can extend up to 50 m to 70 m deep. It is unusual for micro-piles to be deeper than 30 m and particular allowance should be made in the calculations for the elastic compression behaviour of the pile shaft if they are. Bored micro-piles can be installed vertically or at any rake up to 45°.
- **Rotary case and auger bored micro-piles** – These are the simplest and most popular form of rotary bored micro-piles, which are constructed by sealing temporary steel casing into competent strata and then dry open-hole auger drilling thereafter. The depth of the temporary casing varies according to the ground conditions. These micro-piles are popular in London where open-hole auger drilling in London Clay is possible. If ground water and/or obstructions are encountered then case and auger drilling may have to be stopped. The most popular temporary casing sizes range from 140 mm to 343 mm, with respective open-hole auger sizes from 102 mm to 305 mm. Whilst the largest 343 mm/305 mm diameter micro-pile is larger than the 300 mm diameter micro-pile limit, it is still often referred to as a micro-pile because it is installed using a micro-pile drill rig. Temporary casing depths should be limited to ~15 m wherever possible; however, this depth is dictated by the type and power of the drill rig and the ground conditions encountered.

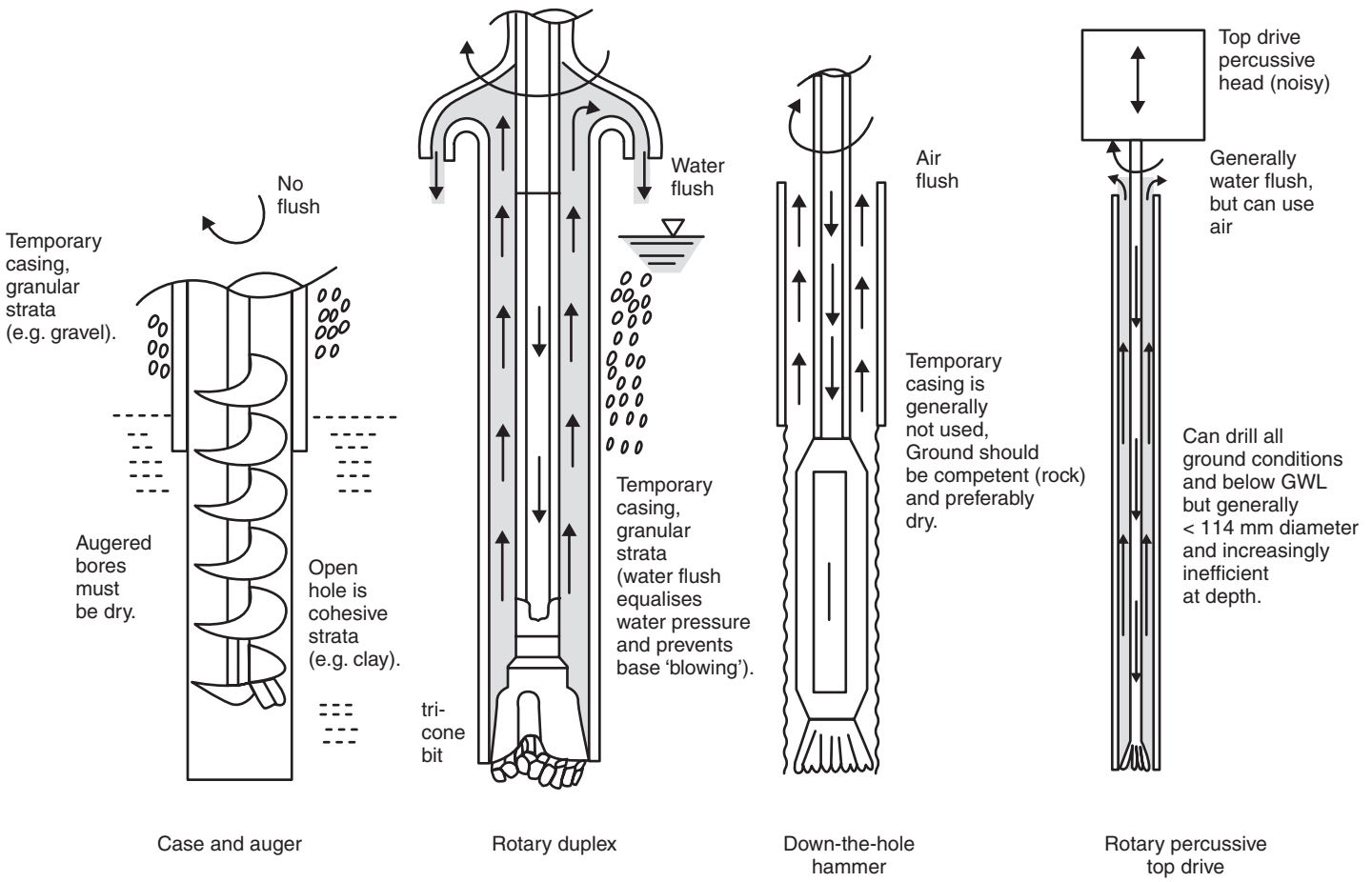


Figure 81.43 Bored micro-pile drilling systems

- **Rotary duplex bored micro-piles** – The rotary duplex drilling technique is useful in poor and/or granular strata with a high groundwater level, where there is a risk of bore hole or basal instability. The temporary casing and drill rod with tri-cone drill bit ('rock roller') are simultaneously rotary drilled into the ground using water flush. The drill water passes down the centre of the drill rods and up the inside of the temporary casing, before exiting through the duplex head. The hydrostatic forces therefore remain positive on the inside of the temporary steel casing, which will prevent 'blowing' conditions in the base. The most popular size is 220 mm diameter temporary casing with a 187 mm diameter tri-cone bit, although a variety of sizes down to 140 mm are available. A wide range of ground conditions can be drilled with this system, including weak rocks, which can be open hole drilled with a tri-cone bit below the temporary casing. However, penetration rates will become progressively slower in stronger rocks, leading to an increased risk of a 'smooth' bore, which can lead to lower grout-to-ground bond values. Large volumes of drilling water are required (~3 litres per second for a 220 mm diameter micro-pile) to ensure that there is sufficient uphole velocity to transport the drill cuttings out of the bore. An efficient drill water recirculation system is necessary, which may comprise sand bag bunds and recirculation/settling tanks. This type of micro-pile is becoming less common, due to the environmental implications of using and disposing of large volumes of drilling water. It is also possible to use air flush; however, this is not commonly undertaken, because of dust, lack of lubrication, bit wear and overheating problems.
- **Rotary down-the-hole hammer (DTHH) bored micro-piles** – This type of percussively drilled micro-pile has become increasingly popular in recent years, primarily due to the technological advancements in drilling equipment. Air-flushed DTHHs drill quickly through masonry, concrete and strong bedrock and typical sizes range from 100 mm to 300 mm diameter. There is also a water-flushed DTHH; however, this is expensive and not commonly used. Conventional DTHH micro-piles require temporary steel casing rotating into the ground until sealed into bedrock. The DTHH can then open-hole drill an efficient and 'rough' bore into the bedrock which can generate high bond values. DTHHs can drill strong bedrock quickly because the percussive hammer remains immediately behind the drill bit and it does not lose any energy as the bore gets deeper. However, there are risks with this type of micro-pile, primarily the failure to obtain an adequate casing seal into bedrock, which can lead to large volumes of air being pumped into the ground and causing problems, and borehole collapses behind the DTHH, which can lead to the loss of an expensive drilling system. DTHHs perform best in competent rocks (> 10 MPa) due to the percussive drilling action and the bore remaining stable and uniform; production rates are directly proportional to the pressure and volume of air provided. A 200 mm diameter DTHH requires a large, 750 cfm/170 psi, expensive-to-operate compressor; however, the equivalent micro-pile cost per metre will be considerably less than rotary drilling in rock. The air-flushed DTHHs require substantially dry bores; otherwise, large volumes of water will be blown into the air. In recent years DTHH overburden drilling systems (essentially a 'cased DTHH') have been developed, such as Odex, Symmetrix and Maxbit. These systems allow full-face DTHH percussive drilling with simultaneous advancement of the temporary steel casing to keep the bore open. Therefore, difficult granular strata with large boulders can be drilled efficiently using cased DTHH systems (see **Figure 81.44**).



Figure 81.44 Cased DTHH micro-piles

- **Rotary percussive micro-piles (including self-drilled hollow bar)** – These micro-piles are installed using top drive rotary percussive techniques and can achieve fast production rates. They generally require larger drill rigs to accommodate the heavy percussive drill heads ('drifter'), although smaller percussive heads are now appearing on ~5-tonne micro-pile rigs. The top drive percussive heads are noisy and the drill string loses energy into the ground as the bore extends deeper. There are two main types of rotary percussive micro-piles. The first is rotary percussive duplex drilled. This is similar to normal rotary duplex drilling, except that there is additional percussion which speeds up drilling through difficult ground containing cobbles and boulders. Unfortunately, large percussive drill heads are required to drive the heavy percussive drill casing/bit and therefore standard micro-pile sizes are 76 mm and 114 mm diameter. There is a 178 mm diameter system available but this requires an even larger, heavier and more powerful drifter, which requires a minimum 25 tonne base machine. The second system is the self-drilled hollow bar micro-pile, which is becoming very popular because of its rapid installation speed and the resultant low cost per metre. These micro-piles comprise a hollow reinforcement bar with an expendable drill bit which is percussively drilled into the ground using grout or water flush. It is preferable to use grout flush in difficult ground; however, if water flush is used during drilling to reduce grout waste, then it must be changed to grout flush immediately upon reaching the hole base.

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Grouting must then be continued until there is a full column of clean grout around the hollow bar. It is important to keep the drill flush emitting from the bore at all times to ensure that there is a full column of grout upon completion. If drilling with grout, then the micro-pile is complete as soon as the required depth is reached, and the installation of the next micro-pile can then commence. Production rates of up to 400 m per shift can be achieved with this system. The drill flush keeps the bore open during installation and there is therefore no need for any temporary drill casing. This type of micro-pile can be installed into very difficult ground conditions, and typical sizes range from 75 mm to 150 mm. The main risks with this type of micro-pile are the hollow bar deflecting off a boulder and/or loss of flush. This system is also commonly used for soil nails and ground anchors (passive ties).

- **Hollow segmental auger (HSA) micro-piles (segmental CFA) –** This type of micro-pile is similar to a continuous flight auger (CFA) pile, except that the auger is segmental. Typical diameters are 300 mm and above, although the larger diameters are not strictly micro-piles. The hollow stem augers are rotated into the ground in nominal 1.0 m sections to the required depth and then sand cement grout is pumped down the central hollow stem while the auger is rotating slowly. This is continued until grout can be seen exiting around the augers at the surface. The augers are then carefully extracted whilst continuing to pump grout and maintaining a positive grout pressure. The grout level in the central stem should be visible at the surface as each section of auger is removed, and if not then it should be topped up as necessary. It is not possible to have a continuous monitoring record of the HSA micro-pile construction as with CFA piles; however, maintaining a positive head of grout at all times will ensure good pile integrity. HSA micro-piles are popular in granular strata with high groundwater levels because the augers provide temporary support to overcome borehole instability problems.
- **Driven micro-piles** – These are generally bottom-driven vertical 150 mm to 200 mm diameter thin-walled 'closed end' steel tubes with SWLs ranging from 100 kN to 300 kN. They are installed quickly and efficiently using a small drill rig with internal drop hammer on to a dry concrete plug with no spoil produced. Alternatively, smaller tubes can be driven into the ground via an internal air powered grundomat hammer, without any drill rig (see **Figure 81.42**). These micro-piles may have SWLs of up to 100 kN. The tubes can be driven in various lengths and welded together as the installation proceeds. The depth of the micro-piles is dictated by a predetermined 'set' and when this is reached the tube is filled with high slump concrete or grout and a central rebar if required.

81.4.6 Micro-pile drill rigs and coring

There is a wide range of micro-pile drill rigs and capabilities, from small ~1 tonne rigs with 2 m masts which can track through house doorways, through to 15 tonne rigs with 9 m+ masts. The smaller rigs have a more limited range of drilling techniques and are generally slower (and therefore they install more expensive micro-piles), whilst the larger rigs can use any drilling system and are faster. Careful thought should always be given to choosing the appropriate micro-pile rig and drilling system and the advice of specialist micro-pile contractors should be sought at an early stage. Micro-pile drill rigs have low rotation speeds and high torque and can rotary core drill though concrete, masonry and other obstructions with standard tungsten-tipped steel casing; however, coring rates can be slow. DTHHs can drill quickly through unreinforced mass concrete. If there are significant amounts of coring and steel reinforcement then consideration should be given to using a specialist coring company (which uses high rotation speed and low torque diamond-tipped coring barrels).

81.4.7 Micro-pile construction techniques

One of the key issues with micro-piles is to choose the most appropriate drilling technique to ensure that the micro-pile can be bored safely and efficiently to the required depth. Look at the borehole logs and take careful note of the stratum type, the groundwater regime, obstruction drilling and whether there are any cobbles or boulders. Micro-piles are normally formed with cementitious grout; however, readymixed concrete can be used with larger 300 mm diameter micro-piles in dry open bores (checked by shining a light down). If the micro-pile is 'wet' then a measured small diameter (25 mm or 38 mm diameter) plastic tremie pipe must be inserted completely to the base. The grout mix is generally a neat CEM1 mix or a 1:1 sand:CEM1 mix, with the neat mix used for the smaller diameters and the sanded mix more popular for the larger diameters. The sand should be a well-graded washed sharp concreting sand or similar (i.e. not a 'soft' sand) and it should preferably be used in 25 kg bags. Bulk sand can be used but it must be measured using calibrated containers on site (e.g. 6 level buckets = 100 kg). The grout is mixed in either a paddle mixer or a colloidal (high shear) mixer, which is more efficient and 'wets' more of the cement particles thus leading to a superior low bleed-higher-strength mix (see **Figure 81.45**). The correct amount of clean potable water should always be added first, followed by the CEM1 and finally the sand (if required). The standard cementitious grout mixes are detailed below:

- Neat CEM1 grout (this 0.40 water:cement ratio mix will produce 72 litres of grout)
  - 40 litres of clean potable water
  - 100 kg of CEM1
- 1:1 sand:CEM1 grout (this 0.45 water:cement ratio mix will produce 113 litres of grout)
  - 45 litres of clean potable water
  - 100 kg of CEM1
  - 100 kg of sharp sand

Both of these mixes will produce high quality cementitious grouts with unconfined compressive strengths of >40 MPa at 28 days. The CEM1 should preferably be grade 42N or 52N. There are a wide range of sands available and difficulties may be experienced adding the full 100 kg of some 'softer' sands. If this occurs, *do not add more water*, which would only weaken the final grout strength; it is better not to add all of the sand (but record the amount added). If higher grout strengths



Figure 81.45 Colloidal high shear grout mixer



Figure 81.46 Colcrete flowmeter

are necessary then plasticiser can be added and the amount of water reduced accordingly. Grout fluidity can be checked using the Colcrete Flowmeter Test if required (see **Figure 81.46**). This involves pouring 1 litre of grout into the cone and then releasing it and allowing it to flow along the level 'wetted' channel. A flowmeter reading of 375 mm (±50 mm) is normal for a 1:1 sand:cement grout and 450 mm (±50 mm) for a neat CEM1 grout. Micro-pile grouting should be carried out in a continuous operation by pumping clean grout into the base of the micro-pile until all the debris has been flushed out of the bore and a column of clean grout remains. The temporary casing can then be removed in sections, ensuring that the internal grout level is fully topped up to ground level each time. Finally, the clean reinforcement cage or bar should be carefully plunged into the grouted pile with appropriate centralisers (~3 m centres). If the micro-pile is very deep (or in dry sandstone, which may cause a flash set) and there is concern over inserting the reinforcement after grouting, then it can be installed first; however, this does increase the risk of smearing the rebar on the side of the bore. Upon completion of grouting the grout level should be carefully monitored for a minimum of one hour and topped up as necessary until it has completely stabilised. In granular strata it is common for the grout level to continue to drop as the pressure head forces the grout to permeate into the surrounding ground. Additional pressure of 3 to 4 bar (43 to 58 psi) can be applied during casing extraction using a pressure head coupling to further enhance grout take, effective pile diameter and micro-pile capacity.

81.4.8 Micro-pile environmental, quality assurance and health and safety issues

Micro-pile construction has several environmental issues which need to be carefully managed:

- disposal of water flush, which may have high silt contents and be harmful to fish;
- disposal of air flush, which will contain fine dust particles;
- noise suppression, especially from top drive rotary percussive drilling systems;
- disposal of grout overflush and washout.

Micro-pile quality assurance is similar to other piling techniques and requires the checking of the plan position, verticality, borehole depth, diameter and reinforcement. In addition, special care needs to be taken with site-mixed grout with regard to quantities of water, cement and sand. Grout cube sizes should be 100 mm × 100 mm × 100 mm.

Micro-pile construction is labour-intensive with several health and safety issues:

- manual lifting of casings, drill rods and reinforcement;
- slippage with water and grout flush;
- working near rotating augers (auger guarding during boring is now a statutory requirement);
- grout mixing on site;
- working in restricted and noisy locations.

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81.5.2 Useful websites

American Petroleum Institute; www.api.org/
Federation of Piling Specialists; www.fps.org.uk
Geosystems digital library; www.geosystemsbruce.com/v20/html/ab_TechPapers.html
International Society of Micropiles; www.ismicropiles.org
Technical Papers from Byland Engineering; www.bylandengineering.com/pages/about/technical-papers-awards
Wave equation analysis of pile driving; www.pile.com

It is recommended this chapter is read in conjunction with

- Chapter 22 *Behaviour of single piles under vertical loads*
- Chapter 54 *Single piles*
- Chapter 82 *Piling problems*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

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Chapter 82
Piling problems

Viv Troughton Arup, London, UK
John Hislam Applied Geotechnical Engineering, Berkhamsted, UK

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The successful installation of piles requires a good understanding of the potential problems that can arise, which are highly dependent on the type of piling and the ground conditions in which the pile is installed. These can affect the geotechnical behaviour of the piled foundation and its structural integrity, as well as its impact on the environment.

For bored piles, the different methods of boring, the type of drilling tool and the way the ground is supported will have different effects on the ground. The structural integrity is also affected by the way in which the concrete is placed, the detailing and installation of reinforcement, and the final trimming of pile heads.

Driven piles are either low or high displacement piles. Installation techniques vary from drop hammers and vibratory techniques to hydraulic press-in methods. The technique chosen and the amount of ground displacement can affect both the behaviour of the pile and its structural integrity. Installation aids such as jetting and pre-boring require careful control, and environmental effects such as noise and vibration also require particular consideration.

Guidance is provided on how problems are commonly identified, how they are assessed, and the ways in which they can be resolved.

82.1 Introduction

There are several key references that provide a comprehensive background to the potential problems that can arise in piling. The more common problems for driven and bored piles are described by Fleming *et al.* (2009) in their handbook *Piling Engineering*. Specific sections are also included covering driven cast *in situ* piles, continuous flight auger (CFA) piles, and cast *in situ* screw piles. Thorburn and Thorburn (1977) CIRIA report PG2 provides a review of piling problems associated with the construction of cast *in situ* concrete piles. Healy and Weltman (1980) CIRIA report PG8 provides a survey of the problems associated with the installation of displacement piles and although some of the techniques have changed, much of the guidance is still valid. Tomlinson (1994) describes the potential problems associated with piles in specific situations. These include piling for machinery foundations and underpinning, piling in mining subsidence areas, in frozen ground, for foundations for bridges on land and over water, foundations in cast and energy piles.

Specifications and codes are also an important source of information and many of the requirements contained in these documents have been incorporated as a result of problems that have been identified in piling projects. The ICE *Specification for Piling and Embedded Retaining Walls* (SPERW) (ICE, 2007) covers most of the common types of piling and includes guidance as well as specification clauses. The sections cover bearing piles and piles incorporated in retaining walls as contiguous piled walls, secant and sheet pile walls and king post walls. The Federation of Piling Specialists (FPS, 1999) have also provided a commentary on the ICE Specification (SPERW) but this relates to the 1996 edition which has been superseded by the 2007 edition (ICE, 2007). Nevertheless, the FPS commentary includes some useful guidance, some of which has been incorporated in the ICE (2007) specification guidance. Execution of bored piles is covered in BS EN 1536:2000 (BSI, 2000a) and execution of displacement piles in BS EN 12699:2001 (BSI, 2001). The different types of bored and driven piles and their methods of installation are also described in Chapter 81 *Types of bearing piles*.

Noise and vibration control in piling is comprehensively covered in BS5228 (2009). Part 1 (BSI, 2009a) covers noise, including methods of noise control, guidance on methods for predicting and measuring noise, and assessing its impact. Part 2 (BSI, 2009b) covers vibration and means of vibration control, as well as guidance on methods of measuring and assessing vibration. Measured levels of vibration for piling are included in an appendix.

The durability of steel piling is covered in EC3 Part 5 (BSI, 2007a) and in the *Piling Handbook* (ArcelorMittal, 2008), which also contains useful practical guidance on pile driving. It details corrosion performance of steel piles and describes protective measures that can be adopted to increase their effective life. The durability of bored piles is detailed in both EC2 Part 1 (BSI, 2004) and EN206 (BSI, 2000b), which describe the exposure classes affecting durability and the requirements for concrete cover to reinforcement.

Many of the common problems in piling can be avoided by selecting the right piling system for the site conditions, and then ensuring that the installation is properly controlled and executed. A comprehensive site investigation is an essential component in selecting the appropriate method and the Association of Geotechnical and Geoenvironmental Specialists (AGS, 2006) guidelines are a very useful starting point and aide-mémoire.

With a good understanding of the ground conditions and the risks that can arise with different pile systems, the designer can select an appropriate method and control measures that will ensure successful construction of the piles. This is all part of the pile design process which is described in Chapter 54 *Single piles*. The following sections cover the common problems that can arise with the installation of bored and driven piles.

82.2 Bored piles

82.2.1 Ground conditions

It is important to ensure that the equipment to be utilised for boring is adequate for the ground conditions. This may seem obvious but the greatest unknown is still the ground. There may be unproven obstructions, both natural and man-made, that could prevent a 'less heavy' item of plant from completing the boring. Delays may occur if break-out tools have to be mobilised to site. Sometimes, the specialist contractor may know if natural obstructions are likely in a given area. There is also the possibility that, due to some unforeseen ground condition, the piles may have to be re-designed utilising a differing pile diameter. Whilst this should not be anticipated by the specialist contractor, his plant should ideally be able to switch to the alternative geometry.

By far the most common problem is the poor understanding of the ground conditions, usually due to inadequate site investigation. When this occurs, the client usually ends up paying for the cost and time lost caused by the 'unknown' conditions. There is also the common problem of a varying depth of founding stratum. This is more acute for CFA piling as the boring is essentially blind. Probing using the piling auger is possible, but is only a crude identifier of such depths.

Boring tools should be checked to ensure that the design diameter is to be achieved. The ICE SPERW (ICE, 2007) recommends a tolerance of up to 5%. Depth indicators and other instrumentation should be regularly calibrated, particularly if drilling tools or auger strings are changed. Other issues that can be of concern are smear in clays and weak rocks, bores left open too long before concreting and bases not adequately cleaned. This list is not exhaustive.

82.2.2 Boring

The correct use of temporary casings is essential for the success of good pile construction. Casings should always be driven or screwed in advance of drilling tools through unstable soils so as to provide a safe working environment and avoid over-excavation. A basic problem is found when, due to variations in the depth of unstable soils, the casing lengths used are inadequate and attempts to push them further into the ground risk the safety of personnel working around the bore (a minimum of 1 m should be left above commencement level as a safety barrier). With the less frequent use of high speed rotation, the practice of 'mudding-in' has become less common and the prevalent method is to screw/crowd (i.e. push down) the casing, sometimes as the auger is rotating within the casing. Mudding-in had the inherent problem of creating a weak zone around the casing which, if adequate concrete head were not utilised during subsequent casing extraction during concreting, could lead to a reduction in the concrete cross-section (known as necking) as the ground slumped into the bore.

This phenomenon of necking can also occur when withdrawing temporary casings if there is insufficient head of concrete to resist the external groundwater pressure. This is illustrated in **Figure 82.1**. To avoid this, the ICE SPERW (ICE, 2007) has specified minimum casting levels for concrete which are included in **Table 82.1**. This type of problem is most acute with long casings. The fall in concrete within the casing must be carefully observed as the casing is extracted in order to top-up

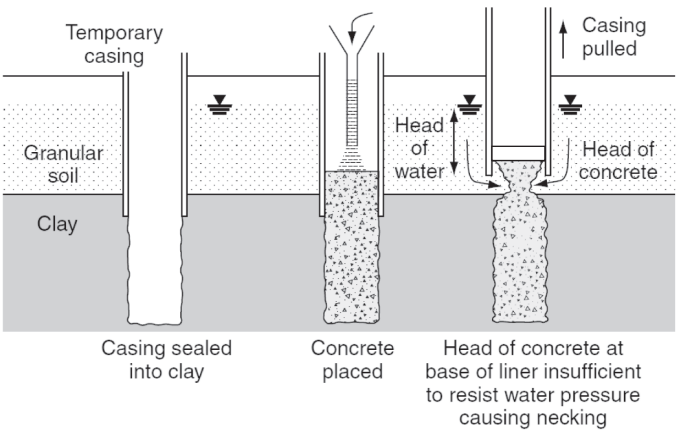


Figure 82.1 Necking in bored piles when the casing is pulled

Cut-off level below commencing surface, H[(1)] (m)	Casting tolerance above cut-off level (m)	Condition
0.15 to any depth	$0.3 + H/10$	Piles cast in dry bore within permanent casing or cut-off level in stable ground below base of casing
0.15–10.00	$0.3 + H/12 + C/8$	Piles cast in dry bore using temporary casing other than above
0.15–10.00	$1.0 + H/12 + C/8$ where C = length of temporary casing below the commencing surface	Piles or walls cast under water or support fluid[(2)]

[(1)]Beyond H = 10 m, the casting tolerance applying to H = 10 m shall apply.
[(2)]In cases where a pile is cast so that the cut-off level is within a permanent lining tube, or for a wall, the appropriate tolerance is given by deleting the casing term $C/8$.

Table 82.1 Concrete casting level tolerance above cut-off levels for specified conditions
Data taken from ICE (2007)

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the bore to prevent such pressure differential affecting the concrete placement. Vibrating casings is an effective method for the larger diameter bores, but care must be exercised as above and disturbance to previously concreted piles or other structures prevented.

The sequence of bored pile construction, whether for discrete bearing piles or wall piles, and whether by conventional rotary bored piling or CFA methods, is of importance to pile integrity. The problems usually occur in weak or soft soils where the shear strength of the soils is inadequate to resist the lateral pressures of the fluid concrete. If the concrete is poured to a level significantly above groundwater level (and it is essential for all bored piles to be concreted to at least nominally 1 m above groundwater level), the excess hydrostatic pressure can be significantly greater than that in the ground.

During boring, the fluid concrete in an adjacent pile can be disturbed and flow laterally if the piles are close together and the ground between them insufficiently strong to resist the lateral pressures. During concreting, the pressurised column of fluid concrete could exceed the *in situ* pressures in a recently cast pile and cause ground movement towards the previously concreted pile – thus possibly causing a reduction in pile cross-sectional area. It is normal practice to leave at least three pile diameters spacing between successive piles in any one working day. When using CFA methodology which uses positive injection pressures to assist sound concrete placement, this spacing may have to be increased.

When boring into aquifers under drilling fluid or water, it is important to raise and lower drilling tools slowly to prevent suction or pressurisation destabilising the bore as shown in **Figure 82.2**. Drilling buckets should have a sufficient bypass for the fluid to pass through or around the bucket, such as the flat-sided bucket shown in **Figure 82.3**, in order to reduce the suction or pressurisation effects. Care must be taken to check the properties of the drilling fluid regularly. If the fluid density is too low this can lead to bore collapse; if the fluid is dense due to suspended solids it can lead to clean-up problems. Bores must be cleaned and the drilling fluid conditioned and re-circulated after boring and cleaning to ensure that no 'sludge' remains at the base which could lead to concreting problems and poor base loading of the pile. Sampling of the drilling fluid should be undertaken with special tools.

One of the most common problems encountered in CFA piling is the tendency to 'flight' material up the bore and cause over-excavation of the weaker soils. This is illustrated in **Figure 82.4**; guidance is given in ICE SPERW (ICE, 2007). Essentially, care has to be taken, particularly in silts and fine sands, to ensure an adequate rate of penetration of the auger versus the number of revolutions. Excessive rotation of the auger without penetration should not be permitted.

Auger displacement piles are a form of piling where the auger displaces the ground without removing spoil. There are two types of displacement piles: the helical (screw) and the soil displacement. For the screw displacement pile, the rate of

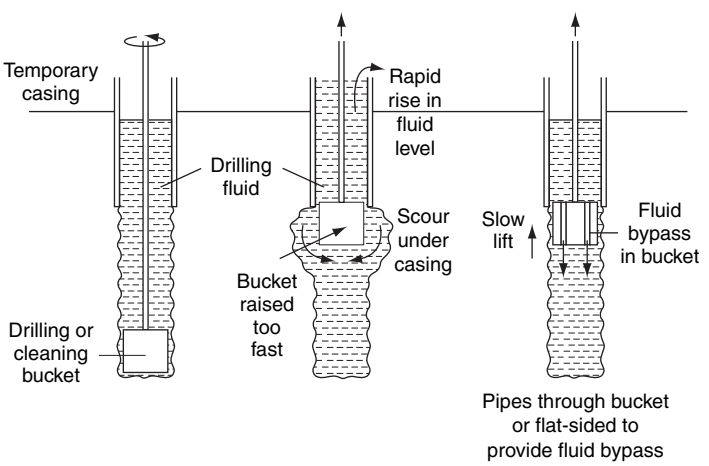


Figure 82.2 The effect of suction and scour on borehole stability when using drilling buckets



Figure 82.3 Flat-sided digging bucket to allow fluid to bypass
Courtesy of Balfour Beatty Ground Engineering

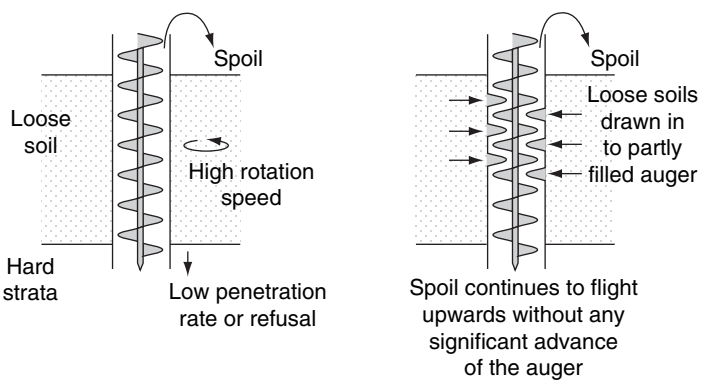


Figure 82.4 Flighting in CFA piles

penetration is controlled during insertion and removal of the auger to ensure that the auger follows the same path of flights – thus forming concrete ribs for the pile. Accurate monitoring and control of the rotation and penetration rate of the auger is essential to ensure integrity of the concrete ribs. For the soil displacement pile type, the soil is displaced without forming ribs. Substantial torque and crowd may be needed to ensure the auger penetrates the bearing stratum to a sufficient depth to achieve the required bearing capacity.

Other problems that may occur are discussed here. One problem is the meeting of natural or man-made obstructions. Even with the best desk study/collation of services information, the chances of hitting services are real. Smear to the bore walls, which can reduce the shaft friction, can occur in clays and weak mudstone due to over-rotation of the drilling tools. This is illustrated in **Figure 82.5** for a rotary bored pile shaft and in **Figure 82.6** for a CFA pile in mudstone which was deliberately over-rotated. If bores are left open too long, clays can soften and again result in lower shaft friction being mobilised. Pile end bearing can also be affected by allowing loose material to collect on the base. In a dry bore, a bladed auger or cleaning bucket is used to remove the loose debris. In piles that are bored under water or drilling fluid, suspended soil particles can settle onto the pile base and a cleaning bucket, air lift or submersible pump should be used to clean the base.

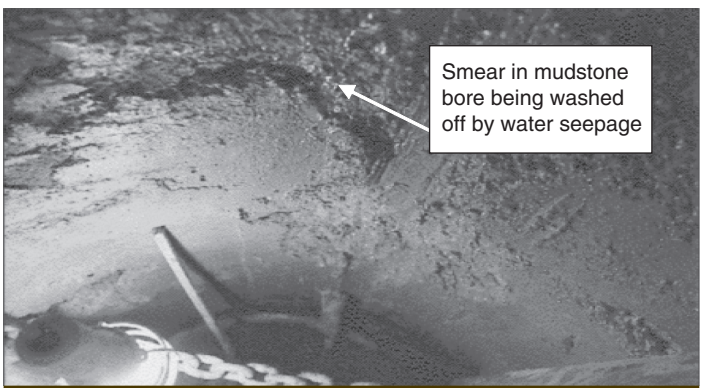


Figure 82.5 Smear in rotary bored shaft in weak mudstone

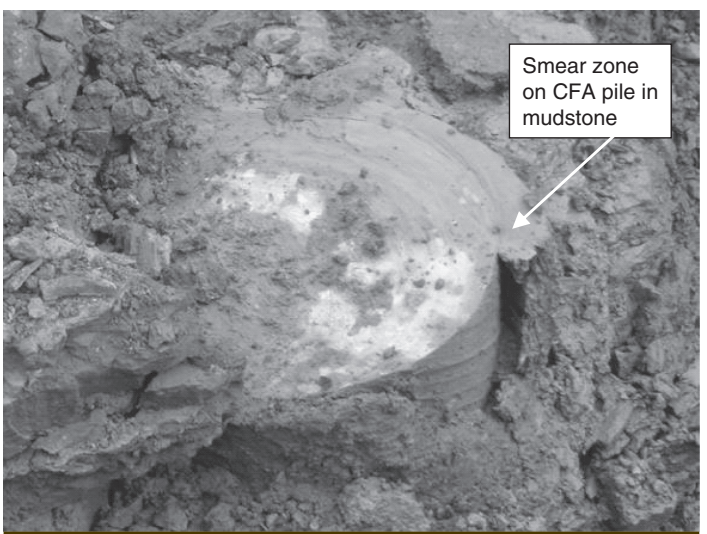


Figure 82.6 Smear on shaft of CFA bored pile in Mercia Mudstone
Courtesy of Balfour Beatty Ground Engineering

82.2.3 Concreting

Some of the problems associated with concrete placement have been noted in section 82.2.2, but this is by no means an exhaustive list. Since the most common method of concrete supply is via ready-mix suppliers, basic checks of the concrete delivery ticket, however mundane a process, can save problems with incorrect batching and distribution. The next most important check is of slump; as with all types of bored piling, high slump mixes are essential. Concrete that is too stiff is unsuitable for the self-compacting nature of the mixes employed and can cause blockages in tremie tubes and CFA delivery pipes. Placement pipes are desirable as they limit the segregation of concrete poured into bores, especially those that have pre-placed reinforcement cages. ICE SPERW (ICE, 2007) recommends the use of a rigid delivery pipe placed centrally in the bore to prevent the concrete hitting the reinforcement and so avoid segregation.

When concrete is placed inside a temporary casing, sufficient concrete needs to be provided to fill the extra volume created by the removal of the temporary casing. It is essential, especially for long casings, to monitor the internal concrete level and allow for topping up as casings are extracted.

There is the thorny subject for conventional bored piling of 'how wet can the bore be' before normal concrete placement has to be replaced by tremie placement. This is usually a problem encountered in otherwise dry cohesive ground, as the water table level in granular soils will normally be known before construction and the appropriate concrete placement method selected. Clearly, water entering the bore, whether from some way down the bore walls or from the base, influences the decision. On occasions where the amount of water entry is very minimal, it is impractical to use a tremie to place the concrete. In this situation it is quite adequate to either 'dry' the bore by the placement of dry cement or leave the final metres of boring until concrete is known to be ready on site for placement. If there is any doubt, the answer is to change concreting methodology and possibly pile design, so as to avoid the problem.

Whilst unlikely, the joints of tremie pipe assemblies can cause blockage if not adequately sealed due to the squeezing of water out of the fluid concrete by the excess pressure head in the pipe. Vigilance is important, particularly when removing and breaking such assemblies, and renewal of the rubber seal rings must be undertaken so as to prevent re-occurrence.

The removal of the tremie must be carefully controlled by logging the tremie depth as each volume of concrete is placed. Over-extraction of the tremie will cause a break in concrete quality in the bore with a zone of trapped detritus. To avoid over–extraction, ICE SPERW (2007) requires a minimum 3 m embedment for the tremie pipe in the concrete.

In CFA piling, a problem can occur either above or below the groundwater table due to the bleeding of water and possibly fines from the fluid concrete after placement. The problem is more prevalent above the groundwater table in dry granular soils and is caused by the squeezing of water/fines from the concrete due to the significantly higher pressure in the concrete versus the ground. Usually this problem can be foreseen and changes to concrete mix design (such as over-sanding and a higher dosage of plasticiser) are made. Problems also occur below the groundwater table for similar reasons but are not always evident before the commencement of work. Such problems are usually only detected when it becomes difficult or impossible to place reinforcement cages into the supposedly fluid concrete. The concrete in the affected depth zone will have lost its workability and present an obstruction to the smooth passage of the reinforcement cage. The remedy in these situations is to extract the reinforcement cage, re-bore, and re-concrete the bore so as to have workable concrete throughout the pile depth.

Another 'golden rule' is: 'once the concrete is placed, leave it alone'. That is to say, there is risk of contamination of the concrete in the bore if over-emphasis is made to clean up the top of the pile. However, with care, clean up and reduction of concrete level is achievable, even to the extent of preparing/casting pile caps integral with the piles.

82.2.4 Reinforcement

Congestion of reinforcement in heavy cages can lead to poor concrete placement with associated problems of design integrity of reinforcement bond. The problem is usually found when lapping multiple cages. BS EN1536 (BSI, 2000a) requires a minimum of 100 mm between main bars or 80 mm when using aggregate of 20 mm or less. On occasion it may be necessary to limit aggregate size to conform to these requirements, or use double circles of main reinforcement or bent bars to crank radially in the pile.

Poor handling and coupling of cages can be avoided by cage welding, which is now becoming more prevalent, or by using stiffeners, particularly for larger diameter cages. Pre-welded cages are usually more accurately produced than hand-tied cages and allow better matching of bars at laps. Correct lifting of cages should be observed to avoid collapse. Cages should be stored so as to remain essentially clean and must not become covered with mud or other debris.

Spacers to ensure adequate cover for the reinforcement to the bore walls do not always get the attention they deserve. After the expensive mobilisation of specialist equipment and manpower, procurement of high quality materials, do not allow the final product to be ruined for the sake of inadequate spacers – either by quality or number. Often the inadequacy of spacers only comes to light when the hardened pile is excavated or cut down – to reveal either no spacers, or bent or twisted ones that have allowed the cage to move eccentrically in the bore before the concrete has set and hence denigrate the finished product.

The level at which reinforcement is placed should be done with care so as to avoid subsequent problems with inadequate projection when follow-on trades take over the adoption of the piles into the works.

82.2.5 Ancillary works

After the completion of any one pile, the important thing is to ensure it is protected until the concrete has attained strength. Plant must not be allowed to travel near such piles as the fluid or partially set concrete cannot offer adequate passive resistance to the lateral pressures so induced (see **Figure 82.7**). This is often the reason for inadequate concrete cover or misaligned reinforcement cages. The clear identification of piles should also be a basic item of housekeeping on any site.

Other good housekeeping should include the removal of spoil and general area cleaning to create a safe working environment and prevent debris from entering the fluid concrete. This should be undertaken using appropriate reach plant, so as to avoid surcharging the piles.

Cutting down piles must be done with care. The use of laterally applied breakers, unless of a multi-circular configuration, must not be permitted. Ideally, proprietary multi-teeth breakers should be utilised. The removal of the top concrete core is assisted by the use of foam sleeves affixed to the reinforcement above cut-off level. FPS (2008) have produced a guidance note covering this topic and there is also a CIRIA (Cox, 2009) paper on the subject.

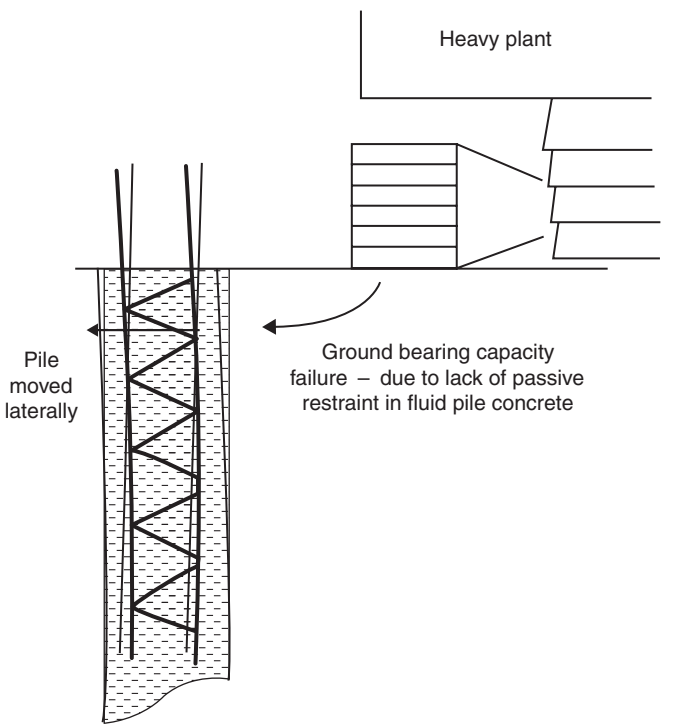


Figure 82.7 Damage to piles from nearby plant movement

82.2.6 Structural problems

Structural problems arise when reinforcement is placed at the incorrect level, or the cage becomes displaced due to poor use of spacers or mechanical displacement. The proving of concrete strength can be fraught with practical difficulties such as poor or dirty cube moulds, inadequate labelling, poor storage, curing tanks not being temperature controlled and poor off-site testing. Fortunately, the industry is moving to a self-certification system that is the province of the concrete supplier. The ready-mix industry has taken it upon itself to overcome the known shortcomings of this facet of construction and now collects samples from sites and produces records of the set concrete properties.

Pile eccentricity can cause problems, particularly if the designer has not taken account of the tolerance latitude that is permissible.



Figure 82.8 Ends of pre-cast piles, not cast square

82.3 Driven piles

82.3.1 Installation methods and ground conditions

BS EN 12699:2001 (BSI, 2001) covers the execution of concrete, steel, timber and cast iron displacement piles. Driven piles include low displacement piles such as open-ended steel tubes, steel sheets and steel H-sections, and high displacement piles such as pre-cast concrete sections, closed-ended steel tubes, timber piles and driven cast *in situ* concrete piles. Piles are installed by using drop hammers, vibratory techniques or by hydraulic press-in methods.

Hydraulically operated drop hammers are most commonly used to drive pre-cast concrete sections, and vibratory methods and drop hammers are both commonly used for steel piles. Hydraulic press-in methods are used to install steel sheets and some tube sections in areas where noise and vibration are to be avoided.

Adequate site investigation information is critical for making an assessment of driveability. The presence of obstructions and hard layers can affect driveability and additional measures such as jetting and pre-boring may be needed for the pile installation.

82.3.2 Pile manufacture

The manufacture of pre-cast concrete piles is covered by BS EN 12794:2007 (BSI, 2007b). The piles should be designed to withstand handling and driving stresses which are often considerably greater than the stresses they will experience in service. Pre-cast concrete piles are cast in moulds; the concrete strength needs to be sufficient for their removal from the moulds, handling and transportation, and then for final driving. The piles are cast in lengths that will subsequently be jointed on site. The ends of the sections therefore need to be cast square to the pile to ensure they fit tightly together and avoid eccentricity of loading (see **Figure 82.8**).

Steel and timber pile manufacture is unlikely to present any particular issues. Straightness of the section is important to avoid induced bending during driving.

82.3.3 Pile installation

Piles installed using drop hammer or vibratory techniques are held in leaders or guide frames to guide the hammer or vibrator, pitch the pile and hold the pile during the driving process. For hydraulic press-in methods, the hydraulic jaws clamp onto the pile to guide and feed the sections into the ground.

Over-driving, described by Healy and Weltman (1980), can often occur when attempting to drive piles to an unreasonable design depth. This can result in permanent damage to the pile head which is visible, or to the pile toe where it is unseen. In the case of toe damage, a pile may appear to be driving normally but the set may not build up as expected due to crushing or distortion of the pile toe. Where it is important to reach a particular depth, jetting or pre-boring should be considered.

Jetting and pre-boring need particular care to ensure that they do not cause disturbance to the ground that would affect adjacent structures such as utilities and buildings, or affect the performance of the piles. For jetting, a high pressure water jet is used to erode or loosen the soil at the pile tip. Water returns and pressures should be monitored to ensure the water is not finding an alternative flow path away from the pile tip. If the returns are blocked, pressure can rapidly build in the ground and cause heave. In pre-boring, an auger is used to loosen the soil prior to pile installation. Spoil removal should be limited to the minimum necessary to aid pile installation without affecting the geotechnical performance of the pile. Pile behaviour is best checked by dynamic or static test loading.

Damage during driving can occur, particularly at the pile head where the stresses are highest. For pre-cast concrete this will take the form of cracking and spalling of the concrete as shown in **Figure 82.9**, and on steel sections the pile head may bend or buckle. In order to prevent spalling, pre-cast concrete pile suppliers normally provide a protective steel band at the pile head and keep the concrete cover to a minimum whilst

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Figure 82.9 Damaged pre-cast concrete pile head
Courtesy of Balfour Beatty Ground Engineering

still complying with durability requirements for protecting the reinforcement (Healy and Weltman, 1980 CIRIA report PG8). Damage can also occur within the pile due to tensile stresses. This can occur anywhere in the pile shaft when piles are driven using excessive hammer energy in soft ground and the damage is not noticed until harder driving occurs later towards the end of the drive. The cracking induced earlier in the pile shaft then leads to spalling and compressive failure at depth.

Pre-cast piles are usually reinforced to resist these driving stresses and breakages are relatively uncommon. They tend to happen when obstructions are encountered. Although piles are relatively slender, buckling is unlikely because of the restraint provided by the ground. Even soft ground will usually provide adequate support to prevent buckling during driving.

Where steel pile heads are damaged during installation, they can be cut back and extension pieces welded on. Requirements for structural welds can be difficult to achieve under site conditions and usually require verification testing which can delay the works. Procedures and testing requirements for both off-site and site welding are detailed in ICE SPERW (2007) and general guidance for arc welding is covered in BS EN 1011 Part 1 (BSI, 1998). Repairs to a concrete section can be made but delays are likely to be considerable due to the time required for the new section to cure.

Particular care is needed where driven piles are to be founded in a dense layer of variable thickness that is underlain by weaker material. Piles may punch through the dense layer in some locations before reaching the required set. In these circumstances, detailed site investigation information regarding the variability of strength and thickness of the founding stratum is critical. This information may need to be backed up by driveability trials, selecting areas close to boreholes where the founding stratum thickness and strength are the most critical.

During driving, steel tube piles can plug. This is where the soil inside the tube moves down instead of up as the pile continues to be driven. The mechanics of whether plugging is likely to occur have been investigated by White *et al.* (2002). They have shown how this affects the pile capacity and describe measures for reducing the effects by fitting a pile shoe. Piles can also be made to deliberately plug by welding an internal plate to prevent upward travel of soil inside the pile. Plugging in H-pile sections is very rare according to Biddle (1997) and should not be assumed unless it is demonstrated during driving.

Particular care is needed with founding driven piles onto buried inclined rock surfaces and rock head which contains boulders. These conditions can potentially deflect the pile off-line, inducing bending and damage to the pile and can lead to uncertainty in establishing a reliable set. Steep-sided rock faces, such as the edges of backfilled quarries, present a particular risk.

82.3.4 Final set

Driven piles are normally driven to a final set which is usually defined as the measured penetration for 10 blows. False sets can occur where there are obstructions or boulders in the ground, or where thin rock bands may overlie weaker layers. Trial drives at various locations around the site are a way of verifying consistent founding conditions and compatibility with the expected ground conditions and design. Pre-boring can be used where the pile driving is likely to be impeded by obstructions.

In very stiff clays and chalk the pile resistance can increase with time; this is known as set-up. A delay in the driving can then lead to extra energy being required to get the pile moving again. The increase in pore water pressure developed during driving reduces the ground resistance to driving. As this excess pressure dissipates, the resistance increases. This is beneficial to the long-term capacity but it is important not to test the pile too soon after installation – otherwise the pile capacity will be underestimated. If the problem is assessed as potentially significant, piezometers can be used to monitor the dissipation of the excess pore water pressures.

In dense silts and some weathered rocks, false sets can occur. This has been suggested to be the result of negative pore water pressures being induced by the pile driving. The dissipation of the negative pore water pressure can lead to a relaxation of the pile set. It is therefore necessary to carry out re-drive checks in these circumstances to check for any relaxation or reduction in pile resistance. The problem of relaxation can also occur in some mudstones where it has been suggested that the clay in the joints squeezes out or consolidates when load is applied after driving.

82.3.5 Downdrag

In soft clays and silts, the driving of displacement piles can cause excess pore pressures to develop which will dissipate over time. As the excess water dissipates, the soil will consolidate leading to an additional downdrag load developing in the pile. Pre-boring and sometimes slip coatings can be used to

reduce these effects. Generally, slip coatings have not proved very effective and their integrity after driving is uncertain. The additional downdrag load will usually be assessed and included as an additional load in the pile design. Downdrag loads can also occur in loose granular soils where the driving causes compaction of the soils.

82.3.6 Ground displacement

The process of driving a pile into the ground causes displacement of the ground which can lead to heave and lateral soil displacement in incompressible soils, or compaction and settlement in loose granular soils. The effects are greatest for high displacement piles in incompressible ground. In a confined situation such as a cofferdam, the cumulative effect of driving several piles can lead to increased ground displacement and lateral pressures. These can affect the cofferdam structure or make further driving of piles increasingly difficult as the ground tightens up. Use of low displacement piles, pre-boring and careful sequencing of the pile installation can all help to mitigate these effects. For large structures such as oil tanks, it is normal practice to drive piles from the centre working outwards. When working near to retaining walls, piles should be installed by working progressively away from the wall.

Preformed piles, such as pre-cast concrete and steel sections, can all be re-driven if there is a suspicion that they have been uplifted by driving adjacent piles. This is not the case for cast *in situ* driven piles, where an assessment would need to be made of any potential effect on performance.

82.3.7 Pile trimming

Poorly controlled pile trimming is probably the most common cause of damage to pre-cast concrete piles. *Breaking down of piles*, FPS (2008) provides guidance on various methods and states that heavy impact breakers should not be used on small diameter and lightly reinforced piles, or piles in soft ground. Cox (2009) also describes the different methods of pile trimming and the failure of the pile reinforcement that can occur with some systems. In particular, he notes the specific benefit of using debonding foam to avoid pile damage.

Trimming of steel piles should be carried out using a disc cutter or steel saw and not with burning equipment which will reduce the strength of the steel.

82.3.8 Jointing piles

Pre-cast concrete piles are usually supplied in sections which are connected by joints. There are two types of joints in general use. The first is a full strength mechanical joint which provides a compression, tension and moment connection. The second non-mechanical joint provides a compression connection only. Failure of joints is rare, but mechanical joints should be used where a lot of obstructions are known to be present.

For steel-cased piles, the problems of jointing casing lengths are associated with the practical aspects of site welding, including preparation and inspection. Again, the recommendations provided in SPERW (ICE, 2007) are highly relevant. Occasionally, threaded joints from second-hand or non-prime drill casing are employed as a means of avoiding the delays associated with welding. The casing sections are joined by driving the end of the casing with the male thread into the female internally-threaded collar. This provides an interference fit that is only suitable for compressive loads (and not tensile loads). For bottom-driven thin wall steel-cased piles, the casing sections are usually provided with simple interference fit spigot- and socket-type connections that can be jointed by tack welding. Alternatively, riveted connections can be used as a temporary expedient for the casing where the connection is only required to resist tension forces during driving. Thin-walled steel casings are also employed for Odex drilled-in piles, and in these cases tack welding is also satisfactory for jointing as the casings, left in permanently, are not considered structurally in the pile design process.

82.3.9 Noise and vibration

Noise is a potential problem for driven piling operations, particularly systems that are top-driven. Noise levels are generally less for bottom-driven methods where the hammer is contained within a driving tube or where vibratory methods are used. Consideration needs to be given to the proximity to noise- and vibration-sensitive areas. Measures can be taken to reduce the noise with acoustic shrouds and by restricting working hours. Alternatively, hydraulic push-in systems can be considered which have very low noise levels. Field measurements using this system have been compared to the existing recommended limits by White *et al.* (2002).

BS 5228 (2009) is the code of practice for noise and vibration control on construction sites and Part 1 (BSI, 2009a) covers noise. It provides guidance concerning methods of predicting and measuring noise, and assessing the impact on those exposed to it. Noise control from piling sites is specifically addressed in Section 8.5 and types of piling are discussed in Appendix H of the code. For top-driven systems, noise reduction can be achieved by introducing a non-metallic dolly (such as timber) between the hammer and the driving helmet. Acoustic shrouds have also been used to enclose the driving equipment. Data will normally be available from piling contractors for their particular system.

The driving of piles into the ground creates stress waves and vibrations that can affect the environment. The effects of vibration can be reduced by modifying the impact or frequency of the pile driving equipment. Vibratory piling methods also induce vibration in the ground. On some occasions, isolation trenches can prove useful in protecting neighbouring structures as they act as an inherent barrier to such ground-borne vibrations. However, 'freak' passages of vibration, possibly through susceptible strata, can also occur. As an alternative for steel piling, hydraulic push-in piling methods can be adopted which result in very low levels of ground vibration. White *et al.* (2002) have provided prediction curves showing that this system has

low vibration levels that enable it to be used much closer to residential buildings than impact or vibratory methods.

The level of vibration depends on the source and the distance from it. Even very low levels of vibration can be sensed by humans but much larger levels are necessary to cause damage to structures. BS 5228 (2009) is the code of practice for noise and vibration control on construction sites and Part 2 (BSI, 2009b) covers vibration. It provides guidance on methods of measuring vibration and assessing its impact on the environment. Annex C provides current measured vibration levels for piling and Annex D provides historic data.

The threshold of perception of vibration for humans occurs at very low levels of peak particle velocity (PPV) in the range 0.15–0.3 mm/s. The threshold criteria for damage to buildings occur at much higher levels and are presented in BS 7385–2 and are summarised in Annex B of BS 5228–2 (BSI, 2009b). These depend on the type of structure, the peak particle velocity of the vibration and the frequency range. In general, for reinforced or framed structures and industrial and heavy commercial buildings, the threshold level for cosmetic damage from transient vibration is 50 mm/s. For unreinforced or light framed and for residential or light commercial buildings the threshold is 50 mm/s for frequencies above 40 Hz, but reduces to 20 mm/s at 15 Hz and down to 15 mm/s at 4 Hz. Minor damage can occur at twice these values and major damage can occur to building structures at four times these values.

82.3.10 Durability

For concrete piles, the durability depends on the concrete quality and the concrete cover to the reinforcement. Healy and Weltman (1980) note that fine cracks of up to 0.2 mm in width may occur in high quality piles and these are not generally regarded as being of concern because they close up under the dead load from the structure. Larger cracks may occur as a result of improper handling, pitching or driving. The seriousness of excessive cracking depends on the aggressiveness of the ground into which the pile is being installed and the durability requirements. For concrete structures, these are covered in Eurocode 2 Part 1 (BSI, 2004).

Corrosion of steel piles is most likely to occur either in disturbed ground or in a marine environment. Corrosion rates for steel piling in natural environments are given in Eurocode 3 Part 5 (BSI, 2007a) as shown in **Table 82.2**. Various methods are available to combat corrosion and these are described by Fleming *et al.* (2009). Guidance is also provided by ArcelorMittal (2008) in their *Piling Handbook*.

82.4 Identifying and resolving problems

82.4.1 General guidance

The following sections describe the way in which piling problems are often identified, how they can be assessed, and the ways in which they can be resolved. Quality management systems and the process of identifying piling non-conformance are described in Chapter 93 *Quality assurance*. Problems often arise from the results of pile tests and useful guidance on pile testing and the interpretation and assessment of tests is provided in the *Handbook on Pile Testing* (FPS, 2006) and by Tomlinson (1994). Poulos (2005) also provides a range of case studies of piling problems and comments on the particular issues involved in investigating and analysing them.

82.4.2 Identification

There are a range of tests and records that can be used to identify potential problems. These include:

(i) Piling records – inconsistencies in ground conditions, driving records, out-of-position piles, uplift and lateral displacement, anomalies in concreting or reinforcement records.

(ii) Integrity tests – acoustic anomalies in integrity traces indicating the possibility of cracks, changes in section or inclusions in the pile. Guidance on different types of integrity tests and their interpretation is provided in Chapter 97 *Pile integrity testing* and by Turner (1997) in CIRIA Report 144.

(iii) Load tests – failure to meet the specified load/settlement behaviour, unexpected settlement behaviour indicating structural failure or low shaft resistance or a soft toe,

		Required design working life (years)				
Water	**Zone**	**5**	**25**	**50**	**75**	**100**
Common fresh water (river, ship, canal, etc.)	High attack (water line)	0.15	0.55	0.90	1.15	1.40
Very polluted fresh water (sewage, industrial effluent, etc.)	High attack (water line)	0.30	1.30	2.30	3.30	4.30
Sea water in temperate climate	High attack (low water and splash zones)	0.55	1.90	3.75	5.60	7.50
Sea water in temperate climate	Permanent immersion or in the intertidal zone	0.25	0.90	1.75	2.60	3.50

Notes
1. The highest corrosion rate is usually found in the splash zone or at the low water level in tidal waters. However in most cases, the highest bending stresses occur in the permanent immersion zone.
2. The values given for 5 and 25 years are based on measurement, the other values are extrapolated.

Table 82.2 Recommended values for the loss of thickness (mm) due to corrosion for piles and sheet piles in fresh water or in sea water
Data taken from BSI (2007a)

sometimes identified by using back analysis of load test results using methods such as those of Chin (1970) and Fleming (1992).

(iv) Materials tests – concrete cube tests not meeting the specified strength, failure of weld tests on steel connections.

(v) Inspection of trimmed piles – damaged pile head, weak concrete, buckled steel section.

(vi) Observations from the resident engineer or inspector staff detecting non-conformances with the specification that lead to further investigations that highlight problems.

Piling problems do not always arise from a non-conformance with the specification and may become apparent from observed inconsistencies. Sometimes the difficulty is to realise when there is a significant issue that needs further investigation. For example, a successful preliminary test on a bored pile in dry conditions in one part of a site may be found to be unrepresentative of wet conditions that become apparent in other areas. Another example is of driven piles which may all reach the required set but some are founded at a much higher level. Where this is unexpected and could affect the design, further investigation or testing may be needed.

82.4.3 Diagnosis

Test results should be viewed critically to ensure they are accurate and reliable, and they should be considered along with all the other information from pile records and knowledge of the site conditions. Further investigation may be necessary in the form of further testing or physical investigations before determining what remedial action, if any, is required. In some cases the test itself may be limited in what it can reveal.

For example, integrity testing in the form of sonic echo tests or cross-hole sonic testing relies on the interpretation of acoustic waves to identify potential anomalies in piles. Whilst these tests can indicate acoustic inconsistencies within the pile and between piles, this should be verified by physical investigation or load testing and a review of the pile records before deciding whether the anomalies are structurally significant. Similarly with load testing, unexpected pile load/settlement behaviour may be the result of variations in ground conditions or an inadequately performed test. Further evaluation and testing may be required to confirm the cause of poor or unexpected performance.

In some cases the anomalous test results may be the result of follow-on works where piles have been damaged by trimming or ground movements due to other works. It is therefore important that the testing is timely and carried out during the piling works or immediately on completion rather than at a later stage – otherwise the cause of the problem may be difficult to attribute to a specific contractor. It is advisable that testing is carried out as an ongoing process during the piling works so that potential problems can be identified during the work and remedial action taken whilst the works are in progress.

82.4.4 Resolving problems

Where tests or records indicate a potential problem with the piling, it is important to understand the impact of the problem for the structure. The issue may affect the durability or the structural performance.

In some cases, the problem may be easily accommodated by the structure without any remedial action, or with only minor remedial works. For example, with out-of-position piles, the pile reinforcement or the pile cap or ground beam arrangement may be sufficient to accommodate the eccentricity without any remedial action. Also, a pile load test that just exceeds the specified settlement limit may not be structurally significant; the structure may be able to accommodate slightly larger settlements than were originally specified without detriment to the overall structural performance. In these cases, whilst there may be a non-compliance with the specification, it is important not to overreact as it may be easily resolved by further assessment and analysis. It is not in the client's best interests to conduct a major investigation into apparent anomalies which may have no real significance for the structure.

Some problems may indicate a more fundamental issue with the piling which may affect the performance, safety and long-term durability of the structure. This will require a clearly defined strategy of investigation and assessment. The process should determine:

- the structural significance of the problem;
- whether other piles may be affected;
- any further testing that is considered necessary;
- remedial works if required.

The procedures for dealing with non-conformance should be defined in the contractor's quality plan. The actual procedure will depend on the contractual arrangements and whether the contractor's workmanship will be verified by an external party or whether the contractor will self-certify his work. The different quality management systems are described in Chapter 93 *Quality assurance*.

A significant area of misunderstanding can arise from the process of exploring the anomaly. Tests to verify that a pile can be safely incorporated into the works are conventionally paid for by the client (as the pile is ultimately acceptable), while tests that reveal defects that need remedying are paid for by the contractor. Payment for consequential costs is often contentious so it is in everyone's interest that the time for resolution is minimised.

The important underlying principle is that the lines of communication and project responsibilities should be clearly defined so that any non-conformance can be reported and remedial action proposed, assessed and approved as quickly as possible. Where this is likely to impact on cost, program or safety, the client needs to be kept informed of the solution.

Disclaimer

The example non-conformances included in this chapter are for illustrative purposes only and are not associated with, or reflective of the authors or their employers.

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82.5.1 Useful websites

Association of Geotechnical and Geoenvironmental Specialists (AGS); www.ags.org.uk

It is recommended this chapter is read in conjunction with

- Chapter 96 *Technical supervision of site works*
- Chapter 97 *Pile integrity testing*
- Chapter 98 *Pile capacity testing*
- Chapter 101 *Close-out reports*
- Section 5 *Design of foundations*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 83
Underpinning

Tim Jolley Geostructural Solutions Ltd, Old Hatfield, UK

Before a final decision is made on the choice of underpinning for a particular application, there are several aspects to be considered. The various types of underpinning available are discussed in order to provoke thought on which type may be worth further consideration for the project at hand. Some factors are given which may influence the choice of underpinning to enable an informed decision to be made. Finally some specific points are covered in relation to groundwater, subsidence settlement, safety and the financial aspects of underpinning.

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83.1 Introduction

83.1.1 Where underpinning is carried out and why

There are three main areas, namely:

(a) rectification of distress;

(b) extending foundations downwards to enable building and civil engineering works to be carried out;

(c) additional loading on an existing building.

83.1.2 Objective of the paper

Irrespective of the reason that underpinning is required, there are some common fundamental principles worthy of consideration prior to design or installation of an underpinning scheme. Some points are highlighted which may need to be borne in mind prior to embarking on an underpinning scheme whether it is at design stage or at construction stage. Information is therefore provided to provoke thought in order to facilitate correct decision-making at an early stage.

83.2 Types of underpinning

83.2.1 Mass concrete

Mass concrete underpinning is used as an extension of an existing masonry wall or footing. This may be necessary as a result of a requirement to lower the ground floor of a building where there is insufficient depth of existing foundation.

Another common use of mass concrete underpinning is to transfer the load of a building to stable ground as in the case of a building having suffered the effects of subsidence-related settlement. Where this is so, it is important to have sufficient information about the nature of the ground at an early stage in order to ascertain the likely depth at which stable ground will be found. This will enable a decision to be made on whether mass concrete underpinning is an appropriate solution in this case.

Where an existing building is being considered for the addition of further stories resulting in an increased loading on the foundations, mass concrete can be used to provide the required increase in bearing capacity of the soil upon which the building is founded. The proposed underpinning can be designed to increase the width of the existing footing or alternatively, to deepen the existing footings in order to found onto soil having greater bearing capacity.

If a new basement construction is proposed below or adjacent to an existing building, then mass concrete may be used to create a new wall to retain the soil behind. Generally a reinforced concrete lining wall is used in conjunction with the underpinning to provide permanent support. In order to maximise the space available in the proposed new basement room, the underpinning profile is often maintained in line with that of the existing walls. The underpinning profile is then widened at a point below the level of the new floor to achieve the same width as that of the original footing. In order to determine the degree of propping required in the temporary case, the proposed underpinning should be designed as a mass concrete gravity retaining wall. The proposed reinforced concrete lining wall will be designed to provide permanent restraint.

83.2.2 Hit and miss underpinning

Where appropriate, for reasons of economy, the mass concrete underpinning bases can be spaced apart in a ‘hit and miss’ pattern. As the loading from the structure above will now be concentrated onto a smaller area it is particularly important to check that the soil upon which the underpinning is founded does not become overstressed.

It is also important to consider the integrity of the structure which spans between the bases. In some cases it may be appropriate to introduce a lintel or series of lintels placed side by side to span between each adjacent base.

83.2.3 Reinforced concrete underpinning

Reinforced concrete underpinning bases are used most commonly when creating a new basement space below an existing building. They should be designed as a series of vertically spanning beams having a typical width of an individual underpinning base, say, 1.2 m. Continuity of reinforcement between the individual bases is therefore restricted to a series of dowel bars in order to ensure the new retaining wall acts as a homogeneous mass.

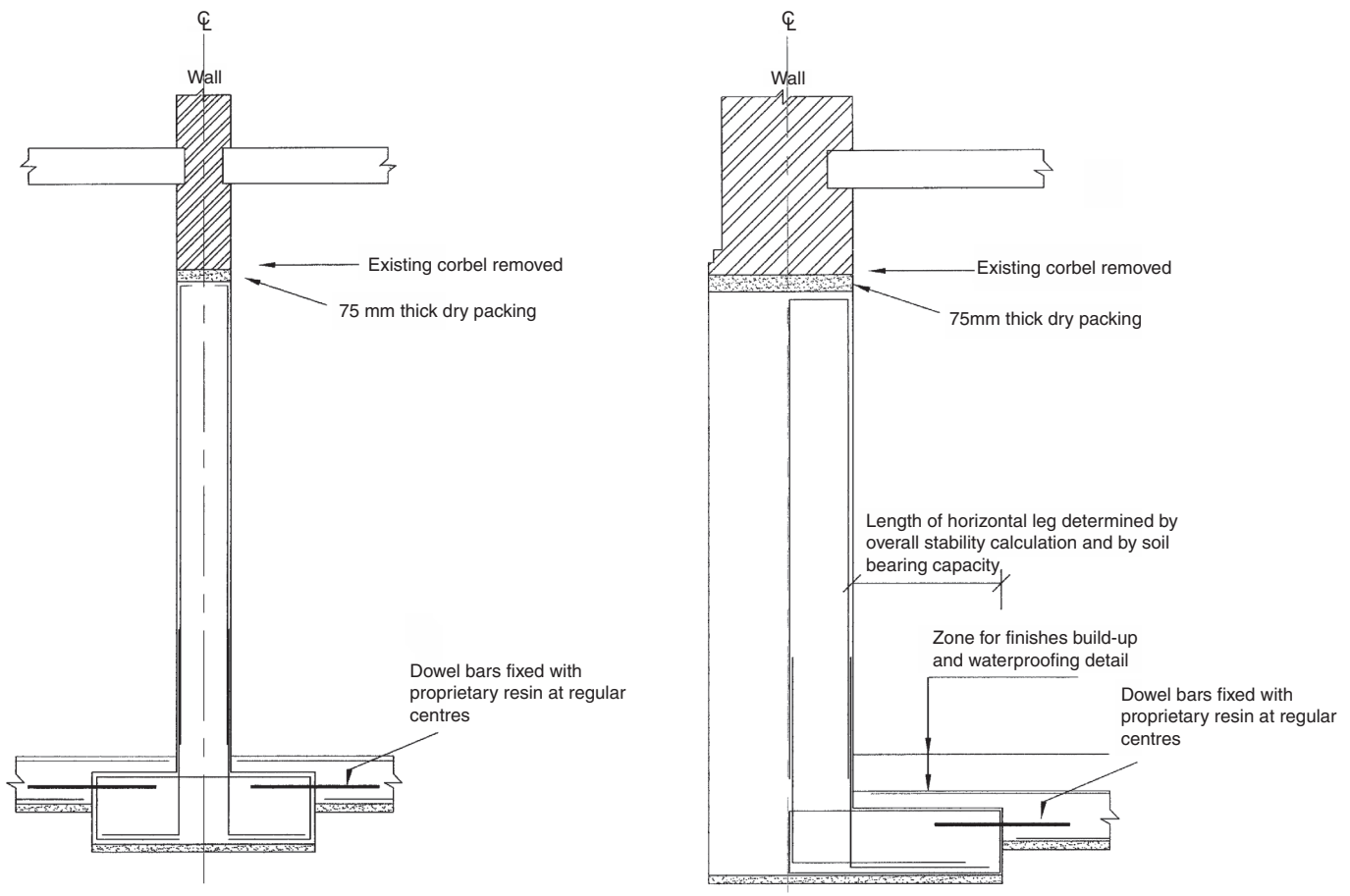
A reinforced wall is designed as the primary means of soil retention. In the permanent case, the wall will span as a beam between the various floor levels but in the temporary case, the underpinning may be required to resist the possibility of overturning and forward sliding. Depending on the circumstances surrounding each individual case, an 'L'-shaped underpinning profile may need to be adopted. The sectional dimensions of the horizontal leg will be determined from design analysis. Given the conditions in which underpinning bases are formed, it is advisable to provide a tolerance for construction of the base slab in relation to the horizontal leg. **Figure 83.1** shows a means of providing an economic design whilst allowing construction tolerance.

For reinforced concrete underpinning, the waterproofing is normally provided internally in the form of a cavity drain membrane or a flexible coating system. Waterproof concrete for construction of the reinforced underpinning bases can be considered but depending on the circumstances for a particular project, is sometimes found to be both impractical and uneconomical.

83.2.4 Piled underpinning

If an entire property is to be underpinned for reasons of subsidence-related settlement then a piled raft scheme should be considered. This is especially so if the ground conditions dictate that the depth of mass concrete underpinning is likely to be in excess of, say, 2.5 m. This figure is based largely on economic and practical considerations which are set out in section 83.10.

A system commonly used to underpin an entire building for subsidence-related settlement is formation of the raft by use of piles situated entirely within the building. A raft slab is formed to support the external walls by cantilevering out from piles placed just inside the building. In this way external drainage and other



(a) An internal wall

(b) A party wall

Figure 83.1 Underpinning detail for (a) an internal wall; (b) a party wall

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services around the building are not affected by the piles. Internal walls are supported by piles placed either side. Sacrificial stools are used to temporarily prop the walls during installation of the reinforced concrete raft. Typically an entire building would be supported on a raft foundation formed from, say, three pours. In this way, stability to the building is ensured at all times.

For partial underpinning a 'needle pile' system can be used. A reinforced concrete beam is formed under the wall or preferably, within the depth of the existing foundation. If conditions permit, piles are placed each side of the wall at regular centres, say, 3.0 m with reinforced concrete needle beams to connect them together. **Figure 83.2** illustrates this technique, used at



Proposed beam spanning between needle



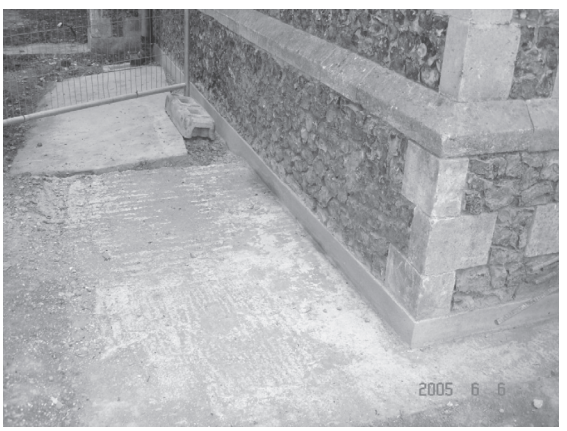
Stools placed in pairs and jacked into position



300mm diameter piles at needle locations



Completed beams and needles prior to dry packing



Finished works ready for landscaping

Figure 83.2 Piled underpinning, St John's Church, Wembley

St John's Church, Wembley. The brief was to underpin the external walls in order to overcome the effects of differential settlement caused by clay shrinkage within the prevailing London Clay soil. Sacrificial stools were used to form a reinforced concrete beam within the depth of the existing masonry structure, and needle beams connected through to bear on piles installed either side of the wall at regular centres.

When access to the interior of the building is not possible or is not preferred then a variation to the above described needle system can be used. Pairs of piles are placed on the external side of the building to act as a tension and compression couple.

Alternatively, a single pile can be used on the external side of the building. In this case, the pile is designed to support the vertical load but is combined with a design bending moment resulting from the lever arm between the pile centre and that of the wall. A reinforced concrete beam can be placed under the wall and extended at regular centres to form a series of pile caps. Depending on the loading conditions, the centre of the piles can be closed up so that the individual pile caps penetrate into the wall to form a series of castilations on a hit and miss basis.

A system of underpinning which avoids the requirement for a reinforced concrete beam using sacrificial stools is called 'Pali Radice' meaning 'root pile'. Typically a vertical pile is formed externally. This pile is coupled with another pile drilled at an angle through the existing foundation of the building. The two piles are designed to act in unison to support the building.

Irrespective of the type of piling used, it is always essential to ensure that there is sufficient bond between the old structure and the new pile to get the required load into the new pile. This is particularly important for the Pali Radice system as, unlike the other examples, there is no transition beam or slab to connect the existing structure to the piles. For the Pali Radice system each proposal would need to be assessed on its merits and a decision taken on whether sufficient bond can be achieved between the new pile and the old structure.

For the other cases it is important to maintain adequate load transfer from the pile into the slab or beam system. The steel reinforcement protruding from the pile should achieve a full bond into the reinforced concrete support system and where appropriate, a check should be made that the pile does not tend to punch through the slab through excessive load concentration.

83.2.5 Beam and pad underpinning

For subsidence-related underpinning the sacrificial stool system can be used to create a reinforced concrete beam below or within the depth of an existing foundation as described above. Isolated mass concrete pads are then formed at regular centres, say, 3.0 m. This form of underpinning is often more economic to build than the equivalent mass concrete system. This is especially so if, due to the prevailing ground conditions, the bases are anticipated to be formed at considerable depth.

Reference should be made to the comments in section 83.2.1 above in relation to the bearing capacity of the soil below the isolated underpinning bases.

83.2.6 Jacked underpinning

Where a building has settled differentially it is often possible to jack the structure back to eradicate or at least reduce the difference in levels. These are usually independent solutions appropriate to each case but as a general rule the principle involves forming a solid base from which to jack and to provide a 'cradle' under the building in order to contain it during the jacking operation. The cradle will then be used to support the building once the jacking is complete.

In the simple case where an extension has rotated downwards and away from the main building, it is important to provide a horizontal tie at foundation level. This is to allow the extension to rotate upwards during jacking but to prevent it from moving outwards at the base.

In all cases the jacking will only be effective if the existing cracks in the fabric of the structure are clear and free of previous repair or debris. If the building is allowed to 'pinch' on any material within the cracks then it cannot rotate back to its original position.

83.3 Factors influencing the choice of underpinning type

83.3.1 Reason for carrying out the underpinning works

Where the underpinning is not scheduled to be used to retain soil, for example, to counter the effect of subsidence settlement, then the choice of underpinning type can be determined by consideration of the most economical engineering solution given any access restrictions and taking in account the prevailing ground conditions. Where the underpinning is proposed to be used to retain soil then mass concrete or reinforced concrete bases become applicable. The use of mass concrete or reinforced concrete will then depend on such aspects of a project as the requirements for space creation, and the design conditions relating to the ability for the wall to act as an independent gravity retaining wall without an undue amount of propping.

83.3.2 Study of the site investigation

Save for the most basic of requirements an appropriate site investigation is essential in order to make an informed choice regarding the type and depth of underpinning. For example, clay soils may have suffered the effect of shrinkage through tree root activity. In certain circumstances, it is more economical to provide piled underpinning as an alternative to traditional underpinning. If the site investigation shows the soil to be desiccated to a depth in excess of 2.5–3.0 m, depending on the prevailing circumstances, it may well be more economical to consider a piled scheme.

If a clay soil is encountered then hand- or machine-excavated underpinning is relatively easily installed as the shoring can be safely placed at intervals leaving sections of exposed ground between the props. On the other hand, if loose uniformly graded sand is encountered then hand or machine excavation becomes much more difficult and therefore expensive as it is likely that a form of close boarded shoring will be required to make the excavation safe. For this form of shoring horizontal and vertical trench sheeting is often used to fully retain the soil.

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The site investigation data should be used as a means to establish the founding depth of the proposed underpinning. In certain cases, for example, dense gravel over soft clay, the ground upon which the existing foundations are formed can have a higher bearing capacity than that of the ground below. This will influence the depth and width of mass or reinforced underpinning.

For ground conditions such as peat or where there is no stable ground to be encountered at reliable depth, a piled solution should be considered. Where the water table is above the proposed founding level for the underpinning, a piled solution should be considered wherever possible.

When commissioning a site investigation, prior knowledge should be gained of the type of ground likely to be encountered. This will enable an initial assessment to be made of the type of underpinning likely to be required. The investigation can then be commissioned to provide adequate information to enable a detailed and informed design to be carried out. A 5.0 m deep borehole may be sufficient for excavated underpinning but will be totally uninformative with respect to a piled solution. Standard penetration testing (SPT) should be undertaken in granular soils with U100 samples wherever possible in clay soils. The level and inflow rate of any water ingress should be noted and recorded. The final level of the water table should be noted and a piezometer should be installed in order to monitor any fluctuations in the level with time.

Soil suction tests should be undertaken when investigating the requirement for underpinning in the case of subsidence settlement. Specialist advice should be sought from an appropriate ground investigation company to ascertain the type and degree of testing to be carried out.

83.3.3 Settlement potential

In general a piled underpinning solution will perform with a lesser degree of potential settlement than an excavated solution. If, however, due to the required function of the underpinning, an excavated technique must be used, then the likely degree of settlement is an important factor for consideration.

It is possible to provide hydraulic jacking to alternate bases in order to pre-consolidate the soil as the bases are installed. In some cases, small diameter steel-cased piles can be placed in the excavated base prior to pouring concrete. This will create a load-sharing mechanism in order to minimise the effect of settlement.

If, for example, a dense granular soil underlies soft clays or loose granular materials, it may be effective to deepen the underpinning over and above the required depth in order to found on the material having a lesser potential for settlement.

In order to avoid the effect of multi-stage settlement at each phase of underpinning, single-stage underpinning should be considered wherever possible.

83.3.4 Gravity retaining wall design

For basement design the underpinning should be designed as a gravity retaining wall in terms of overall performance. A second analysis should then be undertaken to check its performance as a masonry wall or alternatively as a reinforced wall in bending and shear.

83.3.5 Party wall matters

If at all possible it is advisable to open discussion with party wall engineers at an early stage. This will minimise the risk of abortive design work and delay to the progress of the project as a whole. Reinforced concrete underpinning proposed along the party wall line may be considered as a ‘special foundation’ and can cause difficulty in achieving an award. The main principle, however, is not to encroach into the space below the plan area of the adjacent property and the choice of the underpinning system must reflect this requirement.

83.4 Bearing capacity of underpinning and adjacent footings

83.4.1 Bearing capacity theory

The shearing resistance of a foundation may be divided into three parts for ease of explanation (see **Figure 83.3**):

(a) frictional resistance resulting from the surcharge due to the depth of overburden;

(b) frictional resistance resulting from the weight of soil within the rupture zone BCD;

(c) cohesive resistance along the line ABC.

83.4.2 Excavation adjacent to an existing footing

Reference to **Figure 83.3** shows the effect of trench excavation to the level of an existing footing. As the depth of overburden has an influence on the bearing capacity of the soil below the existing foundation its removal can only serve to reduce the overall bearing capacity. This is especially so for granular soil as there is also no appreciable cohesive resistance along the rupture zone ABC shown in the diagram and the ground is much more reliant on the presence of an overburden to achieve adequate bearing capacity.

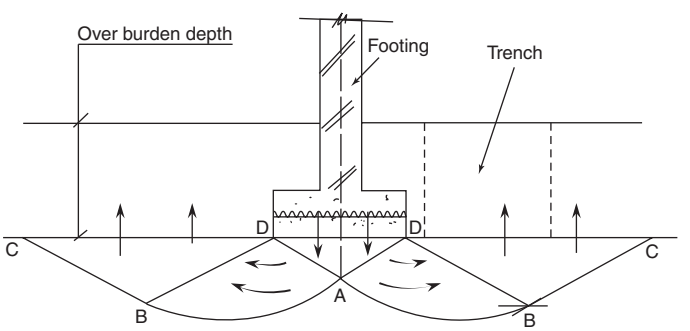


Figure 83.3 Failure of a foundation through inadequate shear resistance. Scott (1980)

83.4.3 Bearing capacity of the proposed underpinning

The mode of failure described above is a catastrophic collapse of the soil beneath the foundation through inadequate sheer strength. Failures of this type are not very common but when they do occur they lead to large distortions of the structure above. A more common mode of failure is through excessive settlement of the foundation.

For sands and gravels, the settlement takes place almost immediately and the cycle is usually complete by the end of the construction programme. This is because the settlement is caused through *compaction* of the soil through the increased load applied by the new foundation.

For clay soils the settlement is caused through *consolidation* rather than compaction. In this case, settlement occurs in diminishing amounts with time and can continue through the life of the structure. The settlement comes by consolidation of the particles in the tight soil matrix. This process is delayed, however, by gradual dissipation of the excess pore water pressure caused through loading the soil with the weight of the new foundation.

When underpinning in a clay soil it is therefore important to consider that the soil upon which the new foundation sits will settle with time by the process described above. It is advisable to leave a newly underpinned structure as long as possible to allow the greater part of the settlement process to take place before applying the final decorative finishes.

Any new foundation should have a calculable capacity and appropriate factor of safety. Wherever possible a site investigation should be carried out. Advice can be sought through the site investigation company on the most appropriate form of foundation to use along with information on allowable bearing capacity. It is of course necessary to brief the site investigation company beforehand on the information required for a particular project so that they may carry out the appropriate testing with samples gained at the time of the investigation.

83.5 Shoring

83.5.1 Types of shoring

Timber, steel or pre-cast concrete are often used. For a typical underpinning base the shoring supporting the face behind the wall will become sacrificial. For this reason, it is wise to use steel or concrete to support this face even if timber is used elsewhere.

83.5.2 Design of shoring

Safety for installation is a key factor in the design of shoring. Whilst pre-cast concrete planks or lintels are relatively low in cost and readily available, they are heavy to lift and are prone to progressive collapse when placed horizontally over the full height of an underpinning face. Steel trench sheets are equally robust but do not carry with them the same degree of safety concern.

For some soils, intermittent horizontal propping may be sufficient whilst for other soils, for example loose sand, full sheeting may be required. The size (and type) of props will be designed in accordance with the calculated soil pressure due to the overburden and any surcharging which may need to be taken into account. **Figure 83.4** shows an example of underpinning carried out in loose sands and gravels.

83.5.3 Minimising the risk of settlement

Any shoring technique will result in varying degrees of voiding behind the props due to lack of fit and local decay of the soil face prior to concreting. Consideration should be given to dry-packing behind the face of the shoring. For basement construction, the shoring to each underpinning base can be left in place prior to excavation of the entire basement. Depending



(a) Excavation



(b) Blinding



(c) Steel fixing



(d) Cast the base



(e) First wall pour



(f) Second wall pour

Figure 83.4 Illustration of reinforced concrete underpinning in sands and gravel

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on the soil type retaining the shoring can be more effective in reducing the risk of settlement than backfilling each base, as the excavated material cannot always be readily compacted.

83.6 Underpinning in sands and gravel

Figure 83.4 shows the construction techniques used to create reinforced underpinning bases in a single stage to a depth of 6.0 m below ground level.

83.7 Dealing with groundwater

83.7.1 Pre-grouting

Figure 83.5 shows a technique for consolidating a granular soil prior to underpinning below the water table. The method relies on the interstices between the soil particles being capable of being filled with an injected material. In many cases, the interstices are too fine to permit cementitious grout to be used and therefore a chemical is required. Often a gel is used which is biodegradable so that the ground below adjacent properties is not affected in the long term.

This form of prevention for water ingress during underpinning is by no means guaranteed. The majority of injected material is excavated when carrying out the underpinning and therefore the most densely compacted areas are removed. The diagrams showing the angle of drilling indicate that the effectiveness of injection decreases with the distance from the drill location. This method of water prevention has been shown to be effective in certain locations but careful advice from specialist contractors should be sought if considering this method for a particular application.

83.7.2 Well pointing

The system typically comprises a series of perforated plastic tubes placed into boreholes and surrounded with granular soil with filter membrane. As water drains into the tubing it is pumped out thus lowering the water table locally. Depending on the soil type and the flow characteristics through it, the water table can be locally drawn down in order to permit the underpinning to be carried out in relatively dry conditions.

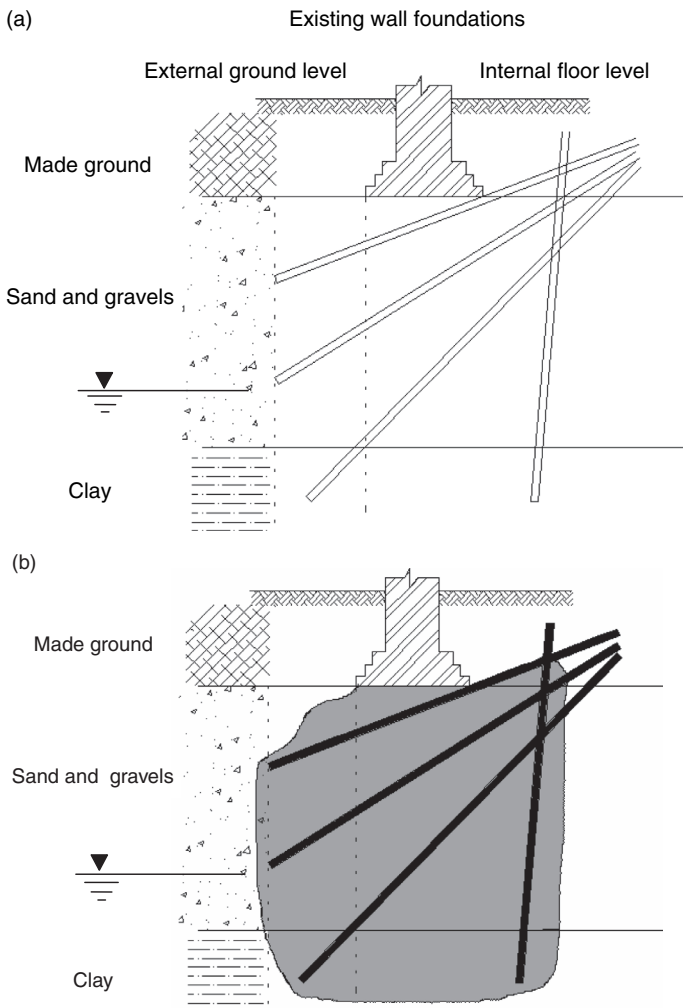


Figure 83.5 Illustration of pre-grouting technique

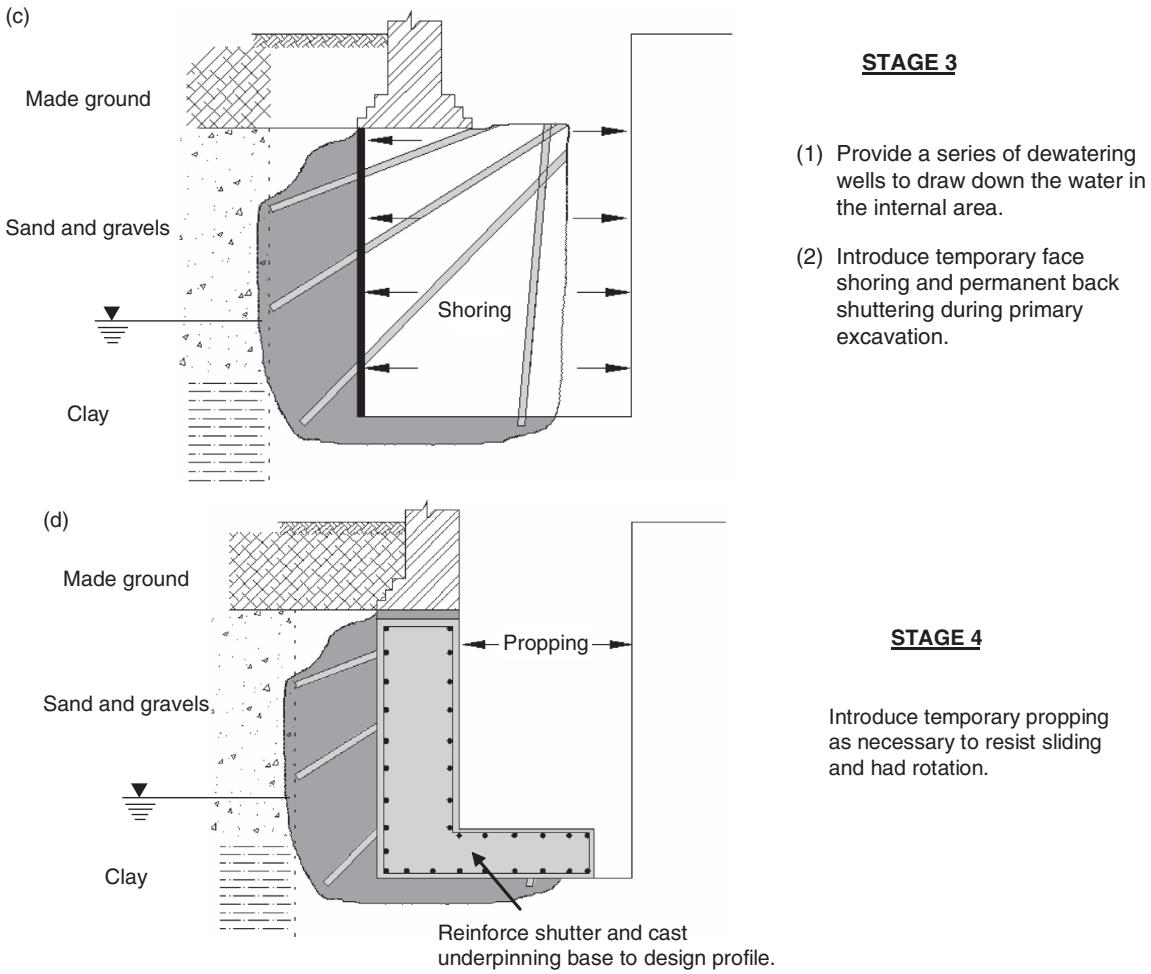


Figure 83.5 Continued

By changing the soil characteristics, lowering the water table can create a much increased possibility for settlement of the adjacent foundations. The movement of water through the ground can also cause settlement through migration of fines.

In the right circumstances however, local dewatering through well pointing can be a very effective way of enabling underpinning works to be carried out below the water table.

Consideration should be given to where the water is to be discharged. If it is to be discharged into the drains then permission should be sought from the local authority. Water which is discharged locally and allowed to soak into the ground will eventually find its way back into the system.

83.7.3 Local pumping

Depending on the nature of water inflow, local pumping can be used to enable an underpinning base to be formed in wet conditions. The use of local pumping should, however, be restricted to conditions which are safe in terms of the personnel involved in carrying out the work and in terms of potential settlement to the surrounding structures. If seepage is allowed to continue unabated during the progress of the works to install the base then fines can be drawn into the excavation. This will compromise both personnel safety and will increase the possibility of settlement occurring in the adjacent structures.

If the underpinning is required to penetrate the groundwater table then depending on the depth of penetration and the prevailing ground conditions local pumping can be effectively used. The point set out in section 83.7.2 above with regard to discharge of the pumped water is relevant here. For local pumping below the water table, it is usually difficult to control the amount of fines being pumped away along with the removal of water. The potential for settlement as a result of the pumping operation is correspondingly increased.

For both well pointing and local pumping the removal of fines within the soil will cause settlement to adjacent structures. The use of filters on the pumps is essential to minimise the possibility of this occurring.

83.8 Underpinning in relation to subsidence settlement

83.8.1 Choice of underpinning scheme

If the underpinning is not generally to be exposed or used as a soil retaining structure, the choice of underpinning is governed by the requirement to satisfy the design brief whilst achieving the best economy of construction. Whilst a performance specification or design brief can be developed by a consulting practice or surveyor, the actual choice of underpinning scheme may be left to the contractor's experience and specialist skills to conceive the best system in view of the prevailing ground conditions, access situation, safety aspects and overall economy.

83.8.2 ASUC

The Association of Specialist Underpinning Contractors (ASUC) is an organisation which has influence on activities related to subsidence settlement and other aspects of underpinning. Information concerning its initiatives and documentation can be found on the website www.asuc.org.uk.

83.8.3 Other subsidence-related stabilisation methods

In conjunction with underpinning works related to subsidence settlement, a programme of crack repair and refurbishment is usually undertaken. Crack repair can take the form of resin injection, 'brick stitching' by cutting out and replacement of the cracked bricks or insertion of steel reinforcing bars into the horizontal joints.

When the cause of the subsidence settlement is clearly defined by the presence of a tree or trees then, depending on the prevailing circumstances, these can be removed or pollarded. This can have the result of halting or even reversing the effects of subsidence settlement. A programme for tree removal/pollarding with subsequent monitoring is often undertaken prior to making a decision to carry out underpinning works.

If tree root activity is the cause of subsidence settlement, it is often important to implement and maintain a programme of continual tree maintenance to ensure that once the underpinning has been carried out, further settlement does not take place. Continual tree maintenance is carried out with the objective of containing the level of root activity to its current state. In this way, the shrinkable clay neither shrinks further nor recovers moisture with resultant ground heave.

83.8.4 Extent of underpinning

For subsidence-related underpinning, the question of whether to underpin at all or how much of the structure to underpin is always a difficult one. In order to find an appropriate solution it will usually be necessary to monitor the structure to establish the pattern of movement or indeed to establish whether the movement is still active and ongoing.

Study of the cracking pattern alone can be deceptive as it only indicates the degree of differential settlement. The whole structure could be settling but at different rates. Partially underpinning the worst area will serve to 'lock' this part of the building but then cracking may continue as the rest of the building settles further.

If, for economic reasons or for any other reason, it is not possible to underpin the whole of the property then appropriate measures should be taken to ensure that an adequate transition is made between the underpinned and non-underpinned structure. This will of course depend on the type of underpinning which has been chosen for the main structure. Typically this would involve stepped underpinning reducing the depth of the bases upwards towards the original foundation. To work effectively, however, each transition base should be separated from the adjacent one with polystyrene strips or similar in order to permit independent movement to take place. Otherwise the transition bases will tend to act as a corbel and therefore will not permit independent movement.

83.8.5 Contractual matters in relation to subsidence settlement

A contract in relation to subsidence settlement will probably contain clauses concerning the following points:

(i) A statement of the depth of underpinning/piling allowed and the practical/financial arrangements should the depth vary upwards or downwards.

(ii) For partial underpinning the extent of underpinning contained within the contract, a statement regarding any transition sections and arrangements concerning responsibility for the sections of the building not currently scheduled to be underpinned.

(iii) The extent of the design brief. Is the specialist contractor expected to design a total solution regarding the type and extent of underpinning required taking into account the damage which can be observed? Will the contractor take responsibility for variation of ground conditions? It will be necessary to accurately define the responsibility for both these questions.

(iv) An appropriate form of guarantee/warranty will usually be required. ASUC provide an insurance-backed guarantee, more details of which can be found on their website.

83.9 Safety aspects of underpinning

Underpinning works are considered by the Health and Safety Executive (HSE) as high risk and there is a requirement to produce method statements and risk assessments. Relatively recently the rule stating that excavations over 1.2 m deep required shoring was changed to include all excavations based on risk assessment.

The HSE publication HSG150 2006 contains a chapter on Groundworks. The publication warns of the danger of excavation in unsupported trenches and states the following:

341 Before digging any trenches, pits, tunnels, or other excavations, decide what temporary support will be required and plan the precautions that are going to be taken against:

- collapse of sides;
- people and vehicles falling into the excavations;
- materials falling onto people working in the excavation;
- undermining nearby structures;
- underground and overhead services; and
- the inflow of ground and serviced water.

Sections 338 to 381 provide information on safe practice with respect to the above subject headings.

Save for relatively shallow underpinning, the Confined Spaces Regulations (1997) will usually apply. As there is a risk of personnel falling into the excavations 'working at height' regulations apply especially with regard to the provision of handrails around all excavations irrespective of depth.

Under the current Construction (Design and Management) Regulations (CDM regulations), it is required that clients provide accurate information to enable the structural stability of the building and those adjacent to be assessed before work commences. A Health and Safety plan should be produced.

83.10 Financial aspects

As a general statement, deep hand-excavated underpinning is expensive and, where appropriate, consideration should be given to a piled scheme. Piles can be considered as a total solution in the case of Pali Radice or more often in conjunction with a reinforced concrete beam/slab arrangement.

Reinforced underpinning is of course relatively more expensive to install than a mass concrete equivalent but other considerations should be taken into account such as temporary works support and overall space creation. In general reinforced underpinning will occupy less space than the equivalent mass concrete system with reinforced lining walls.

In consideration of space creation construction, tolerance should be allowed in the calculation. Due to the nature of construction, it is difficult to achieve tight construction tolerance both in terms of verticality and precise formation level.

83.11 Conclusion

The process for finding the most appropriate form of underpinning to suit a particular set of circumstances is involved and can become complex. At all stages of the process specialist advice should be sought.

It is particularly important to produce a site investigation report which provides information in order to enable an informed and rational decision to be made on the correct type of underpinning to suit a particular set of circumstances. Advice from both specialist underpinning companies and site investigation companies should again be taken.

83.12 References

Health and Safety Executive (HSE) (1996) *The Construction (Health, Safety and Welfare) Regulations 1996*. London: Stationery Office.

Health and Safety Executive (1997) *The Confined Spaces Regulations 1997*. London: Stationery Office.

Health and Safety Executive (2005). *The Work at Height Regulations 2005*. London: Stationery Office.

Health and Safety Executive (2006). *Health and Safety in Construction*, (3rd edition). HSG150. London, UK: HSE Books. [Available at www.hse.gov.uk/pubns/books/hsg150.htm]

Health and Safety Executive (2007). *The Construction (Design and Management) Regulations 2007*. London: Stationery Office.

83.12.1 Further reading

Scott, C. R. (1980). *An Introduction to Soil Mechanics and Foundations*. Abingdon: Spon.

83.12.2 Useful websites

Association of Specialist Underpinning Contractors (ASUC); www.asuc.org.uk/

Health and Safety Executive (HSE); www.hse.gov.uk/

Oxford Hydrotechnics; www.h2ox.net

Geostructural Solutions; www.geostructuralsolutions.co.uk

It is recommended this chapter is read in conjunction with

- Chapter 26 *Building response to ground movements*
- Section 5 *Design of foundations*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 84
Ground improvement

Colin J. Serridge Balfour Beatty Ground Engineering Ltd, Manchester, UK
Barry Slocombe Keller Limited, Coventry, UK

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Ground improvement techniques can often be used as an economical alternative to piled and deep foundation solutions for a wide range of made ground, fill materials and natural soils to support houses, offices, industrial units, tanks, road embankments and similar low-rise developments. Techniques used include *in situ* compaction of clean sands using depth vibrators, adding stone or concrete during compaction to form vibro stone, or vibro concrete columns and dynamic compaction.

As the soil conditions have a large influence on the result, ground improvement techniques require an appropriate level of site and ground investigation to permit satisfactory geotechnical characterisation of the soil profile. Ground improvement also requires an appropriate level of understanding of where the differing techniques work and how to ensure correct and appropriate application. Quality control and monitoring procedures during execution of ground improvement techniques are essential to ensure successful implementation and performance.

Ground improvement techniques permit the adoption of relatively simple shallow foundations and groundbearing warehouse floor slabs. They can also provide significant sustainability advantages in comparison to more traditional deep foundation methods.

84.1 Introduction

Ground improvement involves the enhancement of ground properties, principally by strengthening or stiffening processes and compaction or densification mechanisms, to achieve a specific geotechnical performance.

Of the various ground improvement methods available, vibro techniques employing vibroflots (also referred to as vibrating pokers or depth vibrators) and dynamic methods incorporating the dropping of heavy steel tampers or weights on predetermined grid patterns, are the most commonly used. These form the main focus of discussion in this chapter.

It is important that ground engineering practitioners involved with the design, procurement and supervision of ground improvement techniques have an appropriate level of understanding of the various treatment methods being considered, the likely benefits, including selection of the most appropriate technique and any drawbacks or limitations of the technique selected for the prevailing combination of the ground conditions and applications being considered.

Choice of method will be influenced by the geotechnical requirements and engineering performance objectives of the ground improvement. Appropriate site supervision, quality control and monitoring procedures during execution of ground improvement techniques are essential to ensure successful implementation, as is an appropriate level of post-treatment testing to ensure satisfactory performance will be achieved.

Design principles for these techniques are covered elsewhere (Chapter 59 *Design principles for ground improvement*). Further guidance on these and a wider range of ground improvement techniques can be found in the referenced CIRIA Guides and Building Research Establishment (BRE) Specifications in this chapter. The ground improvement techniques of grouting and soil mixing are covered in Chapter 90 *Geotechnical grouting and soil mixing*.

Where any doubts or uncertainties exist specialist ground improvement contractors should always be consulted. Such advice is normally free of charge, although advance trials may sometimes be recommended.

84.2 Vibro techniques (vibrocompaction and vibro stone columns)

84.2.1 Introduction and history

The modern origins of vibro techniques were conceived in Germany in the mid-1930s. Vibrocompaction was the first technique to be developed, with its first application being for the *in situ* densification of loose sands for a government building in Berlin in 1937. Further development of the technique proceeded in parallel in Germany and the United States in the 1940s. The development of the vibro stone column method in the 1950s permitted the application of vibro techniques to a much wider range of soil types, most notably finer-grained soils (cohesive and mixed cohesive and granular soils), thus increasing the range of soils which could be treated by vibro techniques. Vibro stone column techniques were introduced into Great Britain and France in the late 1950s and have been used extensively worldwide.

84.2.2 Essential features of the vibroflot/vibrator equipment

The principal piece of equipment used to carry out vibro ground improvement techniques (whether for *in situ* densification or

construction of stone columns) is the vibroflot (also referred to as a vibrating poker or depth vibrator), which is either suspended from a crawler crane or mounted on a leader attached to a base machine, dependent upon the specific application. The essential features of the vibroflot are presented in **Figure 84.1**.

The vibroflot equipment comprises three main components – the vibrator head, the isolator and the extension tubes. The main component used to achieve compaction is the vibrator. Vibrations are produced close to the base or tip of the vibrator. These are induced by a series of rotating internal eccentric weights mounted on a shaft driven by an hydraulically or electrically powered motor located in the upper part of the vibrator casing. The rotating eccentric weights within the vibrator impart to the body of the vibrator a gyratory motion in a horizontal plane. Follower or extension tubes of similar or smaller diameter to the vibrator unit are attached to permit treatment of soils to varying depths. An elastic coupling isolates the vibration from the extension tubes and therefore prevents vibration travelling up the extension tubes to the supporting crane or base machines. Stabilising fins maintain stability of the vibroflot in the bore, by preventing rotation influenced by torque when the vibroflot is worked hard in the ground, which would otherwise lead to twisting or snagging of any external hydraulic or electric cables.

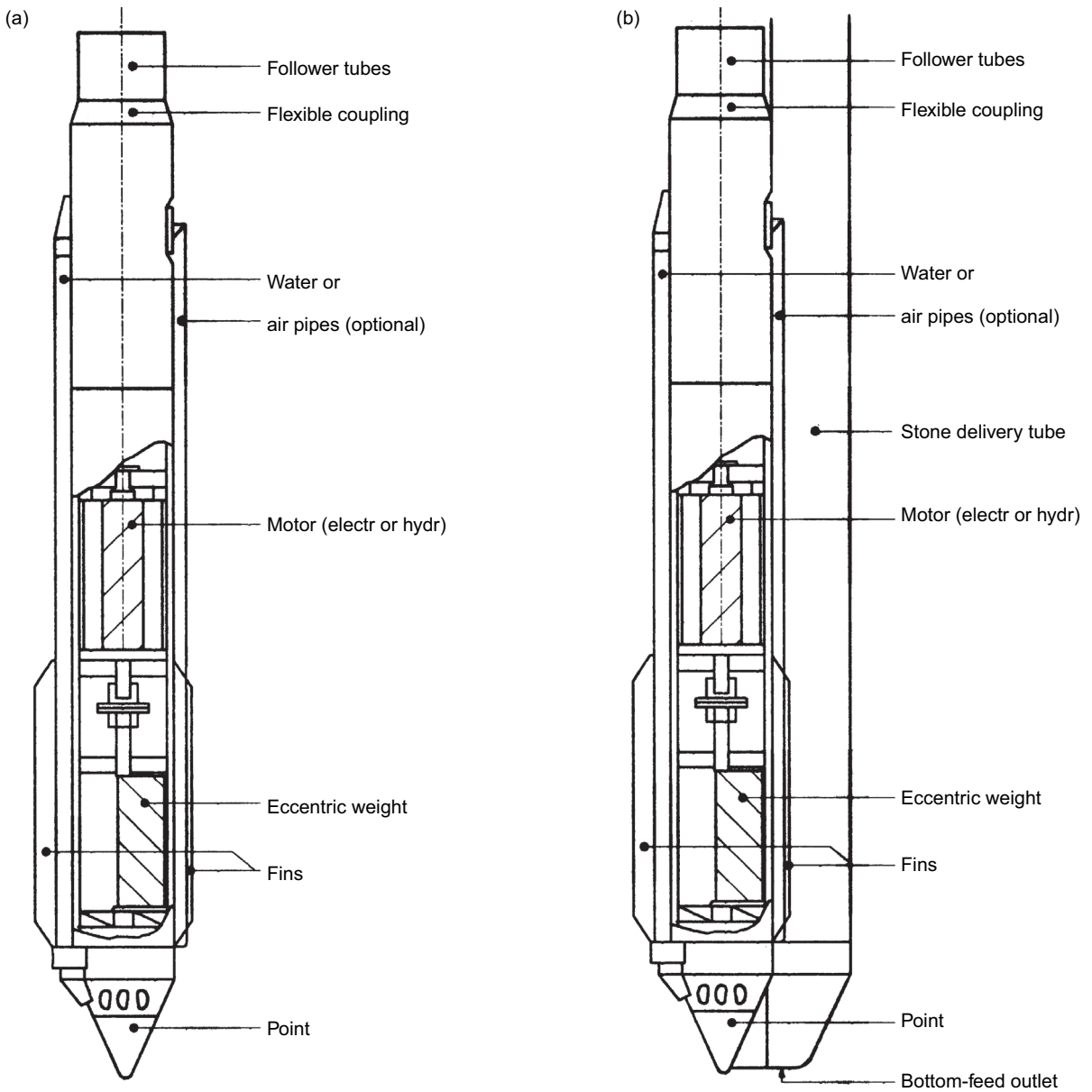


Figure 84.1 (a) Top-feed and (b) bottom-feed vibroflot (vibrating poker) units

Reproduced from Slocombe *et al.* (2000); all rights reserved

Whilst the basic components of the equipment have changed very little over the years, there have been significant developments in the reliability (with extended lifespans and reduced maintenance) and power ratings of the equipment with the objective of achieving greater efficiency in densification and stone column production.

84.2.3 Applicability of vibro techniques

The vibroflot may be used for both *in situ* compaction of cohesionless soils and for forming stone columns in fine-grained soils, as shown in **Figure 84.2**. The actual mechanism of improvement is a function of whether the soils are essentially granular (coarse-grained) or cohesive (fine-grained).

It is generally recognised that *in situ* vibro compaction is appropriate for granular soils with a total fines (particles finer than 0.06 mm) content of up to 15% of which the clay and fine silt content (particles smaller than 5 μm) should be less than 2% (Slocombe *et al.*, 2000).

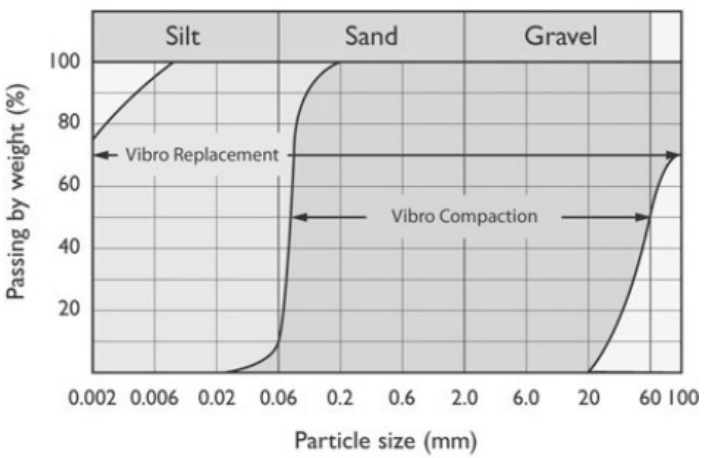


Figure 84.2 Range of soils treatable by vibrocompaction and vibro stone column techniques

84.2.3.1 Granular (coarse-grained) soils

The principle of vibro techniques in granular soils is based upon particles of coarse-grained (non-cohesive) soil being rearranged into a denser state by means of dominantly horizontal vibration from the vibroflot (vibrating poker). The resultant reduced void ratio and compressibility and corresponding increased angle of shearing resistance then permit the adoption of higher imposed design loadings at lesser settlement and increased seismic resistance.

84.2.3.2 Cohesive (fine-grained) soils

With fine-grained soils the cohesion between the soil particles dampens the vibrations and prevents rearrangement and compaction occurring. Improvement is achieved by 'reinforcing' the soil with relatively rigid stone columns. During the stone column construction the introduced coarse granular column material is pressed radially into the soil so that it is displaced beyond the diameter of the vibrator. The column of compacted dense granular material forms, together with the surrounding soil, a composite stone column – soil mass, with enhanced shear strength and bearing capacity, together with a corresponding reduction in settlements (attributed to the 'stiffening' effect of the stone columns). Stone columns in fine-grained soils also assist in the dissipation of excess pore water pressure under applied load or surcharge, which accelerates the consolidation process, together with providing enhanced slope stability parameters.

84.2.4 Vibrocompaction

For the vibrocompaction technique (**Figure 84.3**; see also **Figure 84.7(a)**) the vibroflot unit is positioned over the required compaction point, high pressure water jets in the vibrator nose cone are activated and the vibroflot is slowly lowered into the ground assisted by a combination of the weight of the vibroflot and the jetting action of the water jets. In the immediate

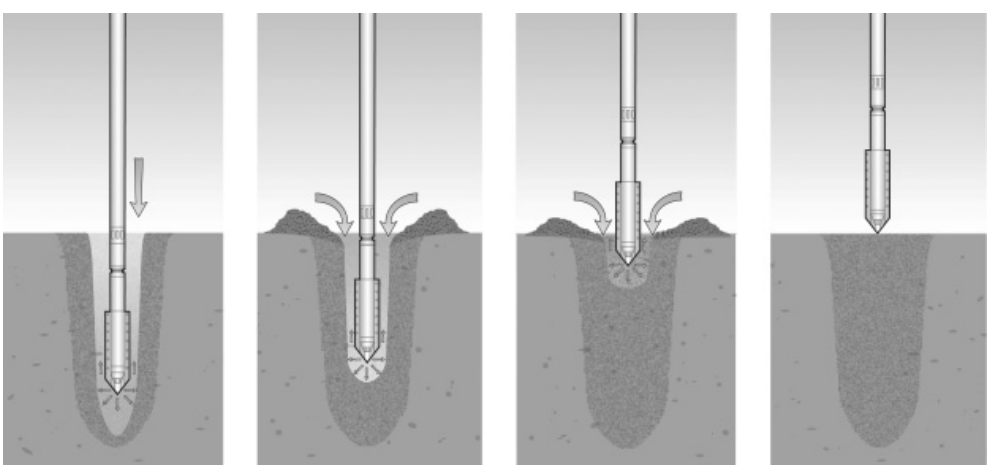


Figure 84.3 *In situ* vibrocompaction (sequence)

vicinity of the vibrator the saturated soil liquefies locally and temporarily under the influence of the vibrations. The vibroflot proceeds to sink rapidly into the soil under its own weight. When the required depth of penetration is reached, the vibrator is halted and the water supply reduced (or sometimes stopped). Under the action of the vibrations, the granular soil particles surrounding the vibrator are rearranged into a denser state. The effect of this compaction becomes visible at the ground surface in the form of a cone-shaped depression which is continually infilled with sand during the vibrocompaction process. When the required degree of compaction has been achieved (normally by monitoring the power consumption) at the depth of the vibrator, it is raised a short distance (typically 300–500 mm) from the base of the bore and the cycle repeated. Insertion of the vibroflot on a grid pattern will enable overlapping of the compacted cylinders and produce a homogeneous mass of compacted soil. Preliminary trials with pre- and post-treatment penetration testing may be employed to optimise the compaction spacings.

Sands of single-size grading or with a higher fines content are more difficult to compact than well-graded sands, as are sands that occur in relatively thin layers trapped between layers of clay and which require particular attention. Carbonate sands are susceptible to particle crushing and typically require closer grid spacings than silica sands.

84.2.5 Vibro stone column terminology and techniques

Terminology for vibro stone column techniques has not always been applied in a consistent manner and some attempt at addressing this can be found in BRE Document BR 391, *Specifying Vibro Stone Columns* (2000). This specification employs terms that reflect the fundamental principles of top-feed or bottom-feed to describe the method of stone aggregate supply, and wet or dry to describe the 'jetting' medium. This has given rise to the following terminology:

- dry top-feed technique;
- dry bottom-feed technique;
- wet top-feed technique.

The technique adopted is a function of ground conditions, bore stability and environmental constraints. Historically, literature has suggested that the dry top-feed technique should not be used in fine-grained soils with undrained shear strengths of less than 30 kN/m^2. For soils weaker than this, the dry bottom-feed technique (or historically the wet top-feed technique) has to be used. In terms of a lower-bound undrained shear strength, a value of 15 kN/m^2 has typically been used for vibro stone column applications. However, developments in vibroflot technology (together with monitoring and quality control systems) have allowed weaker soils to be treated in certain applications. Consideration also needs to be given to soil sensitivity when selecting the appropriate technique, as saturated sensitive soils can undergo significant remoulding when exposed to vibrations. For soft soils, settlement tends to be more of an issue than bearing capacity.

All three vibro stone column techniques described above use similar types of vibroflots, normally hydraulically or electrically driven, the main difference being the dry bottom-feed equipment which has a stone (aggregate) feed tube attached to the vibroflot to permit the stone aggregate to be introduced to the tip of the vibroflot without it having to be removed from the bore, thus overcoming any instability issues in weak ground or treatment to beneath the water table. The stone aggregate is typically handled by front end loaders with a side tip facility.

All the vibro stone column techniques are aimed at constructing well-compacted granular columns, typically of crushed stone or gravel aggregate in the void formed in the ground by the vibrating poker, thus improving bearing capacity and drainage while reducing total and differential settlements.

84.2.5.1 Dry top-feed technique

This technique can be used in a range of soil types, but relies on the bore remaining stable during column construction, with a general absence of groundwater within the treatment depth.

The vibroflot penetrates the soil by shearing and displacing the soil around it. Air flush via jets in the nose cone of the vibrator are used to overcome suction forces. Upon reaching the required depth the vibroflot is then completely withdrawn from the bore to facilitate introduction of granular backfill (aggregate) from the surface. Aggregate is tipped into the bore and the vibroflot then repenetrates to within a short distance of the original depth displacing the aggregate laterally and downwards, thus compacting it. The procedure is repeated in approximately 0.5 m lifts until the stone column construction is completed to the surface (**Figure 84.4**; see also **Figure 84.7(b)**). Stone column diameters of about 0.5–0.6 m are typically achieved, dependent upon soil properties.

84.2.5.2 Dry bottom-feed technique

This technique is used in weak soil profiles with a high water table where bore stability cannot be guaranteed during stone column construction with adoption of the conventional dry top-feed method.

The vibroflot penetrates the soil in the same manner as in the dry top-feed technique, the main difference being that the vibroflot has a stone tube (tremie pipe system) attached to it, allowing aggregate to be introduced to the tip of the vibroflot, without it having to be removed from the bore. Hence the vibroflot by remaining in the ground supports the potentially unstable bore. An airlock in the stone tube allows compressed air to assist discharge of aggregate. The vibroflot unit is mounted on leaders attached to a dedicated base machine which provides pulldown resistance during penetration of the vibroflot into the ground. During initial penetration the stone tube is charged with stone aggregate and when the required depth is reached the vibroflot is withdrawn a short distance from the base of the bore and a charge of stone aggregate is

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discharged (with air pressure assistance) at the tip of the vibroflot, which is subsequently compacted on repenetration of the vibroflot. This cycle is repeated (in approximately 0.5 m lifts) until a compact stone column is constructed to the surface (**Figure 84.5**; see also **Figure 84.7(c)**).

84.2.5.3 Wet top-feed technique

This technique is employed in saturated fine-grained deposits and saturated granular soils deemed too silty for *in situ* vibrocompaction. The technique uses a similar technique to that used in vibrocompaction. When the required depth is reached, the vibroflot is sometimes surged up and down to flush out the bore. The vibroflot is then held a short distance off the bottom of the bore and the water pressure reduced sufficiently to allow a nominal outflow of water (and suspended fines) at the surface such that excess hydrostatic pressure and outward seepage forces support the uncased bore sufficiently long enough to form an annular space surrounding the vibroflot and permit construction of a stone column. A charge of aggregate is then introduced, while the vibroflot is still in the bore, down the annulus against the continuing low pressure upflow of water. This is subsequently compacted by repenetration of the vibroflot into the aggregate. The vibroflot is then lifted sufficiently to allow introduction of further charges of aggregate, which are then compacted and the cycle repeated until a continuous column of compacted stone is formed to the surface (**Figure 84.6**; see also **Figure 84.7(d)**).

The wet top-feed technique requires consideration of provision of water supply, drainage ditches, settlement lagoons and contamination of the site surface with fines as well as disposal of effluent.

Whilst the dry bottom-feed technique has largely superseded the wet top-feed technique on environmental grounds (there is a high water demand and effluent is generated), there are still certain ground types (where contamination is not an issue) where the wet technique may be more suitable and often necessary to achieve satisfactory results. In coastal or estuarine environments where soft clays and alluvial soils persist, the wet technique may be preferred, for example for bulk storage tanks or stockpiles of raw materials. The difference in construction

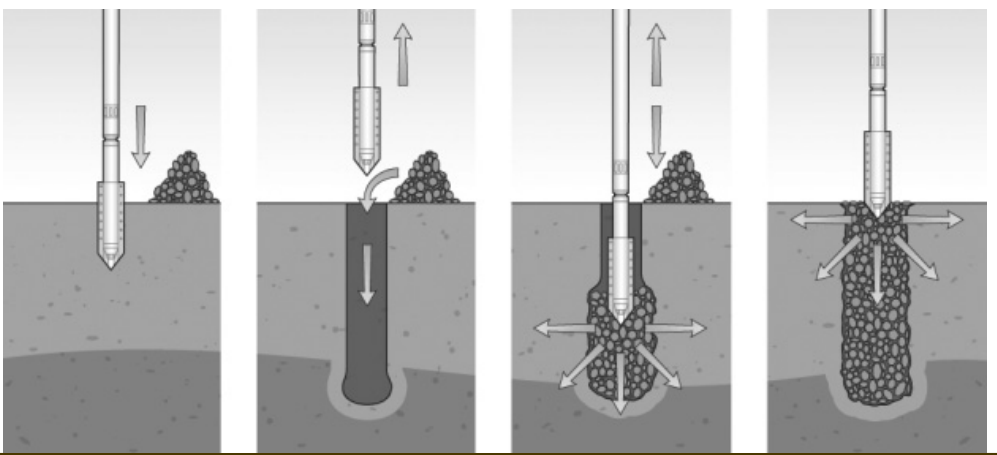


Figure 84.4 Installation sequence for dry top-feed vibro stone columns

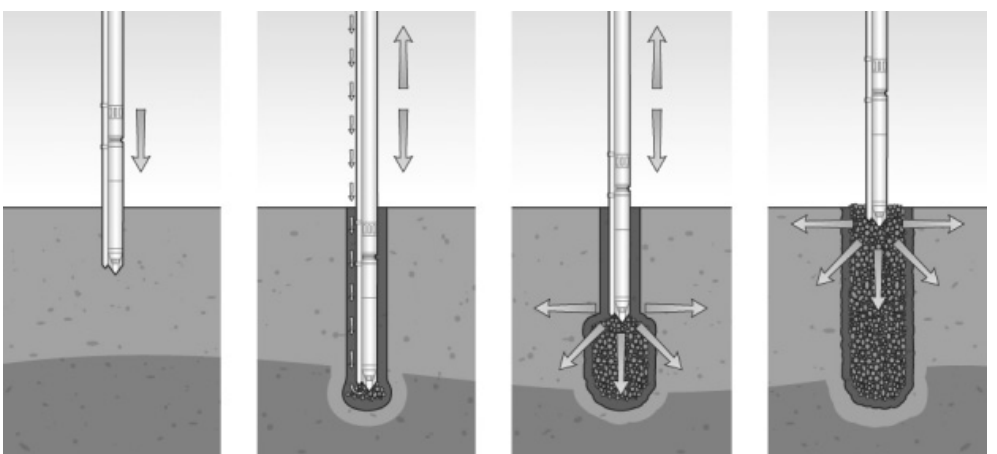


Figure 84.5 Installation sequence for dry bottom-feed vibro stone columns

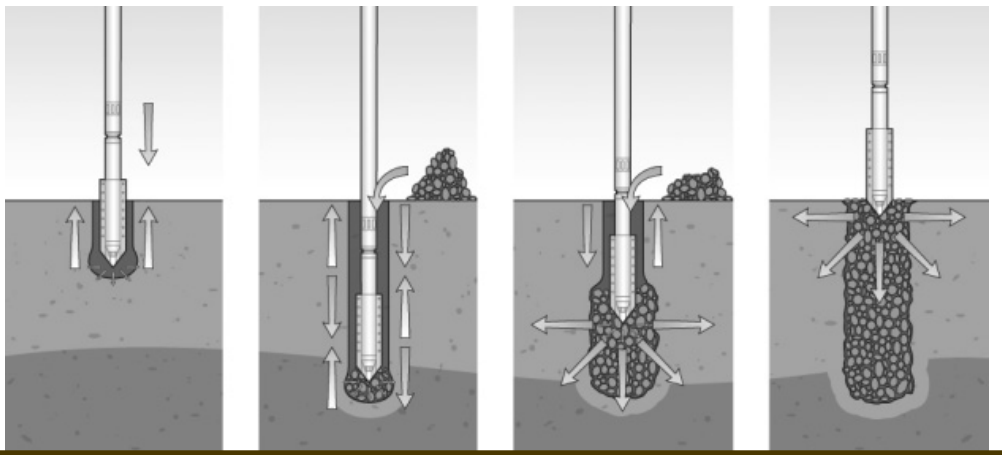


Figure 84.6 Installation sequence for wet top-feed vibro stone columns

technique between the wet method and dry methods typically results in the diameter of the stone column formed by the dry method being smaller than that formed by the wet method.

Vibro techniques are increasingly being used in near-shore marine applications, working off barges or pontoons, sometimes using larger cranes reaching out from existing quays.

84.2.6 Soil geotechnical properties and characterisation

Because of the large influence of the soil conditions on performance, ground improvement using vibro techniques requires a more extensive site and ground investigation programme compared with more conventional deep foundations solutions (Serridge, 2008). Site investigations are often designed to obtain soil parameters for deep foundation pile design. Ground improvement schemes then often require greater detail and laboratory testing on the near-surface soils than is provided. The most common types of ground-related problems encountered concern soil strata boundaries, i.e. geometry not as anticipated, and the geotechnical properties and characterisation of the soil profile. Site and ground investigation requirements for vibro techniques are dealt with elsewhere, e.g. BRE Report No. BR 391 *Specifying Vibro Stone Columns* (2000). In addition to satisfactory site geo-characterisation, important steps in achieving successful ground improvement implementation and performance include: an appropriate design and review process; consideration of the interfacing with other ground improvement techniques (or deep foundation techniques); preliminary trials (where applicable); monitoring; quality control and testing. Management of geotechnical risk is discussed by Clayton (2001).

84.2.7 Vibro stone column design

Within the UK design is typically based on Hughes and Withers (1974) for determination of stone column length and load-carrying capacity of an individual stone column. The guidance of Baumann and Bauer (1974) is frequently adopted for analysis of stress distribution between column and soil. Foundation settlement (without ground improvement) is first estimated using available geotechnical parameters and then applying appropriate settlement reduction factors (within the treated depth), according to Priebe (1995), to allow for the 'reinforcing' effect of the stone columns.

Although primarily applicable to embankment loadings over soft fine-grained/organic soils, it is important that secondary consolidation is considered as in some instances it may contribute to a significant part of the total settlement.

Some vibro design principles are provided in Chapter 59 *Design principles for ground improvement*.

84.2.8 Specifications

Within the UK the main specification documents and guidance notes covering vibro stone column techniques include:

- Institution of Civil Engineers (ICE) *Specification for Ground Treatment* (1987).
- Building Research Establishment (BRE) BR 391 *Specifying Vibro Stone Columns* (2000).
- Charles and Watts (2002) *Treated Ground – Engineering Properties and Performance*, CIRIA Report C572.
- BS EN 14731 *Ground Treatment by Deep Compaction* (BSI, 2005).
- NHBC Standards, Chapter 4.6 *Vibratory Ground Improvement Techniques* (2011).

84.2.9 Stone aggregate requirements

All the above-mentioned specifications convey a similar message with regard to stone aggregate requirements for vibro stone columns – that aggregate used in stone column construction should be sufficiently hard (i.e. must withstand the vibratory impact of the vibroflot); inert; free draining (typically with less than 5% fines); and of appropriate grading, shape and angularity to form dense granular columns with high angles of internal friction. Acceptable particle shapes for stone column material include rounded, angular and irregular. Unacceptable materials would be flaky and elongated.

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(a) (b)

(c) (d)

Figure 84.7 (a) Vibrocompaction technique. (b) Dry top-feed technique. (c) Dry bottom-feed technique. (d) Wet top-feed technique
Courtesy of Keller (a, c) and Balfour Beatty Ground Engineering Ltd (b)

Historically a maximum Aggregate Crushing Value – ACV (BS 812 (BSI, 1990)) of 30 or a minimum Ten per cent Fines Value – TFV (BS 812 (BSI, 1990)) of between 50 and 100 kN (saturated) has been deemed appropriate for vibro stone column aggregate. For the more powerful and bottom-feed vibroflot units a minimum TFV of around 120 kN is typically required. However, with the advent of the new European Standards, the above test methods are no longer recognised and have been replaced by the Los Angeles Abrasion Value – LAA (BS EN 1097 (BSI, 1998)). Whilst no recognised correlation exists between ACV, TFV and LAA, an LAA value of between 30 and 40 is recognised within the industry as

Installation technique	Typical aggregate grading (mm)
Dry top-feed process	40–75
Wet top-feed process	25–75
Dry bottom-feed process	8–50

Table 84.1 Stone column aggregate grading for different vibro techniques
Data taken from BS EN14731:2005

being acceptable for use in vibro stone columns, dependent upon specific applications. Typical grading requirements for stone column aggregate as outlined in BS EN14731 *Ground Treatment by Deep Compaction* (BSI, 2005) are summarised in **Table 84.1**.

Smaller aggregate sizes are used in the bottom-feed delivery system to prevent blockages in the stone feed pipe. Rounded aggregates tend to flow better in the stone tube and it is imperative that the fines content is kept below 5%, also to prevent blockages.

Natural (primary) aggregate resources are not unlimited within the UK and their extraction causes increasingly unacceptable geo-environmental impacts. As part of achieving environmentally sustainable development within the ground improvement sector, there is an increasing desire to utilise recycled (and secondary) aggregates in vibro stone columns. However, where such materials are considered for use it is important that there are appropriate specifications and quality control/assurance procedures in place to ensure 'fitness-for-purpose'. Spent railway track ballast and crushed concrete currently have the greatest potential for this application in the UK and other materials have been investigated. Further information on these aspects can be found in Slocombe (2003b) and Serridge (2005) among others.

84.2.10 Environmental considerations

84.2.10.1 Noise and vibration

Whilst it is rare for noise issues to be a problem with vibro techniques, vibration levels need to be considered when working close to existing structures and services so that these are not adversely affected. The safe working distance will depend on a number of factors including type of ground, the vibroflot power rating, the nature and state of repair of the structure and also plant access, particularly if any basement structures are present. As a guide, minimum distances of 1–3 m are often appropriate. However, each situation must be assessed on its own merits and dilapidation or precondition surveys should be considered, in addition to appropriate risk assessments and vibration monitoring.

84.2.10.2 Contaminants

Appropriate risk assessment and precautions will be required on contaminated sites and to avoid exposure to the atmosphere of chemicals and materials such as asbestos.

84.2.10.3 Vibro concrete plugs

Penetrative ground improvement techniques such as vibro stone columns can generate potential pathways for contaminant migration on brownfield contaminated sites. This raises environmental concerns, particularly if sensitive groundwaters or underlying aquifers are present due to potential pollutant linkages (i.e. source–pathway–receptor linkages). An innovative approach to address this issue has been the development of 'vibro concrete plug' technology, incorporating the introduction of lean mix concrete into the basal section (toe) of the stone column, thereby isolating any pathways for downward migration of contamination via the stone columns. Further guidance on pollution prevention in this context can be found in Environment Agency Report NC/99/73 (2001). The above technology has also been utilised to bridge thin peaty deposits within the treated depth.

84.2.11 Monitoring, quality control and testing

Ground improvement exhibits a high frequency of highly ingenious equipment and solutions employed in difficult ground conditions. Close monitoring of ground response, quality control and testing throughout the duration of the vibro ground improvement works is therefore essential.

Bell (2004) describes an example of a range of quality (control) issues possibly arising from insufficient attention to construction detail on what he considers should have been a routine vibro stone column ground improvement project in mixed (heterogeneous) soils in the UK (**Figure 84.8**). Poor (discontinuous) column construction was attributed to the lack of constructing the columns in discrete lifts, each of which is required to be compacted to a predetermined limit such as target hydraulic pressure or ammeter reading, dependent upon whether the vibroflot is hydraulically or electrically driven. It is also indicated that accurate records of the amount of stone aggregate consumed by each column would have revealed the deficiencies of construction during installation.

The majority of vibro stone column projects in the UK involve the support of structures. Therefore, in addition to the monitoring and recording of installation parameters, some form of validation testing using load testing and/or penetration testing, dependent upon soil type, is normally required. Load tests typically include:

- short-term plate load tests;
- zone load tests or skip tests;
- embankment load tests (surcharge load tests).

84.2.11.1 Short-term plate load tests

Plate load tests are normally of less than two hours' duration with plate diameter comparable to the design stone column diameter, normally standardised at around 600 mm. Loading is quickly applied to the test column to between 1.5 times and 3.0 times the design pressure over the plate area, using a crane or vibro rig as reaction. The tests are principally regarded as a

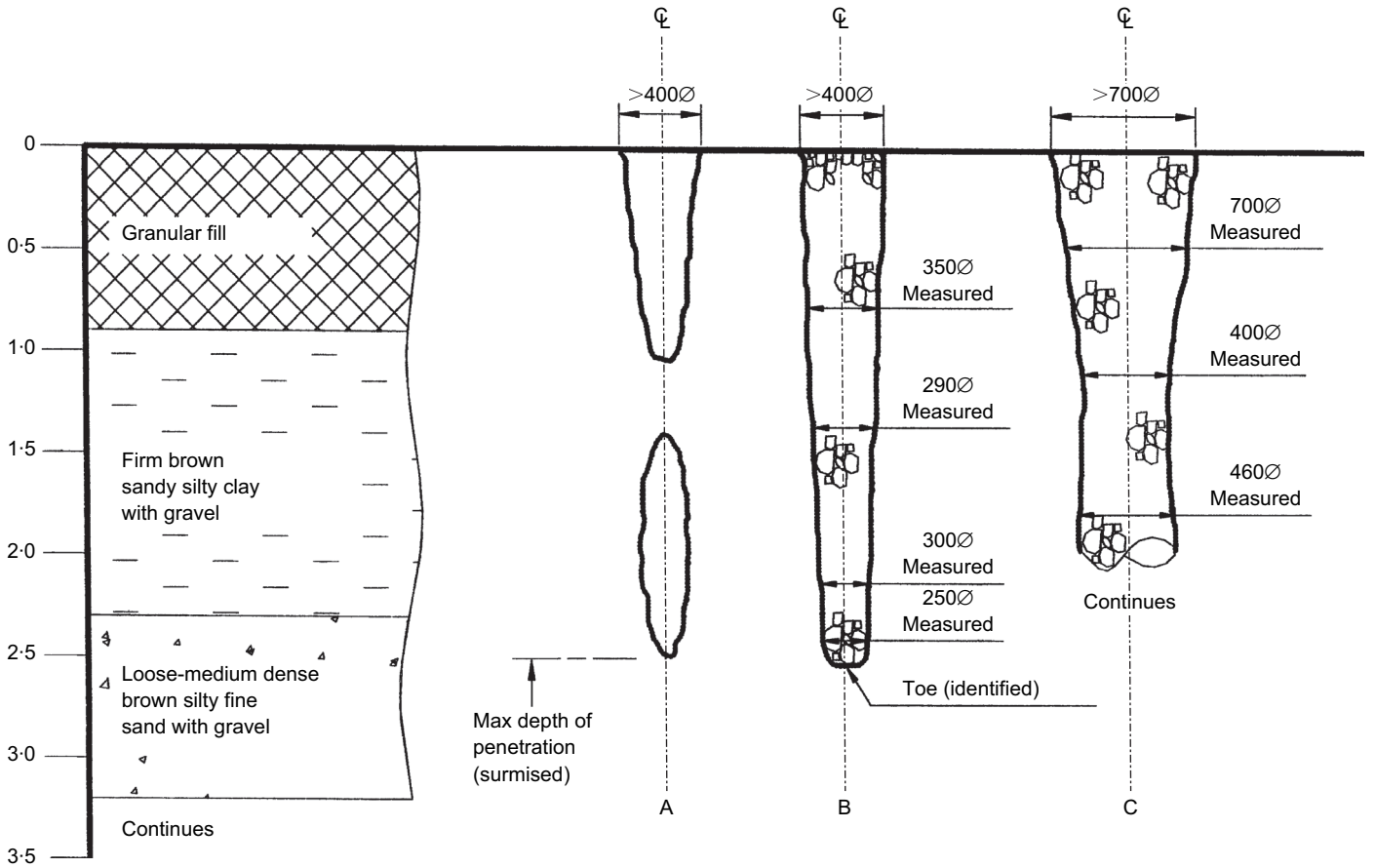


Figure 84.8 Example of poor quality control with vibro stone columns
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quality control test to assess level of workmanship with regard to stone column installation.

84.2.11.2 Zone load tests or skip tests/dummy foundation tests

Zone load tests are normally of similar size to the project foundations and are performed over a group of columns – either by direct loading or against a system of grillage and kentledge for a period of between 24 hours and one week. These tests are more meaningful than the plate load tests since they apply load to the composite structure of both stone columns and soil but are far more expensive because of the cost of importing kentledge to site.

Skip tests/dummy foundation tests provide an intermediate test between zone load tests and plate load tests and may be more practical and economic on smaller projects (see BRE BR 391 *Specifying Vibro Stone Columns* (2000) and NHBC Standards, Chapter 4.6 *Vibratory Ground Improvement Techniques* (2011).

84.2.11.3 Embankment loading tests (surcharge load tests)

These tests can impose loads over a larger area for a longer period of time. They are particularly useful for road embankments on soft ground where time-dependent performance may be important (to ensure post-road construction settlements fall within acceptable limits). As with untreated soft soils, the rate of loading should be carefully controlled and monitored via appropriate field instrumentation. This facilitates better assessment of the performance of widespread loads. Where such tests are not possible, emphasis should be placed on monitoring and observing ground response to stone column installation and ensuring the design column diameters are achieved, supplemented by good quality control.

84.2.11.4 Penetration tests

In situ tests are typically employed to assess the technical success of *in situ* vibrocompaction and performance is measured by the level of densification achieved against a specified target. The densification can be readily checked using standard penetration tests (SPT) or cone penetration tests (CPT). Comparisons can be made between pre- and post-treatment tests and care should be taken to ensure the same techniques of testing are used in each situation. They are not normally employed on stone column projects, particularly where the soils contain a significant fines component.

With both load tests and penetration tests, it is important that there is an appropriate recovery period following completion of vibro ground improvement to allow a satisfactory level of pore water pressure dissipation in order to obtain meaningful results. A suitability rating for different testing methods for vibro techniques in presented in **Table 84.2**.

A modulus of sub-grade reaction or California Bearing Ratio – CBR (BS 1377–9 (BSI, 1990)) value may sometimes be specified for a groundbearing floor slab. It is important to recognise that whilst there may be some improvement in relative density between stone columns in granular soils, this will not be the case in cohesive soils (where the stone columns are acting as 'reinforcing elements'). In this instance the main objectives of stone columns are for control of total and differential settlements. Floor slab design would need to be based upon values obtained at sub-grade between stone columns.

Test method	Granular	Cohesive	Comments
McIntosh Probe	*	*	Pre-/post-treatment essential. Can locate obstructions prior to treatment.
Dynamic cone	**	*	Too insensitive to reveal clay fraction. Can locate dense layers and buried features.
Mechanical cone	***	*	Rarely used in UK.
Electric cone (CPT)	****	**	Particle size important. Can be affected by lateral earth pressures generated by treatment. Most appropriate test for seismic liquefaction evaluation.
Boreholes and SPT	***	**	Efficiency and repeatability of test important. Recovers samples (e.g. split spoon sampler).
Dilatometer	***	*	Rarely used.
Pressuremeter	***	*	Rarely used.
Small plate test	*	*	Does not adequately confine stone column. Limited stress depth. Affected by pore water pressures.
Large plate test	**	**	Better confining action.
Skip test	**	**	Can maintain test for extended period.
Zone loading test	****	****	Best test for realistic comparison with foundations.
Full-scale (e.g. surcharge load test)	*****	*****	Not commonly used. Tends to be confined to highway embankment projects.

* Least suitable ***** Most suitable

Table 84.2 Suitability rating for test methods applied to vibro techniques
Data taken from Moseley and Priebe (1993)

84.2.12 Practical issues

There are a number of practical issues to consider in the context of vibro stone column techniques. The more pertinent ones are discussed below.

84.2.12.1 Dealing with obstructions

Obstructions, dense granular made ground and remnant foundations can be an issue in terms of preventing vibroflot penetration and potential for forming hard spots. These should be suitably remediated (e.g. pre-loosening/excavation with removal and/or screening/crushing of oversize granular material before returning to excavation) and with provision of an appropriate cushion of granular material between underside of foundation or groundbearing floor slab and top of buried remnant foundation.

84.2.12.2 Working platforms and stabilised platforms

Provision of a safe granular working platform is essential for safe access and execution of vibro ground improvement. Stabilised platforms (lime/cement) can be used, but due consideration needs to be given to implications of 'spragging' of rig tracks and ponding of water in inclement weather, which may necessitate placement of a surface granular layer.

84.2.12.3 Pre-boring

Many historic heterogeneous fills in the UK are quite competent, particularly in their uppermost layers. This, together with placement of any engineered/stabilised upfill in advance of vibro ground improvement, can impede vibroflot penetration, necessitating the use of pre-boring. Arisings will be generated which then need to be dealt with.

84.2.12.4 Slopes and interceptor drains

For construction at the base of slopes, it is important that there is appropriate drainage at the toes to intercept water draining off the slope, to avoid any detrimental effects on the treated ground.

84.2.12.5 Lateral forces

Vibrocompaction and vibro stone column construction, as with driven piling, can generate lateral forces that can act on existing retaining structures, sheet piles and quay walls. Consideration of these effects requires specialist input.

84.2.12.6 Gas venting

On historic landfill sites, stone columns can actively vent any gases trapped in the fill. Therefore consideration of provision of gas venting measures may be required following appropriate risk assessment.

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84.2.12.7 Installation tolerances

Accepted positional tolerances for vibro stone columns are more flexible than for piles and are typically quoted as 150 mm.

84.2.13 Conditions where caution is to be exercised for vibro stone columns

There are recognised ground conditions within the ground improvement industry which are either deemed not suitable for vibro stone column techniques, or where caution should be exercised. Some of the more significant ones are discussed briefly below:

1. *Excessive degradable materials* – On sites where the made ground contains excessive quantities of degradable domestic refuse – this would decay with time and cause loss of support to the stone columns and hence settlement. Generally, where the degradable refuse content is less than about 10% (by volume) and fairly evenly distributed, sites can be considered for treatment, but the design approach has to be conservative and whether or not vibro is adopted is dependent upon the building, its function, and also the client's (performance) requirements.
2. *Non-engineered recent clay fills* – This is a situation where self-weight movement will not be complete. Whilst introduction of stone columns may well improve the situation it is very difficult to control self-weight movements. Typically, significant depths of clay fill (up to 5–8 m) should not be considered suitable for treatment if less than five to eight years old. Each site has to be judged on its own merits and with due consideration of settlement performance requirements.
3. *Opencast mineworkings* – A variant on the backfilled clay pit is the backfilled opencast mineworking, where in addition to differential settlement implications for structures spanning the pit edge or buried high wall, there is the issue of restoration of the groundwater table (inundation), the timing of which is totally unpredictable. When the groundwater does re-establish itself there is a breakdown of the clay fill and further settlement, which can be in excess of the self-weight movements. Vibro can be considered if careful attention is given to sealing the tops of the stone columns with clay immediately following their installation and during subsequent foundation excavation by concrete blinding, thus inhibiting surface water ingress due to, for example, heavy rain.
4. *Thick peat deposits* – Thin peat layers can be accommodated (see BRE BR 391 *Specifying Vibro Stone Columns* (2000)). Thickness, depth and lateral variations have to be considered very carefully in relation to the size of foundation and its loading. On some sites covered by up to 2.0 m of peat, the peat has been excavated along the lines of the foundation over a 2.0 to 3.0 m width and replaced with granular fill, vibratory stabilisation then being carried out in the normal way.

84.2.14 Design of foundations after vibro ground improvement

The bearing pressures achievable with vibro techniques are a function of the engineering properties of the soils prior to ground improvement. **Table 84.3** provides a guide as to what can be achieved in terms of load-bearing capacity and settlement performance for a range of soil types and applications after vibro ground improvement. Each site has to be assessed on its own merits.

Following adoption of vibro ground improvement techniques, conventional shallow pad/strip foundations and groundbearing floor slabs can be adopted. Because the stone columns are formed by working against the overburden pressure, the stone aggregate at the top of the constructed columns is not as compact as at depth. For this reason the main load-bearing foundations should be placed at a minimum depth of 600 mm below the level from which the ground improvement was carried out in order to fully realise the enhanced performance. In cohesive soils, in particular, there should be appropriate allowance for minimum top and bottom mesh reinforcement. Levelling off and proof rolling of the sub-grade following completion of vibro ground improvement is normally acceptable prior to construction of groundbearing floor slabs.

84.2.15 Case histories

84.2.15.1 Introduction

Vibro techniques have been applied successfully on many projects and there are many well-documented case histories in the literature. Cases of unsatisfactory performance in vibro ground improvement inevitably occur but are not commonplace. They are usually not well publicised, principally because of legal and contractual ramifications. Nevertheless, there is much to be learned from situations where problems have arisen. Examples of unsatisfactory performance include those relating to ground risk and insufficient site and ground investigation data; those relating to lack of understanding of vibro ground improvement techniques and their application; lack of appreciation of the

Soil type	Bearing pressure (kN/m^2)	Settlement range after treatment (mm)
Made Ground: (cohesive and mixed granular and cohesive)	100–165	5–25
Made Ground: (granular)	100–215	5–20
Natural Sands or Sands and Gravels	165–500	5–25
Soft Alluvial Clays	50–100	15–75

Table 84.3 Typical improvements achievable in terms of load bearing and settlement after vibratory stabilisation
Data taken from Slocombe (2001)

implications of changes in site levels; issues such as inundation; those relating to quality control and workmanship issues (see section 84.2.11); plus later drainage excavations undermining the constructed foundations on treated ground.

84.2.15.2 Successful case histories using vibro stone columns

Case study 1

For highway schemes, the approaches to bridges (or the transition between rigid bridge abutments and the consolidating soil behind the abutment) are a challenge for geotechnical engineers who are exploring cost-effective ground improvement solutions, to ensure a smooth settlement profile and vehicle ride quality and to reduce maintenance. Dependent upon the soil profile and geometry, this can be achieved in critical zones using a combination of ground improvement techniques such as vibro stone columns (VSC), vibro concrete columns (VCC) and vertical band drains (BD) and also addressing issues such as lateral loads on abutment piles due to lateral 'squeezing' of the soil during soil consolidation beneath embankments. One application (see Serridge and Synac, 2007) is summarised in **Figure 84.9**, where vibro stone columns have been used in conjunction with other ground improvement techniques to support highway embankments over soft ground.

A similar application using a combination of VSC and VCC is described by Cooper and Rose (1999), for a project in Bristol.

Case study 2

Long-term renewable energy targets have led to the construction of increasing numbers of wind farms and biomass power plants. Vibro stone columns have been used to support their foundations, one example being for a 44 MW annual power production plant in Scotland with output sufficient to supply 70 000 homes.

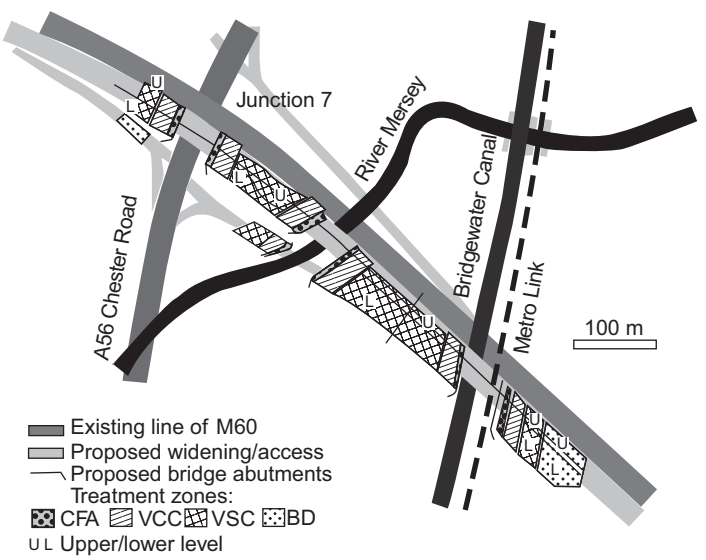


Figure 84.9 Ground improvement applications to support highway embankment construction over soft ground – M60 motorway widening, South Manchester

Reproduced from Serridge and Synac (2007); all rights reserved

The project involved a number of structures of various sizes and loadings, some also including heavy plant that was very sensitive to movement, particularly under the dynamic loadings during operation. Settlements were required to be limited to no greater than 20 mm under loadings of up to 250 kN/m^2. The near-surface ground conditions were highly variable comprising recent alluvial deposits, that were present in some areas but absent in others, variable depth of water table, then interbedded lenses and layers of saturated sands and silts, locally clays, all underlain by sandstone at depth. Some areas were considered suitable for dry top-feed vibro, some vibrocompaction without stone aggregate and others requiring bottom-feed treatment. The ground was so variable, however, that all treatment was performed using the dry bottom-feed technique to construct around 2000 stone columns to depths of up to 8 m. With such tightly specified post-treatment performance and variable soils additional design control checks were carried out using the monitoring systems on the bottom-feed installation rig to correlate the actual energy, depths of penetration and stone consumption as the work progressed. This effectively provided highly intensive investigation of the treatment areas, mapping areas to compare to soil properties reported in the site investigation.

One of the 'worst' areas was then subjected to a zone loading test on a 2.5 m × 2.5 m reinforced concrete base using an hydraulic jack reacting against kentledge blocks placed on grillage for maximum applied loading of 234 t or 1.5 times the design load. Settlements of 8 and 16 mm were respectively recorded at 1.0 and 1.5 times working load and were well within the specified 20 mm limit. The results are presented in **Figure 84.10**.

84.2.15.3 Case history of poor settlement performance

Wilde and Crook (1992) describe the settlement of a steel portal frame factory unit within the River Mersey floodplain in Warrington, Cheshire. The site was shown to be underlain by soft alluvial soils varying in thickness from 5 m to 10 m. Prior to implementation of vibro stone column ground improvement techniques, the site was brought up to the required plateau levels by addition of up to 1.5 m of upfill coincident with the maximum 10 m thickness of soft alluvial deposits. The subsequently installed vibro stone columns did not fully penetrate the soft alluvial soils and upon completion of ground improvement the factory unit was constructed on simple pad and strip footings and with adoption of a groundbearing floor slab. Subsequent monitoring of the portal frame structure showed that over a six-year period 120 mm of total settlement occurred, with maximum differential settlement of 100 mm along the length of the structure. It was estimated that there was probably another 50 mm of settlement during the construction period, but which was not recorded. Levelling of the floor slab revealed similar movements to those experienced by the main foundations.

It is clear that the majority of recorded settlement is attributed to the surcharge effect of the upfill associated with raising of site levels rather than the relatively small weight of the industrial unit. This brief example clearly demonstrates the need for careful consideration of changes in site levels in the context of vibro stone column techniques, when considering treatment to thick layers of soft soils.

This example illustrates some of the difficulties of defining unsatisfactory performance. The building in this instance performed satisfactorily despite the high total settlements and without apparent structural distress or serviceability problems and suggests that some settlement tolerances specified for low-rise lightweight buildings are unnecessarily stringent.

84.2.16 Conclusions

Vibro techniques are adaptable to a wide range of soil conditions and permit adoption of simple foundation solutions. Vibrocompaction will increase the density of clean sands (and mitigate liquefaction potential) and vibro stone columns will reinforce fine-grained soils. Both techniques will improve the bearing capacity of the ground and result in reduced total and differential settlements.

Vibro design relies heavily on good quality site and ground investigation data for accurate prediction of performance.

Good quality control procedures are necessary to ensure the successful implementation and performance of the vibro ground improvement techniques.

Where knowledge of a particular vibro technique is lacking, or there is uncertainty as to the applicability of vibro techniques to a specific set of site circumstances and performance requirements, then specialist advice should be sought to ensure appropriate vibro ground improvement selection and specification.

84.3 Vibro concrete columns

84.3.1 Introduction and history

Vibro concrete columns (VCCs) were first developed in Germany by injecting grout into already constructed stone columns. The aim was to overcome the lack of confining action to stone columns from very soft soils and peat. The grouting operation was then replaced by pumping suitable concrete directly down the bottom-feed delivery tubes, the first use being in the mid-1970s. VCCs were introduced into the UK in 1982.

84.3.2 VCC plant and equipment

VCCs are constructed using the specialist instrumented bottom-feed vibro stone column equipment and vibrators, but without the stone skip that normally runs up and down the leaders (see **Figure 84.11**). The concrete is then delivered to the top of the vibrator via an external pump.

84.3.3 VCC concrete, reinforcement and technique

In Germany both high and low slump concretes with high cement contents are used. However, in the UK the technique generally uses low slump concrete (typically less than around 60 mm) to be able to use vibration and ramming of their bases to densify basal granular soils to enhance end-bearing and generating enlarged 'end bulbs' for higher capacity at shorter length than for conventional pile design. Higher slumps have been used for some applications in the UK, depending upon site-specific circumstances. Once the end bulb is formed the vibrator is withdrawn at a controlled rate from the ground whilst maintaining positive pressure of concrete. The construction process also tends to be faster than normal piling when performed in suitable ground conditions.

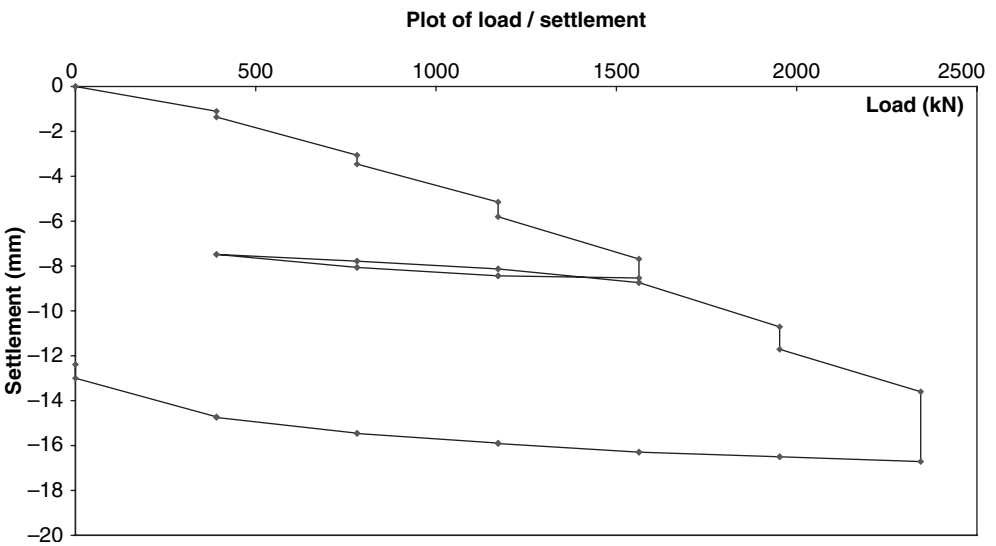


Figure 84.10 Zone load test on vibro stone columns for biomass plant
Data courtesy of Keller



Figure 84.11 Vibro concrete column equipment
Courtesy of Keller

VCCs are normally designed to support vertical axial loads. However, they are generally not constructed using reinforcement cages, sometimes simply with a central large section bar, sometimes with no reinforcement when beneath floor slab areas and road embankments, but normally with tie bars to tie into foundations. As a result individual VCCs have limited capacity to resist horizontal, shear and tension forces.

84.3.4 Applications and limitations

The VCC construction process allows a 'ground improvement' approach that is especially useful when considering the control of settlements of road embankments that are approaching piled bridges and in their superior settlement performance in comparison to vibro stone columns in very soft soils (see Maddison *et al.*, 1996; Cooper and Rose, 1999; Serridge, 2006). Enlarged heads are commonly constructed to assist in the transfer of embankment loadings via granular geogrid-reinforced load transfer platforms, as well as more efficient industrial floor slab design.

The construction of VCCs with end bulbs and tight interlock of concrete to soil can also prevent migration of contamination to underlying aquifers (see Environment Agency, 2001). Also, as the VCCs can develop safe loads of up to 600 kN with only short penetration of a sand/gravel stratum, they are useful where conventional piling would normally have to penetrate deeper. This shorter 'pile length' is particularly useful for the prevailing ground conditions to the east and west of London where there is generally a suitable layer of sand and gravel underlain by either London Clay or Chalk aquifer.

The system is relatively quiet with low vibration in comparison to driven piles and does not require the removal of spoil that would be required for continuous flight auger (CFA) piling.

As with all displacement piles care has to be exercised when installing VCCs at closer than about three times their diameter. It is also common practice to install single full-length central large section bars for spacings of up to about six times the VCC diameter (typically about 400–500 mm) to resist the effects of installing adjacent columns. Significantly larger diameters are generated in weak peat layers.

84.3.5 Site investigation (ground model)

The safe working capacities of VCCs are predominantly derived from end-bearing minus the assessed negative skin friction along the length of the pile shaft. As a result the depth to and penetration into suitable founding stratum has to be established over the extent of the proposed development. In addition, where VCCs are founded within granular soils that are underlain by cohesive strata, the capacity may have to be limited where the granular strata are relatively thin. Where there are thin clay layers within the granular founding strata the VCCs should be designed on the basis of a clay toe.

Such details are often best investigated by cone penetration tests (CPT) since these provide greater detail than standard penetration tests (SPT) in boreholes. It is relatively common for VCC quotations to include for the performance of one or two days of CPT investigation to better refine the VCC design.

The soil chemistry must also be investigated to provide suitable buried concrete design, as per conventional piles.

84.3.6 Monitoring and testing

The specialist VCC rigs include computer-generated instrumentation, comparable to CFA piling rigs, to provide continuous records of the depths, concrete pressures, extraction rates, etc. of every column. Static and dynamic loading tests are often performed to confirm capacity. However, integrity tests are difficult to interpret due to the sometimes irregular shape of the pile shaft, particularly in the presence of thick peat layers.

84.3.7 Concluding remarks

VCCs are effectively an alternative type of pile that is constructed using vibro equipment, often in conjunction with stone columns, for better settlement performance in softer and peaty soils. They are perhaps best considered as settlement reduction piles.

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84.4 Dynamic compaction

84.4.1 Introduction

Dynamic compaction (DC) improves weak soils by *controlled* high energy tamping. The reaction of soils during dynamic compaction treatment varies with soil type and energy input. A comprehensive understanding of soil behaviour, combined with experience of the technique, is vital to successful improvement of the ground. Dynamic compaction is capable of achieving significant improvement to substantial depth, often with considerable economy when compared to other geotechnical solutions.

The principle of dropping heavy weights on the ground surface to improve soils at depth has attracted many claims for its earliest use. However, the advent of large crawler cranes led to the current high energy tamping levels first being performed on a regular basis in France in 1970 and subsequently in Britain in 1973 and in North America in 1975.

An extension of the concept of weights dropped onto the ground – rapid impact compaction (RIC) – was developed in the late 1970s for the rapid repair of explosion damage to military airfield runways using modified (BSP) hydraulic piling hammers acting on a steel foot that remains in contact with the ground and is now used for civilian applications.

84.4.2 Processes

The original concept for dynamic compaction was to collapse voids, particularly for the treatment of natural sands plus granular, mixed and cohesive made ground and fills. This was then extended to finer natural soils where the high impact energy effectively provided localised surcharge to squeeze water out of silts and clays, this being termed dynamic consolidation. Dynamic replacement was then developed to drive large diameter columns of coarse imported materials through soft near-surface soils, particularly for peat and sabkha strata.

The worldwide use of dynamic compaction has resulted in a large number of important terms, some of which can have different meanings to different nationalities or could be confused with other geotechnical descriptions. The following are the main terms that have been adopted in Britain (see Slocombe 2003a for fuller listing):

(a) 'Effective depth of treatment' is the maximum depth at which significant improvement is measurable.

(b) 'Drop energy' is the energy per blow, i.e. mass multiplied by drop height (tonne metres).

(c) 'Tamping pass' is the performance of each pattern of treatment over the whole treatment area.

(d) 'Total energy' is the summation of the energy of each tamping pass, i.e. number of drops multiplied by drop energy divided by respective grid areas (normally expressed in tonne metre/metre^2).

(e) 'Recovery' is the period of time allowed between tamping passes to permit the excess pore pressures to dissipate to a low enough level for the next pass.

(f) 'Induced settlement' is the average reduction in general site level as a result of the treatment.

84.4.3 Techniques

Dynamic compaction, whether to shallow or deep layers, improves the ground to the basal layers first and then progressively upwards to the shallower layers in a series of tamping passes (i.e. a bottom-up process) (see section 84.4.6). In contrast the rapid impact compaction (RIC) improves the soils by first creating a 'plug' or layer of denser ground and then progressively driving this plug/layer to greater depth (i.e. a top-down process). The response of the ground to these two approaches is fundamentally different. Whereas the relatively lower number of high energy impacts at wide grid centres of DC tends to initially bypass the upper layers and then, by subsequent progressive treatment builds up the strength of the near-surface soils, the higher number of lower energy impacts of the RIC requires consideration of the possible generation of pore water pressures that could inhibit the required ground improvement in finer-grained soils.

84.4.4 Plant and equipment

At first sight, the physical performance of dynamic compaction would appear to be simple, i.e. a crane of sufficient capacity to drop a suitable size of weight in virtual free fall from a certain drop height. Most contracts are performed with standard crawler cranes, albeit slightly modified for safety reasons and productivity, with a single lifting rope attached to the top of the weight (**Figure 84.12**). The operation must be performed safely and as a result the Health and Safety Executive in Britain requires that a crane should operate at not more than 80% of its safe working load.

Recent crane developments allow automation of the whole work cycle. This is controlled by a data processing unit which plots for each compaction point its location, number, weight size, drop height, number of blows and measurement of imprint achieved. Some cranes include the ability for synchronous operation of two winches to lift larger weights than the conventional crane rating.

The majority of DC contracts in Britain have utilised weights within the range of 6 to 20 tonnes dropped from heights of up to 20 m. The majority of UK work is now performed using 8 tonne weights dropped from heights of up to 12 m. Standard crawler cranes have also been used in America and Europe for weights of up to 33 tonnes and 30 m height. Specialist lifting frames with quick-release mechanisms have been utilised to drop weights of up to 170 tonnes in France.

Weights are typically constructed using toughened steel plate or box-steel and concrete. The effect of different sizes and shapes of the weight has also been extensively researched

with narrower weights being generally used for dynamic replacement columns. Treatment has also been performed below water using barge-mounted cranes and more streamlined weights with holes cut out to reduce water resistance and increase impact velocity on the sea bed.

Within the UK RIC typically employs a 7 tonne weight dropped repeatedly through a 1.2 m height onto a 1.5 m diameter steel articulated compaction foot which remains in contact with the ground (**Figure 84.13**). Whilst the energy per blow is not large (typically 8.4 tonne.metre), the equipment permits a large number of impacts to be applied at a rate of about 40 blows per minute. Weights ranging from 5 to 12 tonnes are used worldwide.

84.4.5 Applications and limitations

Dynamic compaction can be applied to many of the filled sites that are considered to be suitable for treatment by vibro methods, except where weak natural clays and silts are the main problem layer (see section 84.4.6.2). Many more UK sites are treated by vibro due to the noise and vibration generated by dynamic compaction and the ability of the dry vibro methods to permit closer follow-on operations. Dynamic compaction can, however, provide superior performance in comparison to vibro for treatment to younger deep made ground and in the presence of degradable constituents within landfills.

84.4.6 How dynamic compaction works

In contrast to having constructed a vibro stone or concrete column, the treatment at that location being then completed, dynamic compaction is generally performed as a series of tamping passes over the treatment area with different combinations of energy levels designed to achieve improvement to specific layers within the depth requiring improvement. The most common approach is to consider the ground in three layers. The first tamping pass is aimed at treating the deepest layer by adopting a relatively wide grid pattern and a suitable number of drops from the full-height capability of the crane. The middle layer is then treated by an intermediate grid, often the



Figure 84.12 Dynamic compaction equipment
Courtesy of Keller



Figure 84.13 Rapid impact compaction equipment
Courtesy of Balfour Beatty Ground Engineering Ltd

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mid-point of the first pass or half the initial grid, with a lesser number of drops and reduced drop height. The surface layers then receive a continual (contiguous) tamp of a small number of drops from low height on a continuous pattern to compact the uppermost soil layers disturbed during the earlier higher energy tamping passes.

It is sometimes feasible to combine, and sometimes necessary to sub-divide, the three basic tamping passes for various reasons. The performance of increasing correctly controlled total energy input will normally lead to better engineering performance of the treated ground. This, however, is not a linear relationship and the post-treatment parameters are heavily dependent upon the characteristics of the soil. As a general rule similar total energies, whether per m^2 of area or m^3 of treatment depth, provide better performance to granular soils than mixed soils. Mixed soils are then better than cohesive with refuse-contaminated soils generally offering the least performance.

Some dynamic compaction design principles are provided in Chapter 59 *Design principles for ground improvement*.

For treatment using RIC the operator monitors and can record the number of impacts, the total energy input applied, the foot penetration per blow and the cumulative penetration. When a specified parameter is reached, for example, foot penetration or set per blow, the RIC equipment is moved and positioned at the next treatment point. This primary treatment pass is normally performed on a closely spaced grid pattern, typically 1.5 to 2.5 m. Additional offset and/or lower energy passes, or conventional proof roller compaction, are occasionally performed to achieve better coverage.

84.4.6.1 Granular soils

In dry granular materials, such as sand, gravel, ash, brick, rock, slag, etc., it is easy to understand how heavy tamping improves engineering properties. Physical displacement of particles and, to a lesser extent, low frequency excitation will reduce void ratio and increase relative density to provide improved load bearing and enhanced settlement characteristics.

When granular materials extend below the water table, a high proportion of the energy impulse is transferred to the pore water which, after a suitable number of surface impacts, eventually rises in pressure to sufficient level to induce liquefaction in a similar manner to vibrocompaction. Low frequency vibrations caused by further stress impulses will then reorganise the particles into a denser state.

Dissipation of the pore water pressures, in conjunction with the effective surcharge of the liquefied layer by the soils above, results in further increase in relative density over a relatively short period of time. This can vary from one to two days for well-graded sand and gravel, to one to two weeks for sandy silts. The testing program should therefore recognise the time-dependent response for soils that are normally considered to be free-draining.

Laboratory and *in situ* tests have consistently shown that in order to achieve maximum density, the lowest number of stress impulses to attain the required energy input will provide the optimum result. Saturated granular soils will normally require higher treatment energy overall, in a larger number of tamping passes, than if the soils were essentially dry.

When the individual sand particles are weak, such as the calcareous sands of the Middle East, 'sugar' sands of North America or the Thanet Sands of Britain, crushing tends to occur during the treatment. A similar response affects ash, clinker and weak aerated slags. When these soils are dry, the effect of such particle breakdown is not particularly significant. However, below the water table the higher proportion of fines developing with increasing energy input results in a rapid change from a granular to a pseudo-cohesive soil response.

In summary, excellent engineering performance can easily be achieved in dry granular soils using both DC and RIC equipment. However, care must be exercised for the treatment of granular soils with significant silt content, particularly below the water table.

84.4.6.2 Cohesive soils

The response of clays is more complex than that of granular soils. There is again the distinction between above and below the water table.

With conventional consolidation theory, a static surcharge loading will collapse voids within clay fills and expel water to induce consolidation and increase strength. In contrast, dynamic compaction applies a virtually instantaneous localised surcharge that collapses voids and transfers energy to the pore water. This creates zones of positive water pressure gradient which induce water to drain rapidly from the soils matrix. Consolidation can therefore occur more rapidly than would be the case with static loading. However, as with staged construction, the application of too high energy too soon can lead to problems.

Where the soils occur above the water table, the clays tend to be of relatively low moisture content, generally less than their plastic limit, where even a small reduction in water content can result in significant improvement in bearing capacity. Treatment is relatively straightforward and is mainly the collapse of voids to provide a more intact soil structure. However, care must be applied for the treatment to higher plasticity clays.

Where the clays occur below the water table, a much larger reduction in moisture content is generally required in the presence of a smaller available pore pressure gradient and a longer drainage path. These conditions can, if not properly controlled, result in localised failure of the clay matrix. Control is achieved by using greater numbers of tamping passes of lower energy input, requiring greatly extended contract periods in comparison to normal productivity.

To date, only nominal degrees of improvement have been achieved for thick layers of relatively weak saturated alluvial clays and silts, even with additional measures such as sand or wick drains. It is, however, more common in the UK to adopt vibro stone columns in such soils and then perform dynamic compaction to pre-load the ground (see Slocombe, 1989) with

the benefit of the stiffer columns also acting as drains to control excess pore water pressure.

It cannot be emphasised too strongly that the treatment of clayey made ground and fills together with natural clay soils requires experienced control on-site. Particular care has to be exercised in the timing of successive tamping passes to permit adequate recovery of pore pressures to avoid excessive remoulding of the soils. Efficient treatment is achieved by attempting to provide as much improvement as quickly as possible while recognising that the response of the soils will dictate the speed of the treatment operations.

In summary, dry cohesive made ground and fills respond well to dynamic compaction. Care must be exercised in the treatment of weak natural clayey soils or clayey made ground and fill below the water table, i.e. when saturated. The prior performance of vibro stone columns to both stiffen the ground and enhance drainage has been successfully combined with dynamic compaction to clayey soils.

84.4.6.3 Landfills

The capability of dynamic compaction to treat every square metre of road and parking areas is increasingly used in the development of former landfill sites where, depending on their age, the original degradable constituents have decayed to create extensive voids. It has also been performed to reduce ground levels to avoid costly removal to specialist tips to permit development at the desired site level.

As a general rule, the older the landfill the less the residual presence of matter susceptible to long-term decay and some older landfills, particularly those of high ash content, have been compacted to support structures that would normally be piled. However, the more recent landfills generally contain significant proportions of organic matter and structures would normally be piled.

There is as yet little documented proof that dynamically compacted landfill can affect the rate of decay of residual degradable constituents, although a paper by Sharma and Anirban (2007) clearly records far better post-treatment performance at creep rate of 2% per log cycle than for static surcharge over a monitoring period of about 15 years. As there will be ongoing decay, when this technique is combined with piled structures, increasing differential settlements will become apparent with time and a degree of maintenance may be required at some future time.

The principle of treatment to landfills is comparable to the treatment of mixed clayey made ground and fills but with generally higher energy input than for inert made ground and fills. This is to collapse near-surface voids and to 'overcompact' the remaining inert constituents. If a void then starts to develop due to localised long-term decay then the inert materials will tend to ravel into the void, bulk up and spread the void effect rather than have a localised sharp deformity in the finished surface. Geogrids have also been used for a number of sites where the movements of heavy goods vehicles were critical to the development operations.

84.4.7 Site investigation

As with vibro designs the extent of the site investigation should be appropriate to the type of development. For deep fill sites, desk studies to establish the locations of buried high walls plus age and degree of control of placement are essential data. Water contents for comparison with liquid and plastic limits should be performed for clayey constituents. Densities as revealed by SPTs and CPTs plus the basal soils, whether clay, sand and gravel or rock are also required. The presence of any overhead wires, buried services or nearby structures should also be established.

84.4.8 Depth of treatment

Menard and Broise (1976) originally proposed that the effective depth of treatment was related to the metric energy input expression of $(WH)^{0.5}$ where W is the weight in tonnes and H the drop height in metres. This has now been modified by a general factor of 0.5, reducing to 0.3 for heavier weights and greater drop heights. The range of treatment depths can also vary with initial strength, soil type and energy input as illustrated in **Figure 84.14**, as well as the depth to the groundwater table.

High-speed photography has shown typical impact speeds of about 35 and 50 mph for 8 and 12 tonne equipment to achieve effective depths of treatment of about 5 to 6 m and 6 to 8 m respectively. Within these depths, for similar volumetric energy input, the 12 tonne treatment will generally achieve better engineering performance than the 8 tonne weight, albeit at greater cost in view of the larger cranes required.

The shape of improvement in the ground tends to be similar to the Boussinesq distribution of stresses for a square foundation. Modification of energy levels for each tamping pass can be used to custom-design the treatment scheme to the specific soils profile and engineering requirements.

For rapid impact compaction, the BRE Report No 458 (2003) records depths of treatment of between about 2.0 and 4.0 m at total energy inputs ranging from 80 to 190 Tm/m^2 for granular fills and silty sands. Greater treatment depths using higher energy inputs in favourable conditions, together with a range of applications, have been reported by Serridge and Synac (2006).

84.4.9 Practical issues

There are a number of practical factors that must be taken into account when performing dynamic compaction contracts. The large crawler crane must be safely supported by a free-draining working surface, the thickness of which will depend upon the type of ground being treated. If the surface 1.0 m layer is basically granular, no imported working carpet is generally required. However, when working from a sandy surface, particularly during wet weather, fly-debris has been seen to be ejected through the air up to 60 m from the point of impact. If work is to be carried out near to roads, railways or property a movable screen is often used to intercept such fly-debris, albeit these affect the productivity. Alternatively, the programme should contain sufficient flexibility to permit treatment to be performed within,

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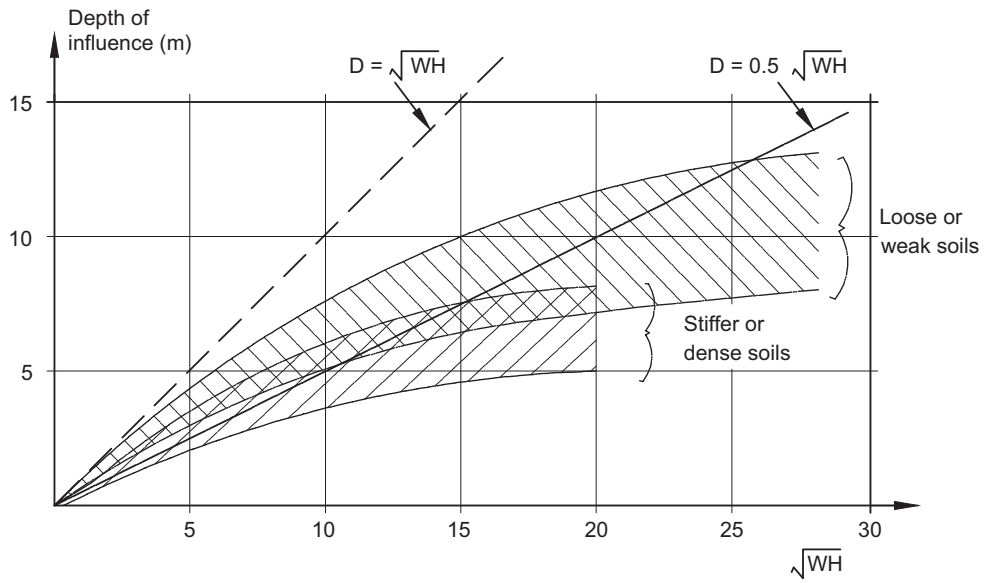


Figure 84.14 Depth of treatment

say, 50 m of such features only when the surface conditions permit its safe operation. As the RIC foot remains in contact with the ground there are no such issues with this equipment.

Where cohesive surface conditions exist, a free-draining granular working carpet is normally required. The thickness can be as little as 150 mm for light energy treatment in reasonably competent soils to up to 1.5 m when treating heavily voided refuse fills. Winter working will place more onerous requirements on the adequacy of the working surface. The general rule is to increase the depth of the granular working carpet by 25% in comparison to summer thickness. When working in arid climates there is often no need for any working surface, even for clayey soils.

The free-draining granular surface materials are also used to infill the localised deep imprints generated by the high impact energy treatment operations by blading in with a large dozer. Alternatively, suitable on-site granular materials, e.g. crushed concrete, can be placed directly into the imprints. In such cases it is useful to compare the imported quantities to the assessed volume of the directly infilled imprints.

As the performance of dynamic compaction tends to induce increases in water pressures, a pre-existing groundwater table within about 1.5 to 2.0 m of the working level can inhibit the productivity of the technique. In such cases bottom-feed vibro stone columns may be the preferred approach.

If more than one rig unit is to be used they should be separated by at least 30 m. Similarly, subsequent operations by the main contractor may have to be delayed until the treatment operations are sufficiently remote. Furthermore, whilst dynamic compaction can be performed over areas of vibro stone columns it has to be performed before any adjacent piling operations to avoid possibly damaging the constructed piles.

84.4.10 Environmental considerations

Dynamic compaction utilises large, highly visible equipment. The process creates noise and vibration, both of which must be considered in Britain under the Control of Pollution Act, 1974. The standards listed in the reference section provide further details (BS 5228–1 and 2: 2009; BS 7385 Part 1, 1990 and Part 2, 1993; BRE Digest 403, 1995).

Airborne noise levels are generated by a number of factors. Of these the point of impact is by far the highest noise level at typically 110 to 120 dB at source. However, its duration only occupies about 0.5% of the lifting cycle. The considerably lower noise values during lifting and idling when combined with the impact noise using the LAeq calculation method will normally meet most environmental limitations at distances of greater than 50 m from the treatment operations.

By far the most important consideration, however, is ground vibration. In addition to the magnitude of the vibration, the typical frequency of about 5–15 Hz is potentially damaging to structures and services, and particularly noticeable to human beings. It is suggested that there are three vibration levels that will influence the design of the treatment scheme. Guide values of resultant peak particle velocity at foundation level for buildings in good condition are:

Structural damage	40 mm/s
Minor architectural damage	15 mm/s
Annoyance to occupants	2.5 mm/s

Lower values must be adopted for buildings in poor condition or environmentally sensitive situations such as schools, hospitals and computer installations. Services and utilities

must be considered on an individual basis depending upon their age, condition and importance with values of 15–20 mm/s normally being considered acceptable, except for higher pressure gas mains.

The level of vibration transmitted through the ground is an imprecise science because of the variable nature of the characteristics of soils. Field measurements of vibrations at ground level have revealed a number of trends which are illustrated in **Figure 84.15**. The upper dynamic compaction limits tend to occur in the presence of granular or refuse-type soils with the lower limits in cohesive strata.

Careful assessment is required where the soil being treated is directly underlain by relatively dense sand, gravel or rock which will tend to transmit vibrations over larger than normal distances with comparatively little attenuation. Pre-existing dense surface or buried layers can have a similar effect of causing the transmission of higher than anticipated vibration levels.

As the RIC equipment is based on lower impact energy but greater numbers of drops this method has been employed as close as 10–15 m from an existing structure that was to be extended.

When vibrations become a problem the main methods of reducing their effect are to simply reduce the drop height, to reduce both the impact energy and penetration of the stress impulse that may have attained an underlying dense stratum, and/or to excavate a cut-off trench to sufficient depth to intercept the surface wave.

Human beings are particularly sensitive to vibrations and have a psychological reaction in believing that damage is caused even though the values are far below the well-established damage threshold levels. A public relations exercise can sometimes help to overcome concern amongst local residents. Building condition surveys prior to the commencement of treatment are often advisable.

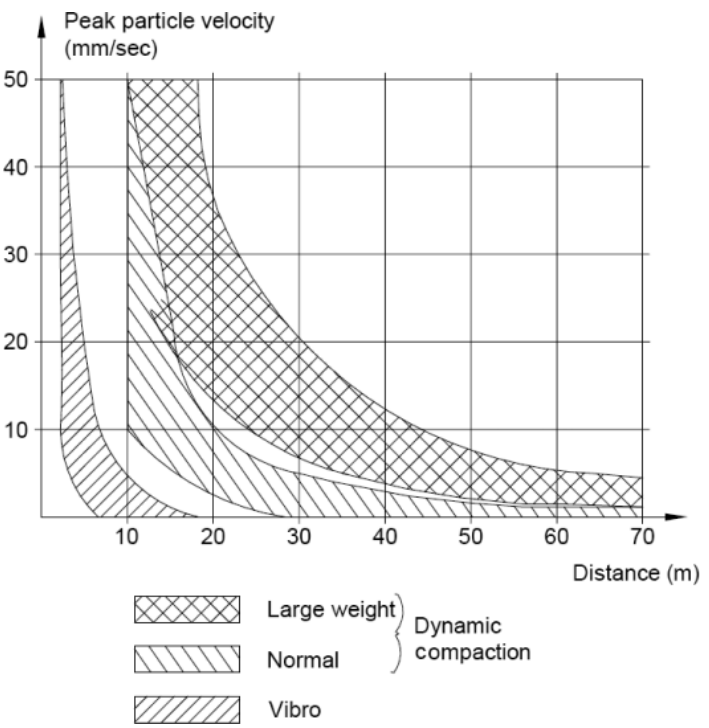


Figure 84.15 Vibrations

Dynamic compaction is a highly sustainable technique since it does not use cement or quarried stone, normally only requiring suitable inert free-draining granular waste as a working platform and to infill localised deep imprints. It is also an area treatment technique that permits changes in foundation layouts and localised loadings, for example mezzanine support foundations, anywhere within the treatment area. A number of dynamic compaction contracts have permitted the rebuilding of developments where fire destroyed the original by simply performing a number of loading tests upon the treated ground.

84.4.11 Monitoring and testing

Many contracts have simply involved the measurement of depth of first pass imprints and monitoring of site levels. Post-treatment *in situ* (SPT and CPT) and loading tests are also often performed in a similar manner to vibro testing (see section 84.2.11 and **Table 84.2**). In clayey soils, as with the performance of the treatment, it is essential that a sufficient recovery period be allowed to avoid ambiguous results.

It is common for dynamic compaction to be performed for sites underlain by coarse made ground or including obstructions that would cause penetration problems for vibro and piling methods or *in situ* testing. Surface loading tests only are more normally performed in this situation.

Tests normally reveal superior performance being achieved when treating granular materials in comparison to clayey soils.

84.4.12 Additional comments

The general densification and collapse of voids will induce general reduction in site levels, the settlement being dependent on the total energy input and the manner in which it is applied. A convenient simple approach is to adopt approximate percentages (see **Table 84.4**) of the target treatment depth for 8 tonne equipment (50 to 100 Tm/m^2) and 12 tonne (100 to 200 Tm/m^2), the total energies with the 12 tonne energy applying to greater depth of treatment.

Higher percentages can be induced. However, the increase in energy input will not be a linear relationship with induced

Soil type	% of treatment depth
Natural clays	1–3
Clay fills	3–5
Natural sands	3–10
Granular fills	5–15
Refuse and peat	7–20

Table 84.4 Guide to induced settlement

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settlement as during the treatment the material becomes progressively stronger and there is less and less potential void reduction available. Care has to be exercised to avoid over-treatment and possible loss in strength in these situations. Also, many sites, especially refuse tips, tend to be capped by clay soils.

Loose materials will obviously be induced to settle more than denser soils. Ash and certain types of slag also tend to break down during treatment to produce induced movements towards the higher value for granular fills given above.

Many sites of former heavy industry are now being reclaimed in Britain. These often present the designer with the problems of deep made ground, uncontrolled fills and massive obstructions from old basements and foundations. Any technique that has to make a hole in the ground will experience difficulty in gaining adequate penetration and large excavations often have to be performed to remove the obstructions. However, with dynamic compaction such features can be left in place provided they occur at sufficient depth to avoid excessive differential performance. The sequence of operation would be for advance earthworks to remove all known features down to a specific level, then perform treatment and normal construction operations.

Brownfield sites with minor contamination are well suited to treatment since the technique does not create a bore that could permit the migration of leachate. However, care should be exercised to ensure that the impact energies do not shear any underlying clay layer that may prevent the downward migration of contaminants into an aquifer. Dynamic compaction can also be controlled to avoid exposure of hazardous material to the atmosphere while still compacting soils at depth in the presence of, say, asbestos contamination.

Sites of former quarries are prime situations for treatment in view of the potential for piles to glance off the buried sub-vertical face between made ground and rock, and the possibility of constructing piles to inadequate depth as a result of false readings from boulders or inaccurate historical information on the depth of the quarry.

An area of increasing importance is the treatment of partially saturated made ground and natural soils that are susceptible to collapse compression due to inundation for the first time. Dynamic compaction reduces such potential by generally reducing voids, inhibiting surface water from penetrating the ground and by creating a 'stiff crust' over the effect of deeper movements, should they occur.

84.4.13 Case histories

84.4.13.1 Case study 1

A former colliery site was to be developed for several very large industrial and retail units including three-storey offices. The existing fills and mounds of colliery spoil were first excavated and then re-compacted in layers to minimum 95% maximum dry density. However, better settlement performance of maximum 35 mm total and 1 in 1000 differential over 25 years was required irrespective of the depth of fill.

On the first phase of 92 000 m^2 development the depths of fill varied between about 5 and 8.5 m with one area being up to 12.5 m depth beneath the proposed levels. The deeper zone was treated from lower level during the filling operations and then from the final level using 12 tonne weights dropped from heights of up to 15 m.

The second phase of 45 000 m^2 was performed after the first phase had been constructed. This area was underlain by between about 1.5 and 5.5 m depth of fill and was also located near to an electricity sub-station. This work was performed using 8 tonne weights and drop heights of up to 12 m.

The two phases of treatment were performed using similar volumetric energy input (Tm/m^3) for the full depths of fill. They were then tested by zone loading tests on 2.0 × 2.0 m test bases to twice working load of 96 tonnes. In addition a number of tests were performed on conventionally compacted fills, i.e. untreated ground. Comparison of their results revealed equivalent moduli of:

Untreated	E = 16–18 MPa
8T DC	E = 30–50 MPa
12T DC	E = 50–60 MPa

84.4.13.2 Case study 2

Dynamic compaction was performed in several contracts to a large area of landfill that was to be developed to industrial and office use. The first contract was to the deepest and newest landfill, parts of the site being up to 11.5 m depth and being filled with predominantly paper and cardboard waste only the year before treatment. Groundwater was typically at about 2.0 m depth.

All structures were supported on driven piles founded in the underlying competent clays with dynamic compaction being performed for the roads, parking and service areas. Treatment was performed using 12 tonne weights with cranes operating from nominal 1.0 m thickness of free-draining working platform.

The treatment area extended near to existing industrial units where the landfill was much shallower. This required a reduction in drop heights, while still capable of improving the full landfill depth within agreed vibration limits. The site was also near to a local railway line where safety precautions were put in place.

Induced settlements of over 1.0 m were recorded within the deepest landfill area. Zone loading tests were performed to confirm that the required short-term performance had been achieved. The site was then monitored for six years with maximum settlement of 75 mm. This was assessed as being equivalent to rates of creep movements of about 1–1.5% per log cycle.

84.4.14 Concluding remarks

The dynamic compaction method is a powerful tool when applied to suitable sites. A large database has been collated over the years to define its limitations and, more importantly, capabilities which can be used with confidence. As with every specialist technique, the designer and contractor performing

this method of ground improvement must understand these capabilities and limitations. Such understanding can only be gained by experience.

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It is recommended this chapter is read in conjunction with

- Chapter 25 *The role of ground improvement*
- Chapter 59 *Design principles for ground improvement*
- Chapter 100 *Observational method*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 85
Embedded walls

Robert Fernie Skanska UK Plc, Ricksmanworth, UK
David Puller Bachy Soletanche, Alton, UK
Alec Courts Volker Steel Foundations Ltd, Preston, UK

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This chapter considers the various forms of embedded wall and in particular focuses upon the selection of and construction issues associated with the different types. It considers the economics of different wall types and gives guidance on issues such as adjacent structures, groundwater, temporary works ground conditions and construction methods. Systems covered include steel piled walls, concrete piled walls and diaphragm walling.

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85.1 Introduction

The principal function of embedded walls is to safely enclose space within the ground to an agreed watertightness, although some embedded walls are used simply as groundwater or leachate barriers. The UK with its range of geology (and its industrial heritage) employs and has contributed to the development of almost all embedded wall types.

The choice of embedded walling type is dependent upon a number of factors including the following:

- ground;
- groundwater;
- proposed depth;
- site logistics;
- economy (including programme economy);
- wall finish and watertightness.

The 'generic' sketch (**Figure 85.1**) indicates the major issues that need to be considered when choosing a wall type (for further guidance, see C580; Fernie Suckling GE; Sperwall ICE).

Walls in the UK seldom fail structurally – a great deal of discussion, debate and hard finance is spent on their aesthetics above the dredge, their contribution to local ground deformation and the effect of their placement and exposure on adjacent structures and particularly their 'watertightness'.

It is often the issue of watertightness that governs the ultimate judgement of the success of the wall, yet to date no agreed qualitative method of assessing this issue exists between users and providers. The publication *Reducing the Risk of Leaking Substructure – A Client's Guide* and the further references within it, is recommended reading for those involved in any aspect of embedded walling.

If walls are judged post-construction on aesthetics and watertightness they appear to be procured almost purely on the basis of economics; **Figure 85.2** tries to indicate 'relative' costs for the various wall types and schematically illustrates where wall types are employed. The schematic indicates how difficult it is to generalise likely costs; at the polarised ends, small schemes for canals and temporary works for motorway cuttings are economically provided for by sheet pile walls but deep shafts in difficult ground require diaphragm walls.

This chapter covers embedded walling by considering the following wall types:

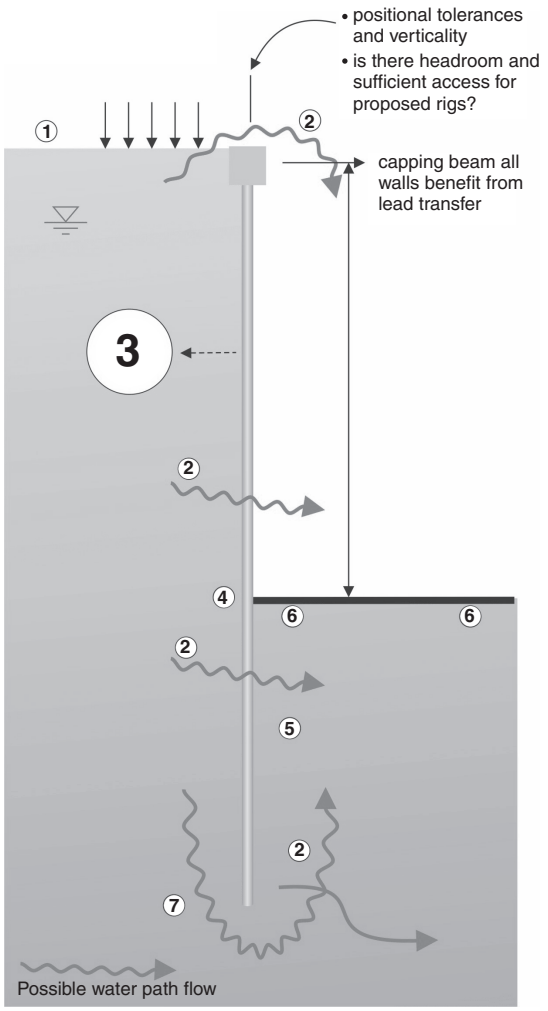
- diaphragm walls;
- piled walls;
- steel walls;
- hybrid and other walls.

Each section addresses the generic issues raised in **Figure 85.1** and indicates how the walls are constructed, the range of their application, their strengths and which particular interfaces need closest attention.

85.2 Diaphragm walls

Diaphragm walls are embedded concrete walls, usually reinforced, formed using the ground as a shutter. The process is illustrated in **Figure 85.3**. To ensure that the ground does not collapse the panel is supported by a fluid suspension – normally a bentonite or polymer suspension. Discrete 'panels' are excavated and supported by the fluid and at their ends by 'stop-ends'. The bentonite support fluid suspends materials not removed by the excavation and cutting process and it requires cleaning via a replacement or recycling process. Reinforcement cages are placed. The fluid suspension is then displaced by specially designed concrete mixes with sufficient consistency to flow to the panel extrados and 'scour off' any bentonite from any element (stop-ends, reinforcement, box-outs, etc.) within the panel.

As illustrated in **Figure 85.2** diaphragm walls tend to be used on larger, deeper walling contracts in more difficult ground – walls have been constructed to 150 m depth and also to 2.5 m thickness, although more commonly wall thicknesses of 800 mm, 1000 mm, 1200 mm and 1500 mm are used. They can be built in nearly all ground conditions, but special works may be necessary in very soft ground (Su < 20 kPa) and hard ground (lightly fissured rock with an unconfined compressive strength



Some Major Issues

① Do adjacent services or structures inhibit access or apply constraints?

② Groundwater leakage possible via paths shown. The greater the number of interfaces/joints, the greater the problem. Walls are often ultimately judged on perceived watertightness.

③ The ground is the major influence on the choice of wall type. Can you drive, bore, drill or jock in wall elements? Is the ground stable? A soil or rock or a melange.

④ How easily are joints effected? How necessary are walings, props, anchors, plates, head details and any connections?

⑤ Depth to dredge dictates whether can be cantilebered or whether it needs props or berms. Depth below dredge can depend on cut-off, bearing and hold down requirements as well as stability.

⑥ Ground at dredge critical to stability, movements, water considerations and connections from wall to base slab.

⑦ Can wall depth be acheived by 'standard' construction methods? Need toe be pinned/castellated/curtailed?

General

- How does temporary and permanent works interface and impact on the wall elements? Is this fully appreciated by all parties?
- Can predicted movement be accommodated or sensibly influenced?
- How important are finishes and aesthetics?

Figure 85.1 The major issues that need to be considered when choosing a wall type

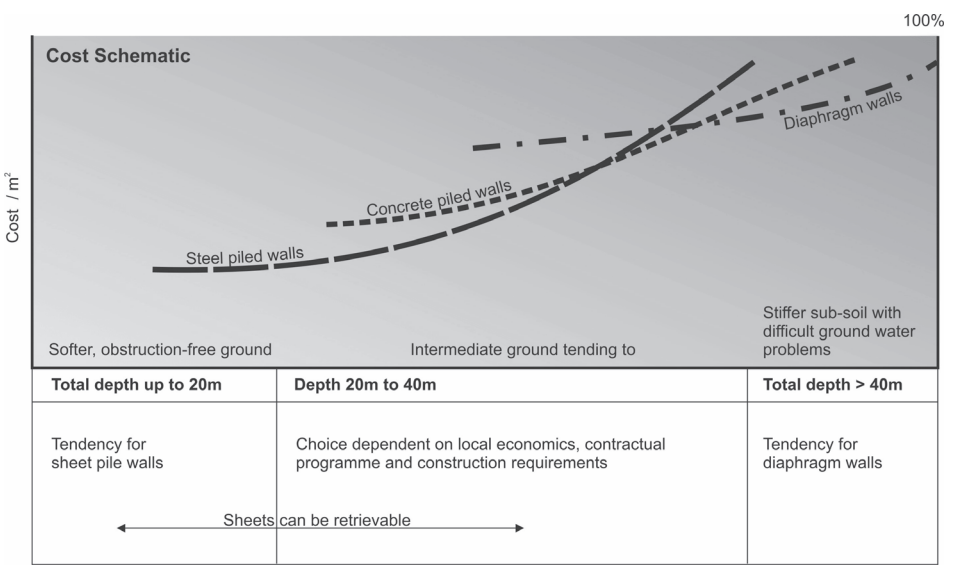


Figure 85.2 'Relative' costs for the various wall types and where wall types are employed

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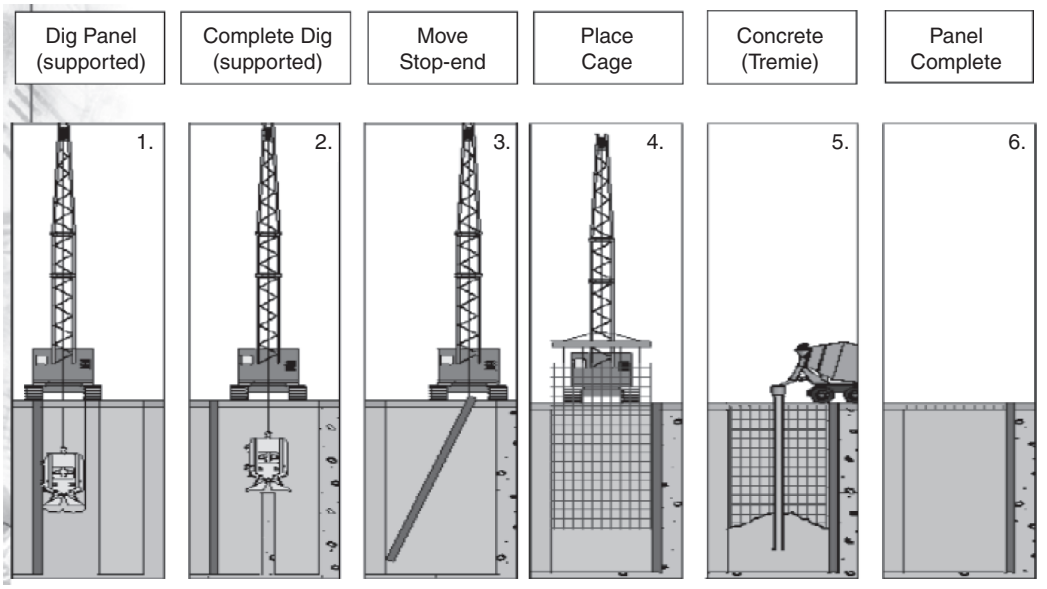


Figure 85.3 Diaphragm wall construction

greater than 20 MPa). Construction techniques can be readily configured to produce circular or near circular shafts of diameter greater than 8 m and panels in various shapes can efficiently carry large imposed loadings. Introduced to the UK in the 1970s by ICOS, diaphragm walling has grown with the tendency for deeper basements and deeper shafts. Extended motorway and metro troughs of up to 1 km in length, shafts 40 m in depth and basements 30 m in depth have been constructed in the UK in recent years. Since diaphragm wall panels are typically 6 m in length (range 2.8–7 m), they have fewer joints (which can also be fitted with water bars) than other embedded walls and are favoured in ground where the control of groundwater, seepage and damp is necessary. It must be noted, however, that no better than Grade 1 watertightness (see Chapter 64 *Geotechnical design of retaining walls*) should be assumed possible for any concrete embedded wall. In western Europe and its influence zones, diaphragm walling seldom provides structural continuity across panel joints. Such continuity is more common in the Far East at a considerable cost premium.

As with most embedded walling expertise, quality workmanship and close cooperation at the interfaces is necessary for successful construction. There is useful guidance in *BS EN1538 Diaphragm Walling*.

Excavation for diaphragm wall panels is usually carried out using grabs or hydromills (cutters or hydrofraise); see **Figure 85.4**. These are dedicated pieces of plant, which experienced diaphragm wall specialists are reluctant to modify since any change affects the process. Designers for economy are encouraged to work with what is available rather than what is optimum. Grab and cutter widths are typically 2.8 m. Grabs (cable or hydraulic) were used in early developments and were used particularly for softer cohesive ground. A typical excavation rate for a grab is 100 m^2/shift. Modern grabs are fitted with instrumentation so that depth and deviations can be monitored in real time and recorded (particularly where rotating head Kelly bars are fitted). Specification requirements of 1 in 75 verticality tolerance can be bettered using such techniques. Grabs are usually employed to start excavation works when mills are to be used.

Mills (cutters or hydrofraises) are hydraulically powered drum wheel excavators, which remove soil via slurry, which also acts as trench support. The process uses reverse circulation with the spoil transport medium in a continuous circuit abstracted by a mud pump, fed by pipework to a separation plant (desanders, desilters) where the solid materials are removed, to be pumped back to the panel again (see **Figure 85.5**). Developed initially for cohesionless soils and weak rocks, excavation rates were typically up to five times that of a grab. However, at this rate of excavation in a soft cohesive soil, it is difficult to provide an efficient cleaning or replacement cycle to remove the finely graded soil material from the support fluid. There has been much use of mills in recent times in stiffer, carbonated and overconsolidated clays and in stronger rocks. Verticality tolerances of better than 1 in 1000 have been achieved using mills but contractors tend to limit their contracted tolerances to 1 in 300. In addition, contracts usually allow for protrusions of not greater than 100 mm from the 'true wall face'. This criterion is based on the experience of many contracts but the truth is that it is controlled by the ground. In loose cohesionless or soft cohesive soils, the overbreak could be much greater, contrasting with overconsolidated clays where the finish can be smooth and dense.

With the correct tooth configuration on the cutter wheels, mills can cut into the 'green' concrete of an existing panel, forming a 'continuous' concrete member (still without rebar continuity). This can be particularly useful for deep shafts

(a)



(b)



Figure 85.4 (a) Hydraulic grab and (b) hydrofraise

(a)



(b)



Figure 85.5 (a) Desanding equipment and (b) cage lifting

(circular or ellipsoidal) where depth precludes the use of other methods of achieving good quality joints between panels. However, this form of overcut joint has been found to have rather dubious watertightness and, hence, its use should be restricted to structures where panels tend to be in hoop compression or where watertightness is not considered to be important. The choice between using a mill or a grab is usually a straightforward comparison between productivity and the difference in set-up costs (mills being more expensive), although site access, logistics, ground conditions and environmental considerations can be relevant.

Diaphragm-wall ancillary equipment includes heavy chisels to aid excavation through rock and boulders, brushes to clean panel joints, service cranes for cage installation and concreting and mud mixing and separation plants.

As noted, diaphragm walls are often employed where groundwater and soft soils are present and, hence, trench stability is critical. In these circumstances, stability is designed for by limiting panel length – thereby encouraging arching effects and by ensuring a positive head in the support fluid with respect to the groundwater level (usually not less than 2 m). This positive head has to be maintained throughout the process and is ensured by harnessing a reservoir or the containment effect of the guide walls. The panel size will also be important in terms of ground movement with larger panels tending to

develop larger ground movements; however, they can be easier and more efficient to prop. The converse is true of smaller panels, which have less inherent movement but greater difficulty in propping. Guide walls have several uses: they act as positional guides for the excavation tools, they indicate the positive fluid support level that must be maintained, they act as a reservoir for this level and they act as a support for (the often heavy) down-hole insertions such as stop-ends, tubes and reinforcement cages. Guide walls are seldom less than 1 m deep and 300 mm thick. Occasionally existing adjacent basement walls are employed as one side of a guide wall.

The specific gravity of freshly mixed bentonite slurry is about 1.02 to 1.04 and with water at 1.0 it is not surprising then that water has occasionally been used as a support medium. Bentonite has the added property of thixotropy and can form a filter cake on the trench wall in many soils, thus, enhancing panel stability by preventing side sloughing. Polymers, where the long polymeric chains clog up the ground pores, are fairly commonly used for bored pile support, but have not been used successfully in diaphragm walling in the UK. The science behind these polymers is only now being developed and the ENs recommend their use only after extensive exhaustive trials or direct comparable experience. Increasingly bentonite support fluids with polymer additives are being utilised.

Early diaphragm walling employed circular pipe stop-ends, which were extracted clear of the 'green' concrete as part of the panel formation process. Circular pipes gave way to 'organ pipe' stop-ends, which were still pulled some several hours after the last concreting. Where panels were very deep or working hours restrictive permanent steel (H-sections) or precast concrete stop-ends were employed.

Currently most practitioners use 'peel off' stop-ends. These are prised off the end of the hardened concrete panel once it has been exposed during excavation of the adjacent panel. They can incorporate a water stop, which will extend into adjacent panels. This system has greatly facilitated diaphragm wall programming. However, experience of successfully placing and removing such stop-ends is limited to about 50 m depth. Accurate and vertical placing of stop-ends is a vital part of the process and if high standards are not achieved then subsequent panel joints and exposed wall surfaces can be seriously compromised.

Diaphragm-wall tool sizes are such that multi-returns and complex wall shapes are not encouraged. Panel dimensions are dependent on plant size, and stability considerations both for the panel and for adjacent structures. The logistics are also important, since excavated panels soften, and common practice is usually to concrete quickly and handling much more than 200 m^3 to 300 m^3 of concrete within a normal working day is often difficult. However, panel volumes up to 1400 m^3 have been cast in the UK. Wherever possible, practitioners will promote simple rectangular panels, although corner panels must be accepted. T-, Z- and I-shaped panels can offer significant structural benefits in terms of stiffness and moment capacity but it must be recognised that they are more difficult to construct. Here, as in the case of corner panels, continuity of reinforcement across changes in panel direction should be avoided if at all possible. However, in the case of re-entrant corners, continuous cages may be necessary for structural reasons.

Two examples of diaphragm walls are shown in **Figure 85.6**.

The tendency for deeper walls, carrying bigger loads over larger spans is leading to heavier densities of reinforcement in diaphragm-wall panels. The need to incorporate connection box-outs and wall cut-outs can exacerbate problems. Panels have construction requirements (tremie space, clearance to stop-end and water-bars), which constrain reinforcement cages. These tendencies act against the need for open, accessible panel faces to facilitate concrete movement, to ensure penetration, to ensure scour of slurry or filter cake deposits and to ensure dense concrete cover. Although EN1538 indicates a minimum clear vertical bar space of 80 mm may be allowed in special cases, the authors of this chapter believe that 100 mm

(a)



(b)



Figure 85.6 (a) Deep shaft (France) and (b) 'peanut-shaped' (UK) diaphragm walls

should always be attempted. ICE (2007) C8.4.4 provides more detailed recommendations and differentiates between vertical and horizontal reinforcement. It is extremely important that discussions take place between the wall constructor and designer well in advance of cage fabrication (ideally prior to commencement of detailing) to ensure that buildability is not compromised.

The challenges faced by the constructor regarding the lifting and handling of reinforcement cages must not be overlooked. Issues such as transportation to site, safe lifting, on-site welding of reinforcement and cage splicing must all be discussed at an early stage and the appropriate risk assessments and temporary works design carried out.

All embedded concrete walls require special 'flowable' retarded concrete mixes, which can be placed reliably via tremie pipes and which self-compact without the need for vibrators or mechanical means. It should be noted that such concrete has evolved over decades to meet the specialist demands of deep-foundation contractors. The mixes are not the expensive 'self-compacting' mixes generally sold to the construction industry.

Diaphragm walls with their flat surfaces can easily be 'joined' or connected to adjacent structural elements (props, walers, beams, slabs). Box-outs in the form indicated are preferred in deep walling for such connections and these can be placed to within +150/–50 mm of the design elevation. Whereas simple pull-out bars will only provide very limited moment fixity and are used generally to aid transfer of shear, the use of couplers can provide a high degree of moment fixity. Alternatively, connection bars can be drilled and grouted into the wall face on exposure during bulk excavation. The joint between a diaphragm wall and the base slab (or roof slab in the case of cut-and-cover tunnels) is most likely to be subject to groundwater or water infiltration and needs particular and serious attention in its detailing.

Diaphragm walls may be lengthened (deepened) where necessary to increase the flow path beneath the toe of the wall or where they are employed as vertical load carrying elements as well as acting as retaining walls. It is not commonly required to reinforce walls below the depth required for embedded wall stability in such cases. There are also circumstances where the ground is too hard and works cannot be economically progressed to the required 'design' depth. Such cases are often resolved by taking a series of pins through the panel into the hard ground below, ensuring lateral support at the toe. This is usually foreseeable at the design stage and steel reservation tubes can be provided in the cage for this purpose as well as providing locations at which toe grouting can be carried out to significantly reduce rock permeability.

Diaphragm walling contracts need large dedicated plant, which usually necessitates sole occupancy of the site. Bentonite plant (silos, mud pumps and cleaning or desanding equipment) typically requires 150 m^2. As noted above, it is often the logistics of the mud plant that controls overall planning of the diaphragm walling activity.

Joints are inevitably 'leaky' and even with best practice can be areas where bentonite or ground inclusions are trapped. Contractors will make good areas with inclusions and will contract to exclude 'running leaks' – as stressed in the introduction this particular issue needs much upfront discussion and a clear understanding of what is likely to be achieved in a particular ground.

Diaphragm walling continues to be pushed to deeper and longer spans and to incorporate higher densities of reinforcement and ancillary inclusions (box-outs, soft eyes for tunnel connections, geothermal pipes, instrumentation tubes, sonic coring tubing, etc.). It is a technique that is likely to grow in application.

Another example of the construction of diaphragm walls is shown in **Figure 85.7**.

85.3 Secant pile walls

85.3.1 Description and use

Secant pile walls are formed from a series of intersecting bored cast *in situ* concrete piles. In most cases alternate 'male' (secondary) piles are reinforced to provide the structural capacity of the wall with the intervening 'female' (primary) piles acting as unreinforced barriers to groundwater flow (**Figure 85.8**). As well as providing retention of soil and groundwater pressures, secant walls can be used to carry superstructure loads. Considerable flexibility in plan shape is offered by secant pile walls (**Figure 85.9**), which leads to efficient use of the basement space. Hence, secant pile walls are a popular form of embedded retaining wall in building projects as well as civil engineering structures such as metro stations, underpasses, cut-and-cover tunnels and pumping stations. Circular and elliptical shafts (**Figure 85.10**) have also been constructed successfully using secant pile walls.

Constructed in the 1970s using mechanically configured rigs, walls were relatively expensive and slow to install. However, the advent of powerful hydraulic piling rigs increased both the range of application and the rates of production. By the mid-1980s, deep secant pile walls were being constructed as permanent walls. Later that decade hard-hard secant pile walls were constructed at a number of sites in the UK using the continuous flight auger (CFA) method. The most recent developments have seen the use of cased continuous flight augers, which are able to combine the benefits of high productivity through using a continuous flight auger with the tight construction tolerances obtained using temporary casing.

Some confusion exists in the terminology used to differentiate between the types of secant pile wall. The key constant in all walls is that the male piles are nearly always reinforced structural concrete and the variable is usually the strength of the female piles, which will typical vary as follows:

Hard/soft walls – this wall type is often used in temporary works or where the long-term durability of the female pile can be neglected as its function is taken over by some other form of construction such as an internal structural liner wall. Female

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(a)



(b)



Figure 85.7 Station box for Channel Tunnel Rail Link (UK)

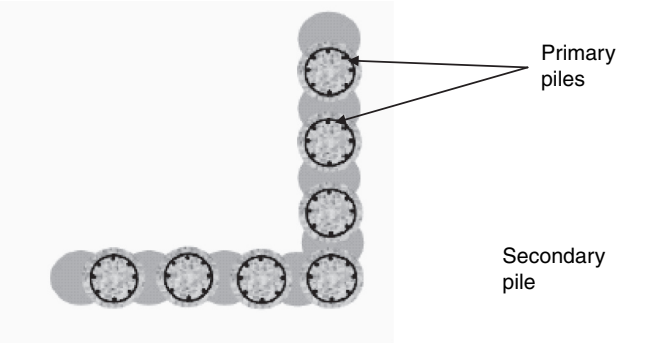


Figure 85.8 Primary–secondary pile sequence

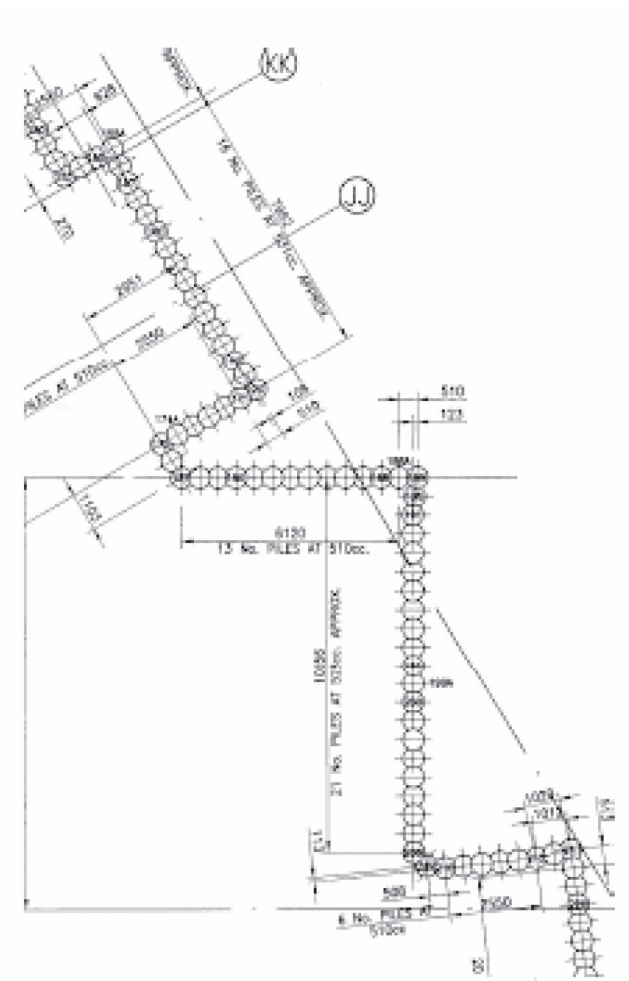


Figure 85.9 Flexibility in plan shape

piles are designed simply as short-term water blockers using a cement/bentonite mix or similar (**Figure 85.11**).

Hard/firm walls – this wall type is often used for basement walls for building projects and constructed usually using either the CFA or cased CFA technique. Female piles are designed to suit the design life of the structure but with a close eye on constructability. There is sometimes a perceived conflict here between the minimum cement requirements to ensure durability and the difficulty encountered when overcutting female piles of excessive strength. Female piles are often designed to achieve a strength of C8/10 at 56 days.

Hard/hard walls – these walls are usually constructed using segmental casing and intended to act as permanent works with

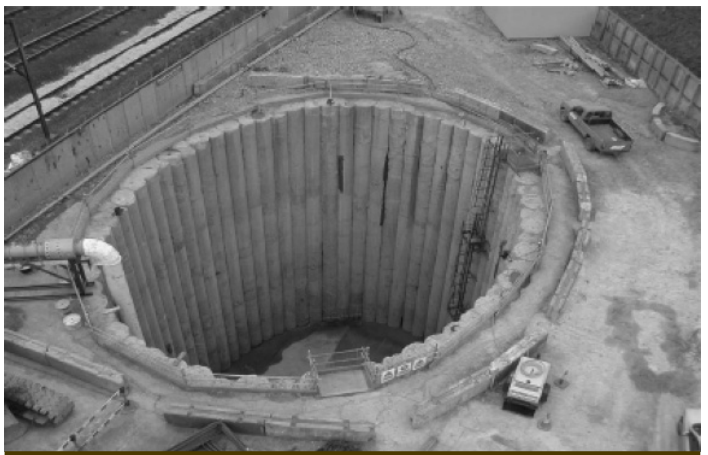


Figure 85.10 Ellipsoidal-shaped shaft



Figure 85.11 Hard/soft secant pile wall

Pile diameter (mm)	Bore diameter below casing (mm)	Minimum overcut (mm)	Maximum overcut (mm)
a. Secant pile walls constructed using the continuous flight auger method			
600	-	100(1)	250(1)
750	-	100(1)	250(1)
900	-	100(1)	250(1)
1050	-	100(1)	250(1)
b. Secant pile walls constructed using the cased continuous flight auger method			
750	-	100(1)	250(1)
880	-	100(1)	250(1)
c. Secant walls constructed using segmental casing			
620	520	100(1)	250(1)
750	670	100(1)	250(1)
880	760	100(1)	100(1)
1000	900	100(1)	250(1)
1180	1060	100(1)	250(1)

(1) Typical values that depend on the concrete mix, ground conditions and retained height.

Table 85.1 Range of pile diameters

equal durability of both male and female piles. Female piles are usually unreinforced but are constructed with a full structural concrete mix typically either C28/35 or C32/40.

85.3.2 Range

The increased use of secant piles has been driven by the development of ever more powerful hydraulically powered rotary drilling rigs with torque in excess of 500 kN·m. However, the range of application varies significantly depending on the installation method.

It is difficult to provide hard and fast limits on the depth achievable with each technique since this is dependent on many factors including rig power, pile diameter and ground conditions. However, in general whilst it is possible to construct CFA piles to depths in excess of 25 m, cage depths are usually limited to about 18 m. Cased CFA pile depths are currently limited to about 18 m and cased secant piles are rarely constructed beyond 30 m depth. It is important to remember that the depth over which the piles will actually be secanted will be determined by tolerances and not by the depth that the piling rig can drill (see **Table 85.1**).

85.3.3 Installation and materials

Piles are constructed at a predetermined spacing designed to ensure that the piles will intersect over the required height of the wall allowing for the combined positional and verticality tolerances of both piles. The provision of reinforced concrete guide walls (**Figure 85.12**) is essential in order to maintain good positional tolerances at the piling platform level. The scalloped profile of the guide walls is usually achieved using special formwork or pods, which are adjusted to achieve the desired pile layout.

Female piles are constructed first at an initial spacing such that there is no risk of interference between recently constructed piles as a result of plant track or concreting pressures. For example, this may require a pile construction sequence of pile numbers 1, 5, 9, 13, 17 then 3, 7, 11, 15. For hard/hard and hard/firm walls, male piles are ideally constructed within three to seven days of construction of the adjacent female piles. This is particularly important for walls constructed using the CFA method where the excessive concrete strength of the female piles can cause severe verticality tolerance issues during male pile installation; male piles actually pop out of alignment at depth leading to watertightness and space-proofing problems. Care must be taken not to open up too much of the wall by constructing an excessive number of female piles without the

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Figure 85.12 Guide walls



Figure 85.13 Cased CFA wall piling with spoil management system

intervening male piles. In addition, in the case of phased construction it may prove necessary to install a gravel pile at the end of a phase by overcutting the last female pile whilst it is still green and backfilling it with gravel; the overcut can be achieved several weeks later if necessary as the phases of the works become joined.

The productivity of CFA and cased CFA rigs can exceed 10 piles per shift and, therefore, it is necessary for continuous sections of wall in excess of 50 m in length to be available for these rigs to work efficiently.

Whilst excavation of CFA piles takes place under the support of the soil on the flights of the auger, cased CFA utilises a stiff temporary casing to support the ground and overcut adjacent female piles. Air is supplied down to the auger tip by a nozzle in the concreting pipe via the hollow auger stem, to assist flighting of the drilling spoil up the auger. At the top, the spoil is directed into a series of telescopic muck bins, which transfer the drilling arisings safely to earth (**Figure 85.13**). The casing and continuous flight auger are powered independently by separate rotary drives.

The guidelines for the reinforcement of diaphragm walls cited in section 85.2 are in many areas equally applicable to secant pile walls. Particularly relevant is the observance of minimum clear bar spacings of 100 mm. Consideration to the pairing of bars should be given to ease congestion. Special attention should be paid to congestion at overlaps and the use of couplers may be required if there are two layers of reinforcement. Furthermore, the cranking of larger diameter bars, such as 40 mm or 50 mm, may not be feasible.

Additional cover (100 mm rather than 75 mm) may be provided in order to facilitate the installation of long reinforcement cages in CFA and cased CFA secant piles. The slight resulting loss of structural capacity is normally recouped by the improvements in productivity (or lack of re-drilled piles due to stuck cages).

Concreting of CFA pile walls follows similar practices to those required for deep CFA bearing piles. Special care should be paid to concrete workability and the longevity of fresh mix properties for cased secant piles to avoid common faults such as cage displacement, stuck temporary casings and damage to green concrete during casing removal.

85.3.4 Tolerances

There are a number of essential ingredients to achieve good construction tolerances in secant pile walling:

- design and construction of robust guide walls;
- selection of appropriate plant and experienced personnel;
- careful sequencing of the pile installation;
- appropriate materials, especially female pile concrete mix design;
- attention to detail in designing the pile layout.

Wall installation tolerances are heavily dependent on the construction method. *Specification for Piling and Embedded Retaining Walls* (ICE, 2007) gives some useful guidelines in Table C9.1. Section B9.4 of this specification gives further guidance on guide wall and reinforcement cage tolerances.

85.3.5 Interfaces

Unlike diaphragm walls, it is not possible to install slab connection reinforcement or box-outs in secant pile walls without excessive risk that they will not be located within the required tolerances. Reinforcement cages installed either by plunging into fresh concrete or by installing prior to concreting within a temporary casing will usually become significantly twisted out of position particularly at depth. As a result, it is usually preferable to achieve slab-to-pile connections by drilling and grouting in bars at the required level.



Figure 85.14 Hard/hard cased secant pile wall, Royal Opera House, London

Secant pile walls that are supported by temporary props will require waling beams, which must be properly packed to achieve contact between the waler and every pile to ensure effectiveness. In many cases temporary props are arranged to support the wall via a top capping beam, which may be required anyway, hence, saving the need for a temporary waling beam. When temporary anchors are used, the provision of a waling beam can be avoided if every female pile is restrained by an anchor using an anchor head block (**Figure 85.14**).

Waterproofing details for secant pile walls require particular attention given their geometrical shape at the slab-to-wall connections and also at the capping level. The provision of re-injectable trays for permanent structures can be beneficial provided access is available in the long term.

85.4 Contiguous pile walls

85.4.1 Description and use

Contiguous pile walls consist of a series of cast *in situ* bored piles, which do not intersect with each other. The gaps between the piles are usually sized so that, at a minimum, the piles do not clash at depth although they can be larger, dependent upon the ability of the ground to arch between the piles; typical gaps are around 100–200 mm but can be bigger.

These walls are designed to resist earth pressure but are generally not designed to be groundwater retaining, which means some form of drainage detail is necessary at the formation level of the proposed structure. For aesthetics, the walls are often faced with a variety of finishes, e.g. panels, tiles, bricks or blocks. To protect both the drains and facing from ground sloughing in the gap, it is good practice to infill this area with a free-draining material, such as no-fines concrete or a propriety geofabric or plastic drain.

Contiguous bored piled walls are most often employed in low-permeability ground. Despite being less structurally efficient than diaphragm walls and suffering from the interface problems common to secant pile walling, they can offer a very economical solution and walls have been constructed in a range of diameters typically 600 mm up to 2400 mm – see **Figure 85.15**.



Figure 85.15 Contiguous bored piled wall

85.5 Sheet pile walls

85.5.1 Description and use

Sheet pile walls consist of a series of interlocked steel sections, which when installed are capable of withstanding both horizontal and vertical loads. Historically sheet pile walls were utilised in temporary situations where, for example, support was required during an excavation, whilst the permanent works were constructed. Advances in both the manufacture of sheet piles and installation methods now mean that steel sheet piles are used as commonly in permanent retaining solutions as for temporary support measures. Common uses for sheet pile retaining walls include:

i. *Steel-intensive basements* – Sheet piles are used as the permanent retaining structure and can also carry the vertical perimeter structural loads. The exposed face of the wall is often left visible and in these cases the steel will generally be cleaned and painted to provide an aesthetically acceptable product (**Figure 85.16**). Further details on steel-intensive basements can be found in *Steel Intensive Basements* (Yanzio and Biddle, 2001).

ii. *Flood defence and quay walls* – Sheet pile walls are used in rivers, inland waterways, ports and harbours to provide both protection against flooding and berthing for vessels. As vessel size has increased over recent years there is a need for deeper berths. There are limits to the capacity of a standard sheet pile wall and as the retained height

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increases it is more likely that the wall will be constructed as a combi-wall where stiffer primary elements are inserted between a number of sheet piles. This technique is covered in further detail in the next section.

iii. *Retaining walls* – Sheet piles are commonly used along highways and in other situations where slopes need to be supported. Generally the piles act in cantilever where there are lower retained heights. For higher walls, other methods can be incorporated into the wall construction to allow greater capacity. The wall can be anchored using soil or rock anchors, or if space is available in front of the wall then waling beams and props can be installed.

85.5.2 Plant

The plant and equipment used for the installation of sheet piles has developed significantly over the last 30 years. Panel driving was the most common method historically; however, this has largely been superseded by rig-mounted hydraulic vibrators and pressing techniques on a pitch-and-drive basis.

The power available in rig-based units has increased as has the size of the rig itself. A modern leader rig equipped with a vibratory hammer can now weigh in excess of 60 tonnes and be equipped with a hammer with a centrifugal force of 1400 kN; only 20 years ago a state-of-the-art rig would have weighed closer to 40 tonnes and its hammer would have had a maximum centrifugal force of approximately 600 kN. This has allowed the range of applications where sheet piles are the most suitable retaining solution to be extended, as heavier sections can be handled and driven in ever more difficult soils. The design of the vibrators has also changed during this period with most equipment now operating at higher frequencies, which means that any vibration imparted into the surrounding soils attenuates much more quickly, leading to less risk of damage to adjacent structures (**Figure 85.17**).



Figure 85.16 Basement construction utilising steel sheet piles and temporary propping

Where sensitive structures are present and noise and vibration needs to be limited, then there are now a range of hydraulic pile presses available, which either use the reaction supplied by previously installed piles or a large base machine to push the sheet pile into the ground. The use of these techniques is more sensitive to the soil type encountered but can be assisted in some circumstances by pre-boring to loosen the soils along the pile line or by water jetting during pile installation (**Figure 85.18**).

Where long sheet piles are required or very accurate installation tolerances need to be achieved, then conventional methods allied to panel-driving techniques will be most appropriate. With these methods, large crane-suspended vibrators and impact hammers can be used to impart high-driving forces into the sheet piles whilst controlling the position and alignment within a set of guides and piling gates.

85.5.3 Materials

Sheet pile walls can be manufactured in either cold-rolled or hot-rolled sections. The use of cold-rolled sections is generally limited to lightly loaded situations, as they are generally of a lighter section and do not interlock as tightly as hot-rolled sections.



Figure 85.17 Sheet pile installation utilising leader rig with hydraulic pile press



Figure 85.18 Kowan hydraulic pile press

The tolerances for the manufacture of cold-rolled sheet piles is covered in BS EN10249 (British Standards Institution, 1996b). The inherent limitations of cold-rolled piles mean that they are more susceptible to de-clutching if the pile is installed inaccurately or in denser more difficult soils. The manufacture of hot-rolled sheet piles is covered in BS EN10248 (British Standards Institution, 1996a). Hot-rolled sheet piling of non-alloy steels and the manufacture to tight tolerances is one of the main advances in pile manufacture in recent years. The provision of a tight interlock allows for their use in more heavily loaded situations with more difficult ground conditions. There are a large range of sheet piles available on the market in both U- and Z-shaped profiles. Profile selection depends upon the design requirements. However, from an installation point of view it is important that the design takes account of the driving conditions. It may be that a design and selected pile section can be completely sufficient for the permanent applied loads, but would fail during installation due to the temporary loads applied by driving. In these cases a heavier pile section would be selected to accommodate the driving forces. The driveability of a sheet pile section is related to its cross-section, stiffness, length, steel grade, quality and the chosen method of installation. Further details on the factors affecting driveability can be found in the *Piling Handbook* (ArcelorMittal, 2005), along with detailed product information regarding their manufactured sections.

85.5.4 Installation

The method of installation will vary depending upon the type of wall, the selection of pile section and physical site characteristics. However, the method will generally fall into one of the following categories:

85.5.4.1 Pitch-and-drive

This methodology utilises specialist equipment to pick up and pitch-and-drive single piles to the required depth. Originally this technique was limited to situations where the soil was loose and the piles short. In recent years the equipment has become more powerful and larger allowing the technique to be used on more complex sites with more difficult soils. Care needs to be taken to ensure that the piles remain vertical during installation as the lack of any guidance frame can allow the piles to lean in the direction of driving or rotate, particularly in stiffer soils or where driving longer piles.

85.5.4.2 Panel driving

With this methodology a number of piles are inserted into a pre-constructed frame located along the required wall line. Three basic driving techniques are used:

85.5.4.3 Impact driving

The pile is repeatedly hit with a falling mass travelling over a controlled stroke length. The method can generate high levels of low-frequency vibrations and high-noise levels. The advantage is that, given a variety of hammer weights and stroke lengths, high levels of energy can be put into the piles overcoming most soil conditions.

85.5.4.4 Vibrodriving

Piles are installed using high-frequency vibration from sophisticated and powerful hammers, which can overcome most problematic soil types. However, the hammers are expensive to operate and repair compared with impact hammers.

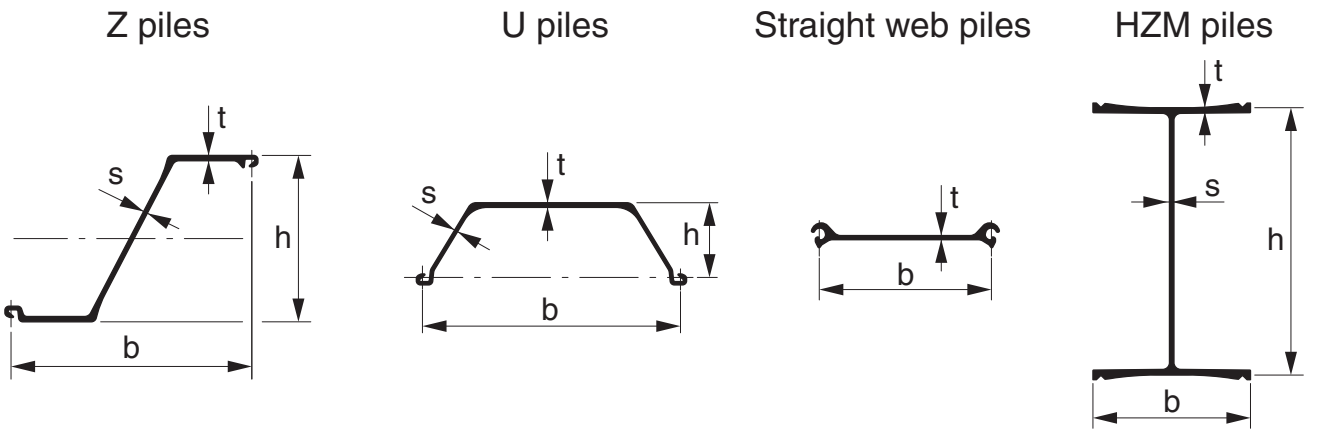
85.5.4.5 Pressing

With pressing, installation in granular soils can be limited unless assisted by other methods such as water jetting or preboring. The major advantage is that equipment generates low levels of noise and vibration and can be used near to existing structures.

85.5.5 Tolerances

Installation tolerances have been established for many years and are summarised in a number of documents including the *Specification for Piling and Embedded Retaining Walls* (ICE, 2007), the *Piling Handbook* (ArcelorMittal, 2005) and in BS EN12063 (British Standards Institution, 1999). There is a movement within the piling industry to review these tolerances, as much tighter accuracy is now possible and can be offered where there is a structural need for piles to be installed to within say 25 mm of plan position as opposed to the standard specification of within 75 mm. As sheet piles are a manufactured product it is also important to consider manufacturing tolerances when installing a sheet pile wall (see **Figure 85.19**). For example it is not uncommon for a standard 600 mm wide U-section to actually measure 606 mm and as such be within the specified manufacturing tolerances. However, at site level this would mean that for every 100 piles one less pile would be installed than anticipated from the theoretical layout.

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Height				Width	
				Single piles	Interlocked piles
Z piles	h 200mm ±5mm	200mm < h < 300mm ±6mm	h 300mm ±7mm	± 2% b	± 3% nominal width
U piles	h 200mm ±4mm	h > 200mm ±5mm		± 2% b	± 3% nominal width
H piles	h < 500mm ±5mm	h 500mm ±7mm		± 2% b	± 3% nominal width
Wall thickness					
Z piles	t 8.5 mm ± 0.5 mm	t > 8.5 mm ± 6 %	s 8.5 mm ± 0.5 mm	s > 8.5 mm ± 6 %	
U piles	t 8.5 mm ± 0.5 mm	t > 8.5mm ± 6 %	s 8.5 mm ± 0.5 mm	s > 8.5mm ± 6 %	
H piles	t 12.5 mm +2.0 / -1.0 mm	t > 12.5 mm +2.5 / -1.5 mm	s 12.5 mm +2.0 / -1.0 mm	s > 12.5 mm +2.5 / -1.5 mm	
Straight web piles	t 8.5 mm ± 0.5 mm	t > 8.5mm ± 6 %			
All sections					
Straightness	**Length**	**Squareness of cut**	**Mass**		
0.2 % of pile length	± 200 mm	± 2 % b	± 5 %		

Figure 85.19 Manufacturing tolerances for steel sheet piles
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85.5.6 Interfaces, joints and connections

One of the major benefits of utilising a sheet pile wall is that each element is manufactured in factory conditions and is supplied with a designed connection detail, which is capable of providing the full section modulus. This interlock is complemented by a series of special sections, such as 'corners and junctions', which allow a sheet pile wall to be installed in numerous configurations. Some of the more common corner products available are shown in **Figure 85.20**.

With all wall types the issue of watertightness is most critical at the location of joints and interfaces. For sheet pile walls where watertightness is important, a number of methods can be utilised to exclude or minimise water penetration. There are a number of clutch sealants on the market, which can be placed within the clutch prior to pile installation. However, these products usually provide resistance to prevent water ingress in a temporary situation. When permanent exclusion of water is required it is more common to weld the clutches after installation, during basement construction. This provides a permanent seal, which will not degrade with time. As the seal is created post installation, it can normally only be carried out over the exposed length of pile, from the surface level down to the formation level of the basement. This means that some sort of flange will need to be installed linking the sheet pile wall with the basement slab to ensure that water cannot penetrate through the base. As sheet pile walls are made from steel, there

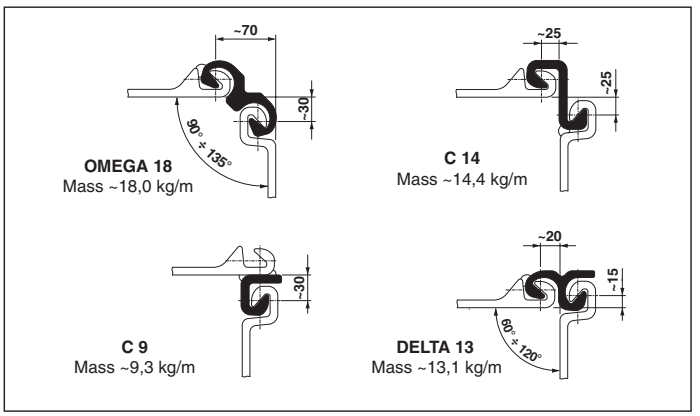


Figure 85.20 Range of corner profiles used to change wall direction
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Figure 85.21 Detail of floor/wall connection in a permanent steel basement

are more possibilities for carrying out post-installation work to improve or maintain watertightness, such as welding at clutch locations or installing additional steel products over damaged joints. **Figure 85.21** shows a typical steel flange connection at the base of a permanent sheet-pile basement construction.

85.6 Combi steel walls

85.6.1 Description and use

Many of the issues relating to combi steel walls are closely related to the previous section on sheet pile walls. The main difference is that the design approach relies upon any load being retained on individual primary elements, which are separated by infill panels (secondary elements), usually consisting of steel sheet piles. As the retained height is generally larger, much larger plant and equipment are required to install the heavier sections. The table below gives some common configurations for various combi steel walls and is taken from Figures 85.1 to 85.5 of BS EN1993–5 (British Standards Institution, 2006), which details many of the requirements for combi steel piling.

Common uses for combi steel pile retaining walls include:

- deep steel-intensive basements and cofferdams;
- flood defences and quay walls;
- retaining walls.

85.6.2 Plant

Most combi steel walls will be installed using traditional crane-hung vibratory and impact hammers due to the size of the wall elements (**Figure 85.22**). As the elements are relatively large and, thus, heavy, greater amounts of energy are required to drive the piles to the required depth. A further advantage of using crane-hung equipment is that when working near water the heavy plant and equipment can be set back from the edge, which, therefore, minimises any surcharge loading on the unprotected slopes.

In some cases, such as deep basements within city centre locations, it may not be possible to use either vibro- or impact-driving methods due to existing structures. In recent years, pile-pressing methods have been developed to allow the installation of combi-walls with low levels of noise and no vibration. These techniques can be slower and, therefore, more expensive than traditional methods but have extended considerably the range of locations where combi steel walls can be utilised.

85.6.3 Materials

Most materials used will conform to relevant standards such as BS EN10248 (British Standards Institution, 1996a) for hot-rolled sheet piling. However, it is not unusual for primary elements, e.g. tubular steel piles, to be sourced on the open market and may be second-hand. If this is the case then it is essential that all materials used are certified and if necessary tested, to ensure that they comply with the design and project requirements.

85.6.4 Tolerances

Due to the greater retained heights leading the design towards a combi-wall solution, the weight and physical size of the wall elements will be much larger than for sheet pile walls. In order to ensure that the various elements are installed accurately it is generally found that the position and verticality will be controlled through the use of gates. These consist of a framework of temporary steelwork set at two levels, which ensures that the wall elements are installed accurately (**Figure 85.23**).

85.6.5 Interfaces, joints and connections

In order to join the various elements of a combi steel wall together, it is common that before delivery to site standard clutch profiles are welded to the primary elements. This allows them to be clutched directly to the steel sheet piles, which will act as the infill panels. See **Figure 85.20** for typical details of corner

and junction profiles, as detailed in the *Piling Handbook* (ArcelorMittal, 2005).

85.7 Soldier pile walls (king post or Berlin walling)

The soldier pile, usually a steel or concrete member, is driven or set in a pre-excavated hole at about 2–3 m centres.

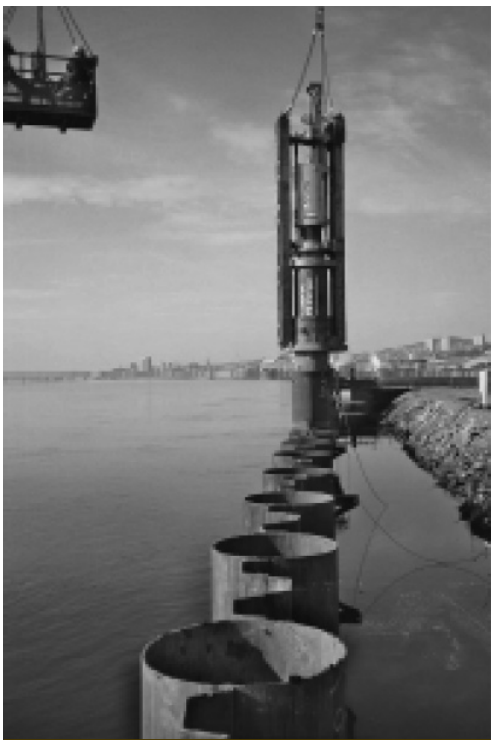


Figure 85.22 Driving of combi-wall elements using a hydraulic impact hammer

Excavation then proceeds at around 1 m 'steps' and horizontal sheetings or 'laggings' (timber, steel or concrete) are placed to restrain the ground between the soldier pile or king post. The system is illustrated in **Figure 85.24**.

The lagging encourages horizontal arching and the king posts carry the major thrust of the ground. By incorporating walings and tie backs, significant depths of excavation can be achieved. In very competent soil (stiff clays and soft rocks), lagging may not be necessary and simple local safety measures such as mesh pinning or guniting may be sufficient. The works are not considered permanent although the soldier piles may be incorporated into the final structure.

King post walls are most suited to competent ground, typically dense granular material (with lowered groundwater) and stiff to very stiff clays. Conventional soldier piles are not used in soft clays and loose granular material, which may run and contribute to loss of ground. Such works are comparatively cheap to construct in suitable ground. Of all the wall types, they are the most associated with 'failure' (usually excess deformation and local movement), probably caused by attempting to use them in the wrong conditions.

King post walls are not often employed in the UK (because of ground conditions) but are much employed in the USA, Asia and in central Europe. Local practices and useful semi-empirical processes and rules have evolved to suit local ground. Examples of such are:

- the German wedging practice, which encourages arching in poorer soils (**Figure 85.25**);
- the 'louvre' lagging gaps to permit back-packing of the ground in timber and concrete lagging practice (**Figure 85.26**);
- the empirical evolution of timber capacity to loading – illustrated for competent and difficult soils (**Table 85.2**).

The range of king post walling is typically up to three or four basements although in good conditions this is only constrained

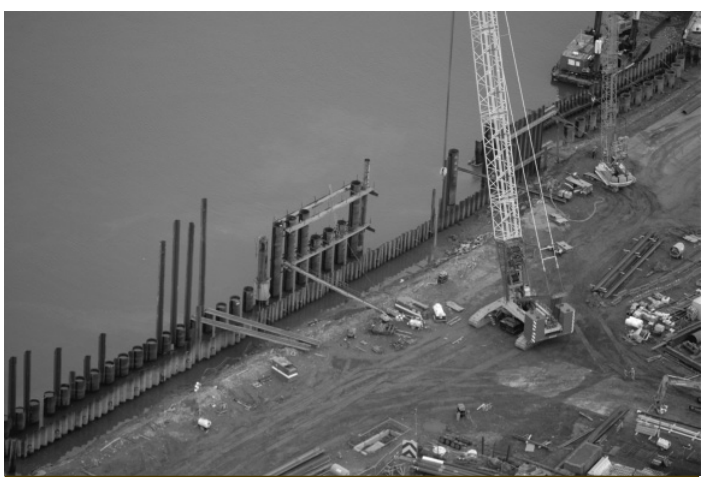


Figure 85.23 Typical combi-wall arrangement showing primary tubular piles and infill sheet pile elements



Figure 85.24 Typical soldier pile wall showing use of timber laggings

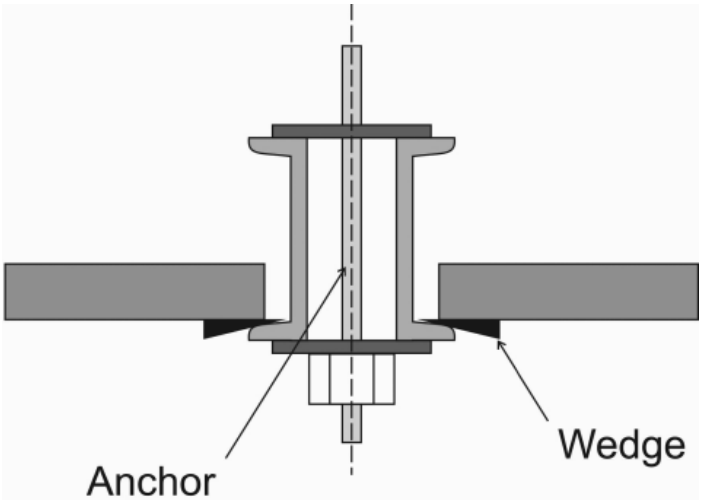


Figure 85.25 Plan of anchored retained king post with lagging wedges

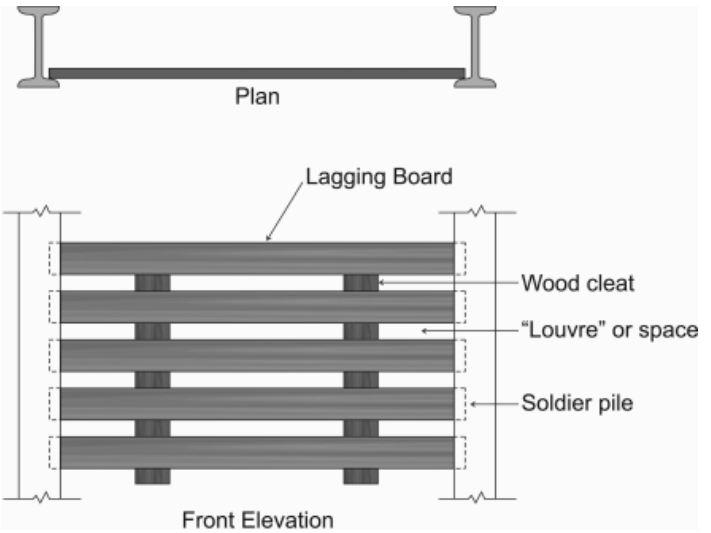


Figure 85.26 Typical simple king post wall with wood lagging

Soil description	Unified classification	Depth (m)	Recommended thicknesses of lagging (rough cut) for clear spans of: 1.5 m	1.8 m	2.1 m	2.5 m	2.75 m	3 m
Competent Soils								
Silts or fine sand and silt above water table	ML, SM-ML		(mm)					
Sands and gravels (medium dense to dense)	GW, GP, GM, GC, SW, SP, SM	0–7.5	50	75	75	75	100	100
Clays (stiff to very stiff); non-fissured	CL, CH	7.5–18.5	75	75	75	100	100	125
Clays, medium consistency and	CL, CH							
Difficult Soils								
Sands and silty sands (loose)	SW, SP, SM							
Clayey sands (medium dense to dense) below water table	SC	0–7.5	75	75	75	100	100	125
Clays, heavily overconsolidated fissured	CL, CH	7.5–18.5	75	75	100	100	125	125
Cohesionless silt or fine sand and silt below water table	ML, SM-ML							
Potentially Dangerous Soils*								
Soft clays	CL, CH	0–4.5	75	75	100	125	-	-
		4.5–7.5	75	100	125	150	-	-
Slightly plastic silts below water table	ML	7.5–18.5	100	125	150	-	-	-
Clayey sands (loose) below water table	SC							

Note:
*For 'potentially dangerous soils', the use of lagging is questionable.

Table 85.2 Empirical lagging sizing

by the depth to which the structural king post itself can be sensibly placed or driven and the propping arrangements. Shallow walls may be able to cope with larger lagging spans. The arching effect means that a fairly modest lagging provision can be used at depth, provided the horizontal spans are not excessive, i.e. about 2 m.

Plant used for the king post or soldier pile is dependent upon the construction system but is similar to the driven or bored

pile techniques described above. Where bored piles are used, then below the dredge larger diameter piles can be employed to increase bending capacity with an appropriate steel or concrete soldier pile plunged (or placed) into the bored pile.

Soldier pile walls have only the king post as their embedded component. The majority of their exposed area is dependent on the excavation, lagging and packing process and this needs much care, discipline and good working practice and interface consideration in construction. Suitable local drainage arrangements and alert observational techniques are required for the successful completion of the system. Particular attention has to be paid if the works are adjacent to sensitive structures, since local 'blowouts', squeezing or runs can be catastrophic; this process is more prone to systemic failure than other walling types.

Despite having to cope with many significant interfaces, in the correct ground conditions and with careful processes, soldier pile walls (and their derivatives) can give very effective and economic solutions to earth-retaining problems.

85.8 Other wall types

Although categorised as 'other' wall types, the remaining types of embedded retaining wall are in essence variations or hybrids of those mentioned previously. They are generally used in niche markets in areas where local ground conditions are appropriately favourable.

Four such 'other walls' are identified – walls employing:

- concrete sheeting;
- soft walls with pre-cast walls or steel sheet insertions;
- unreinforced mix-in-place walls;
- firm walls with king posts.

a. Concrete sheeting is much less robust than its steel precursor but nevertheless can be competitive if used in deep soft soil deposits – such as the clays of Mexico City and deep soft marine alluvium. Other than specialist head protection, its process and provision follows very similar patterns to its steel counterpart. In the correct situations, it is a real and useful alternative and it can give, for example, more robust protection in the aerobic zone. It tends to be limited in depth because of joining and transportation issues for the pre-cast panels. It has been used to support dredge depths up to typically 5 m.

b. 'Soft' walls have steel sheeting or pre-cast concrete inserts. Normally when a trench with support fluid is constructed, the most economical solution is to move on to a full diaphragm wall. Occasionally (often because of space constraints), it is necessary to provide pre-cast solutions, which can take the form of concrete panels or sheet piles. This technique has been used successfully in Germany on sites where a deep cut-off is required.

Plant and construction techniques are identical to diaphragm techniques. The 'soft' mix, which surrounds the inserts, is often a cement bentonite, which can be removed from the front of the wall and which aids 'watertightness' at the joints.

c. Unreinforced mix-in-place walls are employed where the ground is suitably granular such that a reasonably consistent and homogeneous mixing of cement can guarantee a sensibly 'strong' compressive strength material. This can be employed, for example, in shafts. The walls will be relatively thick since they must act in compression, and relatively shallow (<10 m). The required plant is highly specialised for an effective mix-in-place (**Figure 85.27**). The technique is more often encountered in Asia and increasingly in the USA.

d. Reinforced mix-in-place or 'firm' walls, where the reinforcement is essentially a series of king posts, have grown in popularity in areas with suitable (again mainly granular) ground. Steel and concrete pre-cast sections carry the majority of the structural loading with the unreinforced mix-in-place material containing the ground between the soldiers. King posts are set at a spacing sufficiently close to encourage arching or to cope with the applied loading (**Figure 85.28**).

Unlike soldier pile walls, such walls are fully embedded, do restrain local ground movement and groundwater movement at the excavation stage. The containing mix-in-place material requires considerable attention to its process to ensure that the king posts can be accurately plunged, yet the mix will 'firm



Figure 85.27 Plant for mix-in-place walls

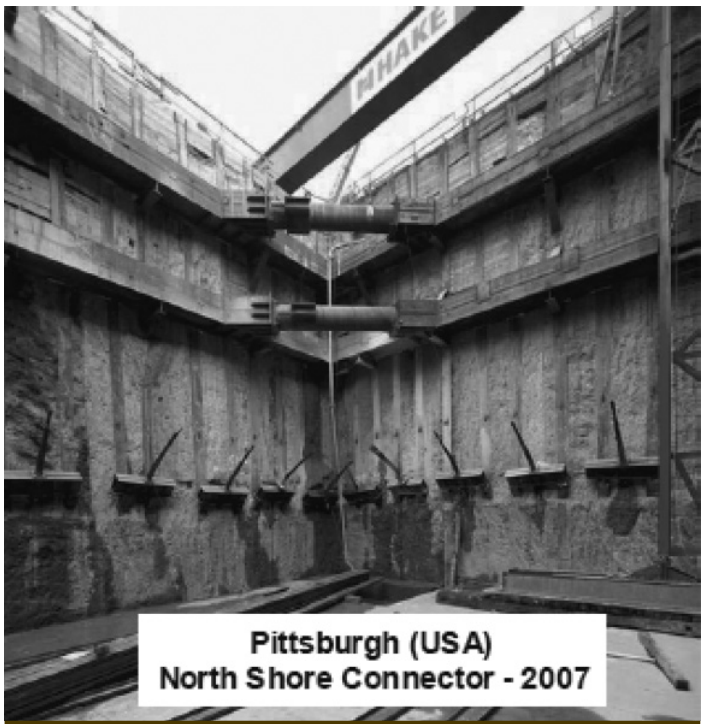


Figure 85.28 Reinforced mix-in-place or 'firm' walls

up' (in time) to sufficient strength to cope with local anomalies and flexure. Such walls are not encountered in the UK but are increasingly popular in Asia and the USA where ground conditions are suitable.

Plant for mix-in-place applications is continually developing and can vary from drumhead rollers to multi-armed auger systems.

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It is recommended this chapter is read in conjunction with

- Chapter 100 *Observational method*
- Section 6 *Design of retaining structures*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

Chapter 86

Soil reinforcement construction

Chris Jenner Tensar International Ltd, Blackburn, UK

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The process of construction of soil-reinforced structures is made up of many different operations. These include the assembly of all relevant information, planning, materials testing for validation purposes, delivery, acceptance and storage of materials, the construction itself, and maintenance requirements. The contractual links between the various parties to the project are also a very important consideration, as the construction process itself is often carried out by a separate contractor appointed just for that particular aspect of the works. The two main components of soil-reinforced structures are soil and reinforcement – hence the soil itself has a very important part to play in the success of the final result. The reinforcement materials should have had their assessment completed before the installation process commenced, whilst the combination of the elements can only be achieved in situ. The importance of placement and compaction of the soil materials in order to achieve the strength properties used in the design, and hence a strong, stable reinforced soil block, must be appreciated by the construction team.

86.1 Introduction

Soil-reinforced structures are composite structures which combine the properties of soil (earth) and soil-reinforcement materials. The applications can be divided into two main areas:

(i) *Vertical and near-vertical walls, and steep slopes* Walls and steep slopes are essentially soil-retaining structures. The composite material of soil and reinforcement is designed to have the mass and geometry necessary to prevent the retained unreinforced soil from failing. Vertical and near-vertical walls are generally faced with some form of hard facing material, whilst steep slopes are normally vegetated to provide a green and environmentally pleasing face. The function of the facing is essentially for erosion protection and aesthetics. In terms of the design, the contribution of the facing is relatively small, but the consequences of the facing not providing the appropriate protection could be quite severe. The importance of the soil element of the composite material must be appreciated and is dependent on the quality of the construction process.

(ii) *Embankment foundations* Embankment foundations applications are generally used when the existing soils beneath a proposed embankment, which is stable within itself, are not capable of supporting the new construction. Embankment foundation applications can range from relatively light reinforcement materials providing a working platform through to very robust, piled embankment solutions with very high-strength geotextile reinforcement sheet materials. There are two British Standard publications that are relevant to the construction phase of a project, BS EN 14475:2006 (BSI, 2006) and BS 8006–1:2010 (BSI, 2010).

The construction process can be divided into three distinct phases: pre-construction, construction and post-construction, which are discussed below.

86.2 Pre-construction

The gathering of information before construction commences is a very important part of the process and can be the key to avoiding problems later on. This is particularly important if the contractor who is actually carrying out the construction is outside the original contract team. Information regarding existing site conditions, services, and any restrictions that may apply to the work, should be supplied to that contractor at a very early stage. A comprehensive list of particular information is given in BSI (2006) but there are some areas that warrant additional emphasis.

86.2.1 Roles and responsibilities

The roles and responsibilities of all the parties involved in the reinforced soil construction should be defined very clearly at an early stage of the process. Good definition at this early stage can ensure that any problems that occur during the construction process are dealt with quickly and efficiently by the responsible party. Poor definition inevitably leads to confusion and a much slower response to a problem, which may have escalated in the interim. The split of the responsibilities between the engineer, the reinforced soil designer, the reinforcing material supplier, the main contractor and the reinforced soil sub-contractor could be quite complex, and the contractual links need to be well defined. In many cases one party will carry out a number of these tasks, but there could be situations where all the functions are separate.

86.2.1.1 Reporting channels

The arrangements for change management need to be understood by all parties to deal with any unforeseen circumstances in soil conditions, for example, and the inevitable alterations to geometry and detail that occur during the life of a project. The definition of the line of communication is vital to the smooth

running of the process for standard design-generated structures, and becomes even more important when the observational approach to design is adopted.

86.2.2 Site investigation

An adequate site investigation (see Section 4 *Site investigation*) should be carried out to determine the existing soil conditions and also the possibility of using existing soils in the reinforced soil structures. There may be situations where the reinforced soil contractor needs to design and install temporary works in order to carry out construction of the permanent works. In these cases, the contractor will require soil testing in order to be able to assess the suitability of fills and soil properties for the temporary works design (see Chapter 49 *Sampling and laboratory testing*).

If the reinforced soil contractor has some choice in the selection of the reinforcement material, then the chemical properties of the existing soils and the fill could be important in that selection process. Guidance on the suitable fill types and durability data for different materials is given in BSI (2006) and BSI (2010).

86.2.3 Planning

The reinforced soil element of a project may be very large – in some situations being the whole construction; in others it may be only a very small part of the complete project. It is very important, therefore, to plan for the reinforced soil construction and ensure that all the components are available in the right place at the right time.

86.2.3.1 Materials

The design and drawings for the reinforced soil element of the works will include material properties, and facing details if appropriate. There may be some element of selection that is available to the contractor in terms of the source of the fill material and the reinforcement materials. In some cases, the required facing appearance and the reinforcement type will be linked by a proprietary connection detail. Information on suitable fill types for particular applications is given in BSI (2006).

86.2.3.2 Material testing

The design will include the specified properties of the fill materials that have been used to derive the reinforcement layout. The actual fill to be used should be tested or verified in some other way to ensure that it satisfies the design value (see also Chapter 99 *Materials and material testing for foundations*).

The reinforcement materials are the subject of the Construction Products Directive (1988), and as such will have CE marking information to verify the properties – providing that the harmonised properties are relevant to the application for which the reinforcement is being used.

Once the fill material and the reinforcement have been selected, the interaction parameters between the two materials need to be determined to verify whether the design parameters are satisfied. More comprehensive details of the information that would be expected to be in a design output are given in BSI (2006).

86.3 Construction

86.3.1 Delivery and storage of materials

Identification of the different components involved in the construction, separation and isolation of apparently similar materials is very important in preventing mistakes. Facing components should be stored carefully as they are the face of the structure on completion. Some reinforcing materials may require specific storage conditions; details should be obtained from the material supplier.

86.3.2 Preparation of the foundation

The foundation level and extent will be indicated in the design and, as with any structure, the accuracy of the initial foundation layers is vital in the production of a serviceable end result. In situations where the foundation soils are soft, it is important to limit the disturbance that they have to endure during the construction process. The parameters used in the design (see Section 5 *Design of foundations*) will generally be those of the foundation soils in their pre-construction condition using the parameters from the ground investigation report (see Chapter 50 *Geotechnical reporting*) and hence any major disturbance of those soils will have a significant effect on the final result.

Notes on the installation of vertical drains and reinforcement over piled foundations are available in BSI (2006) and BSI (2010).

When hard facing materials are used in the construction of reinforced soil walls, there is generally a requirement for some form of levelling pad, concrete or sometimes gravel, on which to place the first face elements. This pad does not have a structural purpose but it does provide a firm surface from which the wall face rises. As with any engineering structure, errors in line or level at the foundation level will be very difficult, if not impossible, to rectify later.

Soft-faced walls and slopes, such as vegetated faces, do not generally require a levelling pad.

86.3.3 Installation of drainage

Drainage of structures is important from two points of view.

Firstly, there is a design aspect regarding the possible presence of water within or outside a structure. Water that is not allowed to drain away can have a significant effect, so if there is a possibility that the materials being used are not free-draining, then drainage layers should be included and linked into the drainage system on the site. Some particular fill materials such as pulverised fuel ash (PFA) require particular drainage arrangements to ensure isolation of the PFA block.

The second important reason for drainage provision is aesthetic. A number of facing block finishes have open joints and staining can occur if there is no drainage layer behind the block face to draw any water seepage down to the toe of the wall.

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86.3.4 Placement of facings

General information on the placement of facings is given in BSI (2006). It must be noted that whilst the facing does not have a true structural function, it does provide erosion protection and it is the visible result of the construction. The accuracy of placement of the facing and the care with which the fill is placed and compacted behind it define the quality of the final structure. A good understanding of these principles by the reinforced soil contractor's team will result in sound and serviceable structures.

The flexibility of modular block facings for reinforced soil walls enables complex geometric shapes to be constructed. Care should be taken to ensure that the heights of free-standing blocks without reinforcement are limited to those advised by the supplier or manufacturer. Exceeding the recommended heights can cause great difficulties in keeping the line of the wall within acceptable tolerances. Indicative tolerances are given in Annex C of BSI (2006); see **Figure 86.1**.

Lifting and placing of the larger panel facings is carried out using craneage, whereas the modular block facings can generally be manhandled. The limitations of block size and weight to comply with the Health and Safety requirements for manual lifting must always be observed.

86.3.5 Placement of fill materials

86.3.5.1 Walls and slopes

Whilst all the tasks that are involved in the construction of a reinforced soil structure are important to produce a satisfactory and pleasing result, the placement and compaction of the fill is probably the most important operation of the whole process. The development of a true composite mass of soil and reinforcement depends on the two materials being intimately related to each other by the appropriate mechanism. Strip-type reinforcement materials generally rely on a frictional interaction on the surface of the reinforcement, whilst grid-type materials interact through an interlocking of the soil particles into the geogrid apertures. Whichever mechanism is appropriate, it will only be mobilised effectively if the soil is placed and compacted to standard specification levels. The design of structures, vertical walls and steep slopes to BSI (2010) uses the peak friction angle for the reinforced fill and for the unreinforced backfill. The peak friction angle will only be achieved if the compaction of the soil is carried out to specification. The case study in **Box 86.1** shows how a well-compacted, good-quality fill material can produce a completely stable and durable soil/reinforcement composite mass even under severe circumstances. BSI (2006) gives a large amount of guidance on this aspect of the construction, recognising its importance. Particular care should be taken to follow the guidance regarding compaction close to the face; see **Figure 86.2**.

Incorrect operations in this area can cause face deformations which may be irretrievable and result in an unsatisfactory appearance.

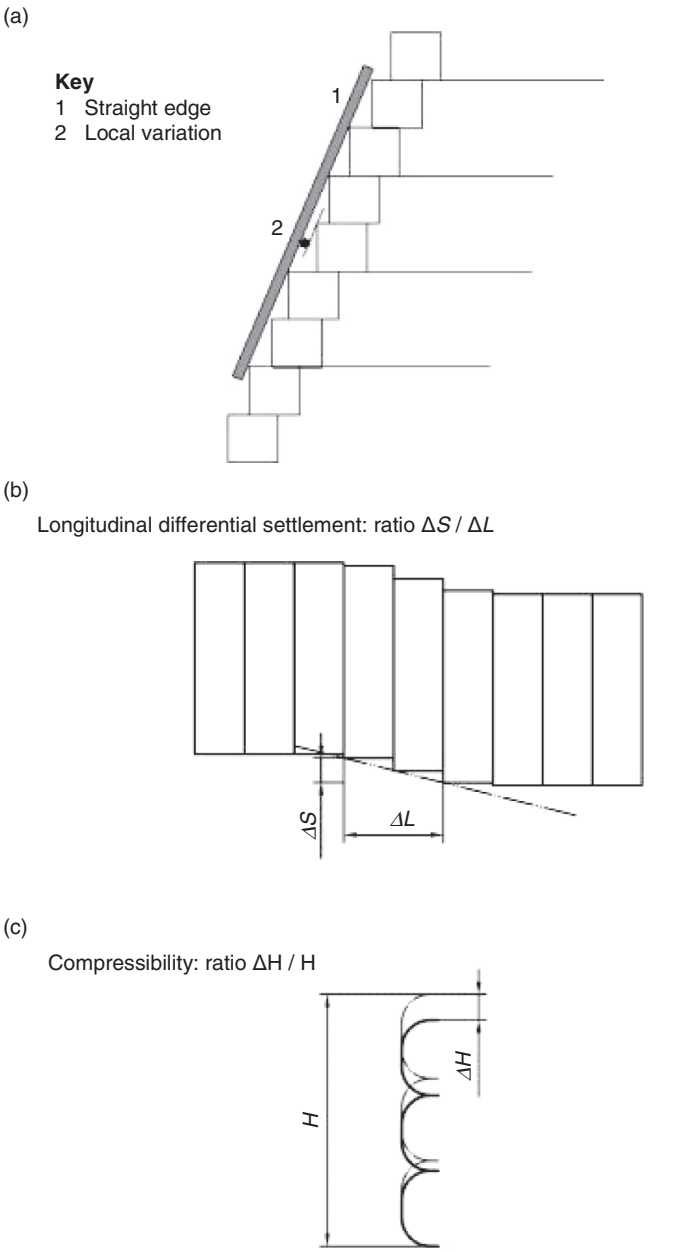


Figure 86.1 Indicative construction tolerances from BS EN 14475. (a) Cross-section through wall; (b) front elevation of wall; (c) cross-section through wall

Reproduced with permission from BS EN 14475, Annex C (2006) © British Standards Institution

86.3.5.2 Embankment foundations

BSI (2006) and BSI (2010) give good guidance on this quite specific application area. The reinforcement of embankment foundations generally involves the requirement to place fill over soft soils or piles which are hard points within a generally soft soil. Whilst the construction of walls and slopes can generally follow reasonably standard procedures, the construction sequence techniques have to be more site-specific. Placement of



Figure 86.2 Limitations of compaction plant close to structure face
Courtesy of Tensar International Ltd

relatively thin soil layers over a wide area and a gradual increase of load onto the foundation are general principles, but there will be many variations and these should be considered with care.

86.3.6 Placement of reinforcement

86.3.6.1 Walls and slopes

Guidance on reinforcement placement is generally available from the manufacturer or supplier, and BSI (2006) and BSI (2010) also give helpful information. The main points to bear in mind are that the reinforcement should not be damaged in the construction process and that any flexible reinforcements should be pulled tight, not tensioned, to ensure that any slack is removed from the system. Construction equipment should not run on the reinforcement, regardless of type – there should always be a layer of soil above the reinforcement before any trafficking is allowed whether by the compaction plant or other site equipment.

Connection of the reinforcement to the face unit is a critical aspect of reinforced soil structures and can be achieved in many ways. Steel strip reinforcement may be bolted to anchor plates embedded within the facing panel, while there are many proprietary fixing devices for the connection of geosynthetic reinforcement to modular block facing systems. Reference should be made to manufacturers' literature for full details.

Pulling the reinforcement tight minimises the possibility of post-construction face movement. This may happen if the face connection between the reinforcement and a facing block, or a joint between two pieces of reinforcement is left with slack. A typical example of removing the slack is shown in **Figure 86.5**.

Protection of the connections during the construction process, particularly the placing and compacting of the fill materials, is very important. The restrictions on compaction equipment weight and thickness of fill over the reinforcement layers must be followed. Long-term durability requirement of the connection should also be carefully implemented in accordance with the manufacturer's specification to avoid possible post-construction problems.

Box 86.1 Case study

A 19-m-high reinforced soil wall constructed in a very dry climate was subjected to a flash flood before the contractor had placed sufficient scour protection at the toe of the structure. Consequently, over a short length, the levelling pad was undermined and the facing blocks fell away from the reinforced soil block. **Figure 86.3** illustrates a flash flood that occurred later at the site.



Figure 86.3 Example of severe flood water flow
Courtesy of Tensar International Ltd

However, the fill material was of very good quality and the contractor was careful to ensure that the compaction matched the specification required, with the result that the main structural element, the reinforced soil composite block, remained completely serviceable throughout this very severe encounter. During the face reconstruction, the reinforced soil block did not require any treatment and the road remained open and serviceable with no signs of cracking or damage. **Figure 86.4** illustrates the stable reinforced soil block.

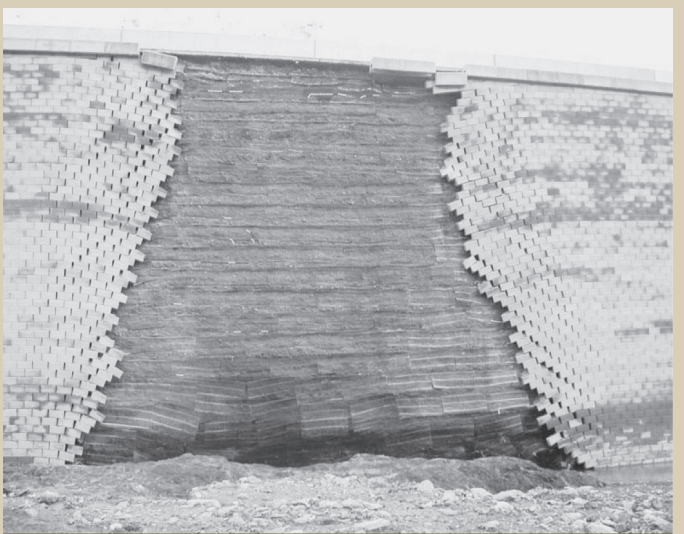


Figure 86.4 Result of lack of scour protection, stable reinforced soil block after critical event
Courtesy of Tensar International Ltd

The wall face was reconstructed with the correct scour protection in place to prevent a recurrence of the problem.

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(a)



(b)



(c)

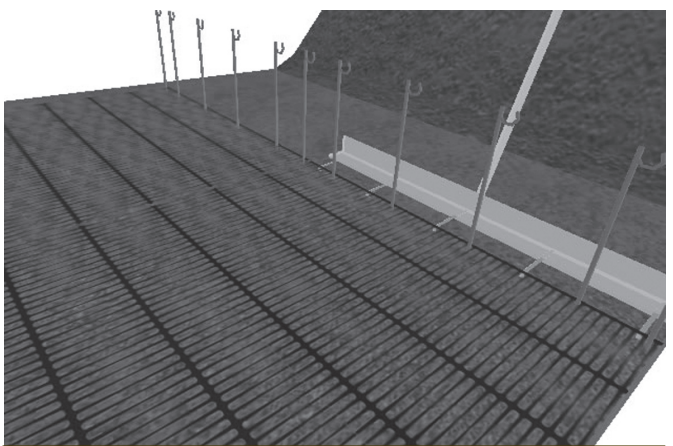


Figure 86.5 Typical procedure for the removal of any slack in flexible geosynthetic reinforcement (a) A length of geogrid reinforcement having a nominal manual tension applied through a hooked beam to remove any slack in the system. (b) The nominal manual tension being applied to the connection between the geosynthetic and the facing blocks. (c) Temporary fixing of the nominal tension with anchor pins whilst the fill is placed on to the reinforcement

Courtesy of Tensar International Ltd

86.3.6.2 Embankment foundations

The principles of reinforcement placement in embankment foundations are essentially the same as for walls and slopes, except there will not be a face against which to take any slack out of the system. Joints or overlaps will probably be required and these are the areas that require the most care. Working below water brings its own special requirements when using materials which actually float; weighting down may be required.

86.3.7 Vegetation

A very common facing for steep slopes is vegetation, where the nutrient soil for the vegetation is placed behind some sort of open facing and the vegetation grows through to provide a soft face to the structure. There is a very comprehensive section on vegetation in the revised BSI (2010) but the main requirements for survival of the face are water and nutrients in the face soil. Selection of the type of vegetation is also quite critical in ensuring survival – naturally growing local vegetation probably has the best chance of successful development. Special drought-resistant plant species have also been developed for particularly difficult situations.

Timing of the introduction of vegetation is a planning issue and careful thought can improve its establishment.

86.3.8 Monitoring and supervision

86.3.8.1 Walls and slopes

The construction of reinforced soil walls and slopes is a relatively straightforward operation using well-accepted engineering earthworks principles. The main areas of interest – the supervising and monitoring of the construction – can be summarised as:

(i) Compaction of the fill. Ensuring and verifying that the compaction meets the specification is of major importance with regard to the development of the design soil properties and the true composite block performance.

(ii) Diligent observation of the line of any facing during construction will ensure an early warning of any tendency to veer out of tolerance.

(iii) Monitoring any reinforcement connection details, reinforcement to face or reinforcement to reinforcement, and ensuring that any slack is taken out, particularly of flexible reinforcement materials, will benefit the final appearance.

86.3.8.2 Embankment foundations

The construction of reinforced embankment foundation projects is not as formalised as for walls and slopes, and each project will have its own specific characteristics. A construction sequence, specific to the project, should be part of the design and any monitoring and supervision requirements will be specified in that document. General earthworks principles will, of course, still apply.

86.4 Post-construction

86.4.1 Records

Records of weather conditions, etc., during reinforced soil construction should be the same as those conventionally kept on construction sites. A particularly important area of record keeping is the location of the reinforced soil structure itself. The fact that a structure is constructed of reinforced soil may not be obvious; a concrete-faced wall may look the same as a reinforced concrete wall, and hence an indication that soil reinforcement is present is important in the event of further construction in the future.

86.4.2 Maintenance

In general, reinforced soil structures are low-maintenance. The usual requirements of ensuring that drainage is maintained, etc., still apply and in extreme conditions can be very important. Maintenance of vegetated faces is an area that may need some consideration in terms of irrigation and plant species selection.

86.5 References

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British Standards Institution (2010). *Code of Practice for Strengthened/Reinforced Soils and Other Fills*. London: BSI, BS 8006–1:2010.

86.5.1 Further reading

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Koerner, R. M. (2005). *Designing With Geosynthetics* (5th edition). New Jersey, USA: Prentice Hall.

McAleenan, C. and Oloke, D. (2010). *ICE Manual of Health and Safety in Construction*. London: Thomas Telford.

Raymond, G. P. and Giroud, J. P. (eds) (1993). *Geosynthetics Case Histories*. International Society for Soil Mechanics and Foundation Engineering. Canada: BiTech Publishers.

86.5.2 Useful websites

International Geosynthetics Society (IGS); www.geosyntheticssociety.org

UK chapter of the IGS; www.igs-uk.org

It is recommended this chapter is read in conjunction with

- Chapter 72 *Slope stabilisation methods*
- Chapter 73 *Design of soil reinforced slopes and structures*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 87

Rock stabilisation

Richard Nicholson CAN Geotechnical Ltd, Chesterfield, UK

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Rock faces can be natural or man-made features of the landscape that can arouse significant local or national interest. A designer needs to be sensitive to the position and location of a rock face and give careful consideration to the likelihood and consequences of a failure. This is likely to determine the conceptual approach to any stabilisation works. Risk mitigation measures can comprise a management approach where failure of the rock face is permitted, but the consequences of a rockfall are managed. This can be achieved through the installation of warning systems, rockfall netting or catch fences. Alternatively, an engineered solution can be sought, typically comprising the removal of the hazard or a reduction in the likelihood of a rockfall. Examples of remedial works adopting this approach would include scaling, installation of ground anchors or the application of sprayed concrete.

Installation and subsequent maintenance of any remedial works should be undertaken by a specialist with experience of working in potentially exposed locations on an unstable rock face. Careful consideration needs to be given to access requirements and the safety of the workforce and third parties. It is, therefore, essential that a full risk assessment is undertaken at the design stage and reviewed throughout construction.

87.1 Introduction

87.1.1 Principles

Rock faces can form naturally or as a result of construction activities. The stabilisation of rock faces can, therefore, be undertaken on existing rock slopes or newly formed faces, either during or after their excavation. Both landowners and employers owe a duty of care to ensure that the safety of the site user is not compromised.

In situ rock is unlike the majority of materials encountered by engineers. Rock can be highly variable within a short distance and usually only the front face is visible. When working with *in situ* rock it is critical to have a clear understanding of the geological context and the engineering characteristics of the rock mass, as well as the forces that are acting upon it. The design process is likely to require an inspection of the exposed face and an intrusive ground investigation to obtain an understanding of not only the engineering properties of the rock but also the quality of the rock mass as a whole. The design of stabilisation measures may, therefore, need to be undertaken by a specialist.

When selecting the design approach to be adopted, the designer will need to consider both the likelihood and consequences of a failure of the rock face. The land use both at the toe and crest of the face is likely to determine whether risk mitigation measures comprise a management approach, where failure of the rock face is permitted, or whether a fully engineered solution is required.

87.1.2 Access considerations

The design of any remedial measures will need to give careful consideration to access requirements, both for construction and subsequent maintenance of the works (see **Figure 87.1**). Newly cut faces can be stabilised during excavation in a top-down manner as long as there is a clear understanding of the scope of the whole works.

Existing rock faces in remote locations may be difficult to access from the nearest highway. Alternatively some rock faces will preclude working from the toe or crest (e.g. due to the presence of infrastructure or water). The inherent nature of existing steep and irregular rock faces may also preclude the use of traditional plant and scaffolding. Consideration may need to be given to employing a contractor specialising in difficult access techniques. This will typically involve specialist plant reaching up from the toe, or reaching down from the crest of the rock face. Alternatively, equipment may be suspended by steel cables attached to a temporary access system located behind the crest and winched into position on the face.

Personnel undertaking the works must, therefore, not only be competent in undertaking the specified work tasks but also



Figure 87.1 Different access solutions
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be competent in accessing the work site. Rope access methods in accordance with BS 7985 (British Standards Institution, 2009) have become increasingly accepted as offering a safe, cost-effective access solution.

87.1.3 Safety considerations

Given the inherent nature of rock stabilisation works they are often undertaken on faces that pose a risk of injury due to rockfall. In assessing the stability of the face careful consideration must be given to the safety of the workforce, other contractors and members of the public. The works should be planned such that a safe working environment can be provided at all times. Consideration may therefore need to be given to the implementation of the following mitigation measures:

- temporary works including removal of loose rock or strapping of blocks;
- implementation of an inspection and monitoring regime during the works;
- a method of working such that construction is undertaken in a controlled, systematic manner (e.g. top-down);
- installation of quarantine areas at the toe or crest to protect the workforce and third parties.

It is, therefore, critical that a full risk assessment is undertaken at the design stage and reviewed throughout the construction stage. It is also important that only competent contractors are employed, with a proven track record for the safe delivery of rock stabilisation works.

87.1.4 Environmental considerations

Existing rock faces are natural features of the landscape that often arouse significant local or national interest from various organisations. When designing or undertaking any stabilisation works, due regard for the conservation, recreation and access requirements for the rock face and the surrounding area should be given. Environmental considerations may include:

- setting and location;
- fauna and flora;
- geological interest;
- historical interest;
- recreational amenity (walking, climbing).

Each of the above may affect the design approach and the scope of works or the time of year in which the works are undertaken. In addition, the rock face or the surrounding area may have statutory protection (e.g. RIGS, SSSI) and consultation with regulatory bodies will be required.

87.2 Management solutions

Risk mitigation measures can comprise a management approach where failure of the rock face is permitted, but the consequences of rockfall are managed to minimise the effect of a failure.

87.2.1 Warning systems

Warning systems can be quick and easy to install and provide a reactive solution whilst permanent works are designed, or alternatively can be installed as part of the ongoing management regime for the site.

Warning systems might be simple signage to warn site users in areas of known rockfall (e.g. as typically seen on highways). More sophisticated warning systems include the regular inspection or monitoring of rock faces where the consequences of a failure may be considered more significant (e.g. as used on railway cuttings). This may include the installation of simple trip wires or instrumentation such as load sensors that can be monitored remotely by data logger and modem. If agreed trigger values are reached then alarms can be activated and appropriate action taken.

87.2.2 Rockfall netting

Rockfall netting is a cost-effective method of controlling the fall of rock debris and discrete blocks (see **Figure 87.2**). There are several products readily available including polymer geogrids and PVC-coated double-twist steel mesh. The netting is typically hung as a drapery with rock allowed to fall to the toe of the face.

Consideration must be given to the typical block size, mass and trajectory to ensure that rockfall will be contained by the netting. It is important to realise that the netting will only be as effective as the weakest component within the system. The netting must, therefore, be secured to a robust anchor system installed behind the crest. This typically comprises a crest cable fixed to rock bolts with a bond length within competent rock. Should soil materials be present at the crest then driven earth anchors are often used to provide the necessary restraint.

Adjacent rolls of netting must be securely connected by tie wire or steel hog rings fixed along the seam, or selvedge wire, at regular centres. This join is often the weakest component of



Figure 87.2 Typical rockfall netting installation
© CAN Geotechnical Ltd

the system. Steel cables can be woven or fixed to the selvedge to act as reinforcing elements.

Consideration needs to be given to the proximity of infrastructure to the rock face, as accumulations of trapped debris may extend well beyond the toe of the face. Careful detailing of the toe fixings is also important to ensure that rock debris can be easily and safely released for periodic uplift and clearance.

87.2.3 Catch fences and ditches

Where there is room at the toe of a rock face, consideration can be given to the construction of catch ditches or bunds. Simple earthwork solutions can be cost effective, particularly where the height or condition of a face would require extensive remedial measures to be installed. Bunds are typically used in working quarries where the location of rock faces can be temporary and earth-moving plant is readily available.

Ditches and bunds can be constructed in combination where there are soil materials at the toe of the face (e.g. a scree slope). Ditches can be lined with uniformly graded aggregate to dissipate the impact energy from falling rocks, to reduce the bounce height and to further mitigate the risks associated with rockfall. Bunds can also take the form of walls (typically gabion structures) as often seen in alpine areas to contain or divert debris flows away from infrastructure.

Catch fences may be often installed where trajectory analysis suggests that bounce heights would require substantial bunds or ditches (see **Figure 87.3**). Catch fences can be sacrificial, designed to slow the speed and reduce the kinetic energy of a falling rock, or permanent being designed to withstand repeated impacts. The location, height and capacity of a fence are critical to contain rockfall. Rigorous trajectory analysis should be undertaken, utilising one of the software products that are widely available. Catch fences can be cantilevered from the face (e.g. above a tunnel portal) or free-standing structures located downslope from the toe of the rock face, often on scree slopes or areas of unconsolidated ground.

Various proprietary products are available, typically tested and certified from 100 to 5000 kJ. Whilst the fence and individual components are likely to have been subjected to rigorous testing within a controlled environment, considerable uncertainty may exist in relation to the foundation detail for the fence posts. The foundations may comprise concrete pads with compression or tension piles securing the fence to the underlying competent strata. Additional raking and lateral anchors may also be required to secure stay and end fence cables, depending on whether the fence is a dynamic or static system.

Figure 87.4 summarises the typical management solutions.

87.2.4 Case study – Merstham cutting

Failure of a steep-sided chalk cutting near Merstham, south London, resulted in the reactive installation of remedial works. The cutting is located on the northern approach to the South Downs tunnel on the London to Brighton railway. The first phase of works comprised the installation of rockfall netting



Figure 87.3 Construction of a 3000 kJ catch fence in Gibraltar
© CAN Geotechnical Ltd

over approximately 40 000 m^2 to provide an effective containment system (see **Figure 87.5**). This netting was installed, predominantly with the line open to trains, within a six-week period. A second phase comprised the construction of an additional king post wall at the toe of the cutting towards the tunnel portal utilising temporary possessions of the railway.

87.3 Engineered solutions

Engineered solutions typically comprise the removal of the hazard or reducing the likelihood of rockfall. Stabilisation of the rock face is often achieved by reducing pore water pressures or changing the force regime acting on the face (e.g. by the provision of structural support or re-profiling the face).

87.3.1 Scaling, block removal and re-profiling

Scaling comprises the removal of rock debris and loose blocks from the face utilising light hand tools (see **Figure 87.6**). Care should be taken not to dislodge material that could result in the stability of the face being compromised. It should also be noted that scaling is a palliative treatment, often being undertaken at the commencement of works to mitigate against the short-term risk of a rockfall.

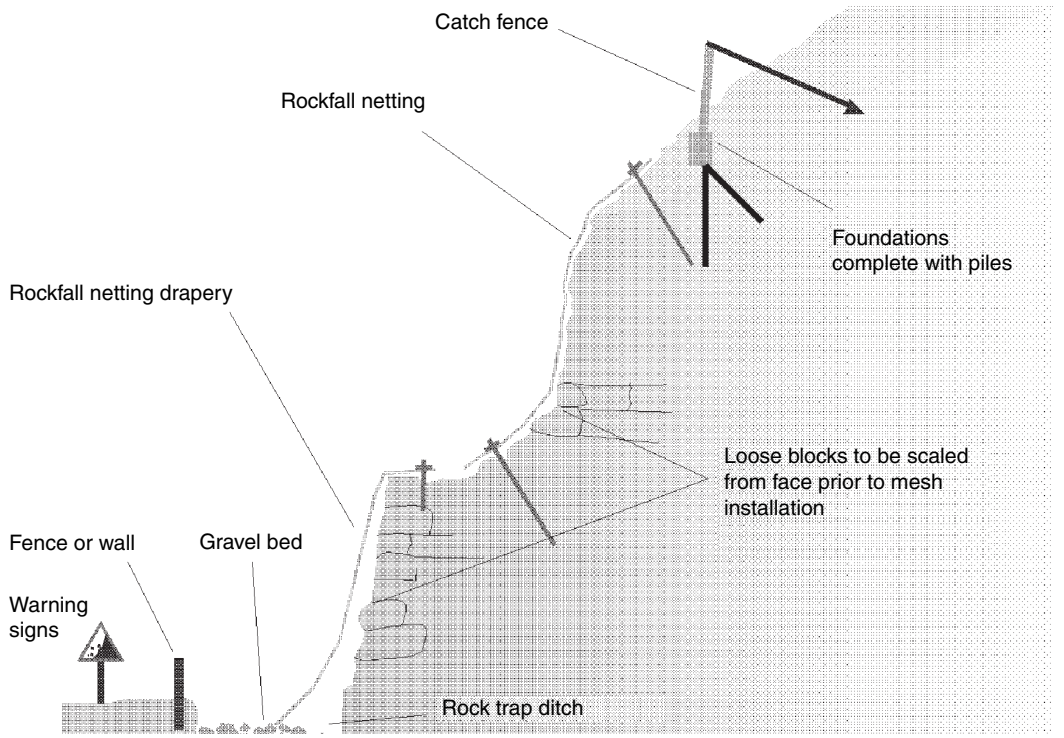


Figure 87.4 Summary of typical management solutions
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(a)



(b)



Figure 87.5 Installation of rockfall netting
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Block removal is the controlled dislodgement of discrete blocks, often by the application of pneumatic or hydraulic breakers or jacks. Expansive grouts and gas systems can also be used to break rocks.

Where there is available land at the crest, wholesale re-profiling of the rock face to a stable angle will provide a long-term, low-maintenance solution. However, consideration needs to be given to the method of excavation as blasting can dilate joints resulting in further instability. Alternatively benches can be formed to reduce the face height, contain rockfalls or provide access to undertake further remedial work.

87.3.2 Ground anchors

Further details of ground anchors are provided within Chapter 89 *Ground anchors construction*; however, they are also included within this section for completeness (see **Figure 87.7**).

Significant rock features can be restrained by the introduction of reinforcing elements. Monobar or multi-strand ground



Figure 87.6 Controlled lowering of a block
© CAN Geotechnical Ltd

anchors are installed into pre-drilled large diameter holes. Substantial compressive loads can then be applied to ensure the stability of the rock mass.

A ground anchor can, therefore, be considered as an installation capable of transmitting an applied tensile load to a load-bearing stratum and comprises an anchor head, free length and bond length.

Given the high-capacity, safety-critical nature of ground anchors they are manufactured to a specific requirement for corrosion protection, in accordance with BS EN1537 (British Standards Institution, 2000). Care must be taken to ensure that the integrity of this corrosion protection is not compromised during anchor installation. Rigorous testing and commissioning of the anchor must also be undertaken to demonstrate the efficacy of the installation.

Low-capacity ground anchors that are not safety critical are often referred to as rock bolts. They are available in a range of materials (stainless steel, glass-reinforced plastic, epoxy-coated or galvanised high-yield steel) and can be grout or resin bonded. In addition, dowels are passive installations relying primarily on the shear-strength characteristics of the bar being used. They are often installed into small diameter drill-holes and resin bonded.

Ground conditions will determine the method of drilling and the need for any additional ground improvement or grouting to ensure the integrity of the anchor bond.

87.3.3 Surface protection

Often the stability of a rock face can be compromised by the effects of ongoing weathering, particularly where the rock mass is weak or fissile. Protection of the face can be afforded by the provision of 'soft' or 'hard' facing systems.

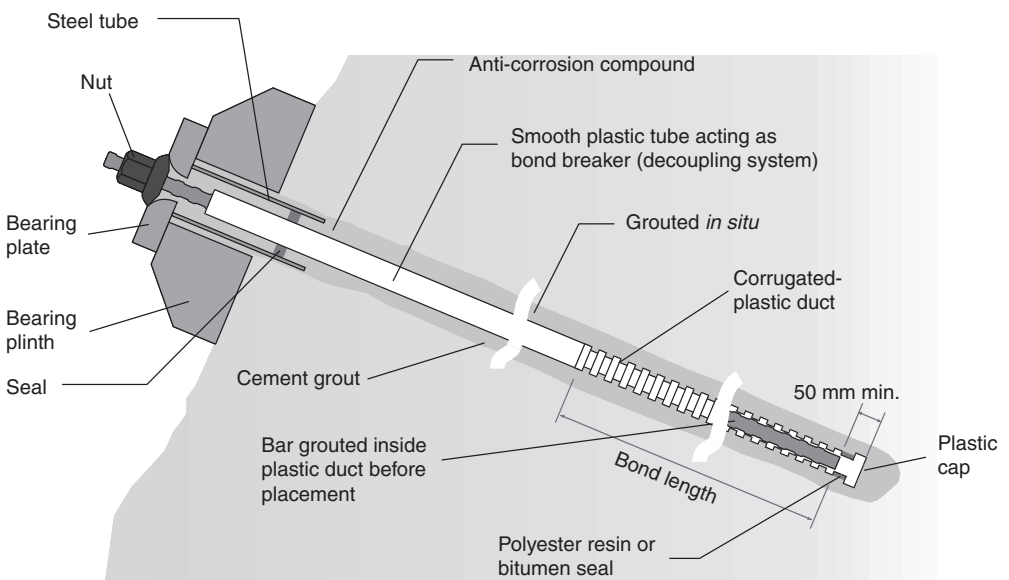


Figure 87.7 Ground anchor detail
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Soft facing systems typically comprise rockfall netting, which can be tensioned using an array of pre-installed rock bolts. The rock bolts are installed within surface hollows so that the netting will be tight against the rock face. This method of installation may, therefore, be appropriate for the stabilisation of the near surface materials, and can in turn encourage indigenous vegetation to become established. This can provide an aesthetically pleasing finish; however it should also be noted that vegetation may become a maintenance liability.

Hard facing systems typically comprise concrete pneumatically sprayed to suit the profile of the rock face. The concrete can comprise 'wet' or 'dry' mix and may be reinforced with glass fibres or steel staples. The mix can also include pigment in an attempt to complement the rock face. Weepholes or drainage membranes should be provided to alleviate any build-up of groundwater. Careful detailing of the tie-in detail is also required to avoid delamination or spalling of the finished concrete. Sprayed concrete is particularly effective where preferential weathering of a weak bed within the face has resulted in the undercutting of more competent strata above.

87.3.4 Dentition and buttressing

Sprayed concrete can also be installed in conjunction with steel reinforcement (either fixed *in situ* or weldmesh) to provide structural support to the face (see **Figure 87.8**). Care must be taken to remove any loose material from the surface to be treated and depth gauge markers should be used to ensure that the required depth of concrete is applied to both the face and reinforcement.

For larger structures, cast *in situ* buttresses can be constructed. These will typically be tied back into the rock face by ground anchors with the head blocks either concealed within the fabric of the structure or exposed on the front face. Buttresses can be faced with locally won masonry to provide a sympathetic finish.

Crib walls, pier and beam structures and Reno mattresses can also be used to provide toe support or protection of the rock face.

87.3.5 Drainage

The provision of effective drainage is often overlooked for rock faces. Groundwater can be the controlling factor in rock stability, particularly where there are inter-bedded or steeply dipping strata, or where out-dipping clay seams are present. An understanding of the hydrogeological characteristics of the rock mass should, therefore, be obtained if the implementation of an effective drainage regime is being considered as a stabilisation solution.

Adits have long been used for the deep drainage of mines and rock faces. Face drainage such as the installation of inclined drains can also reduce groundwater levels, particularly within cuttings. Cut-off or interceptor drains can be installed behind the crest of the rock face to divert surface run-off away from the rock face. This will reduce potential infiltration of joints and the consequent build-up of pore water pressures behind the face.

(a)



(b)



Figure 87.8 Application of sprayed concrete and dentition works
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Figure 87.9 summarises the typical engineered solutions.

87.3.6 Case study – Camp Bay, Gibraltar

A 15 000 m^3 failure of the former sea cliff above the main beach area in Gibraltar resulted in the multi-phased stabilisation of a 100-m-high rock face. The works comprised the installation of over 250 rock anchors. Typically, these were 100 tonne, multi-strand rock anchors up to 38 m in length. The fault-brecciated, karstic limestone resulted in the need to pre-grout the drill holes and undertake rigorous testing of all installed anchors. Other

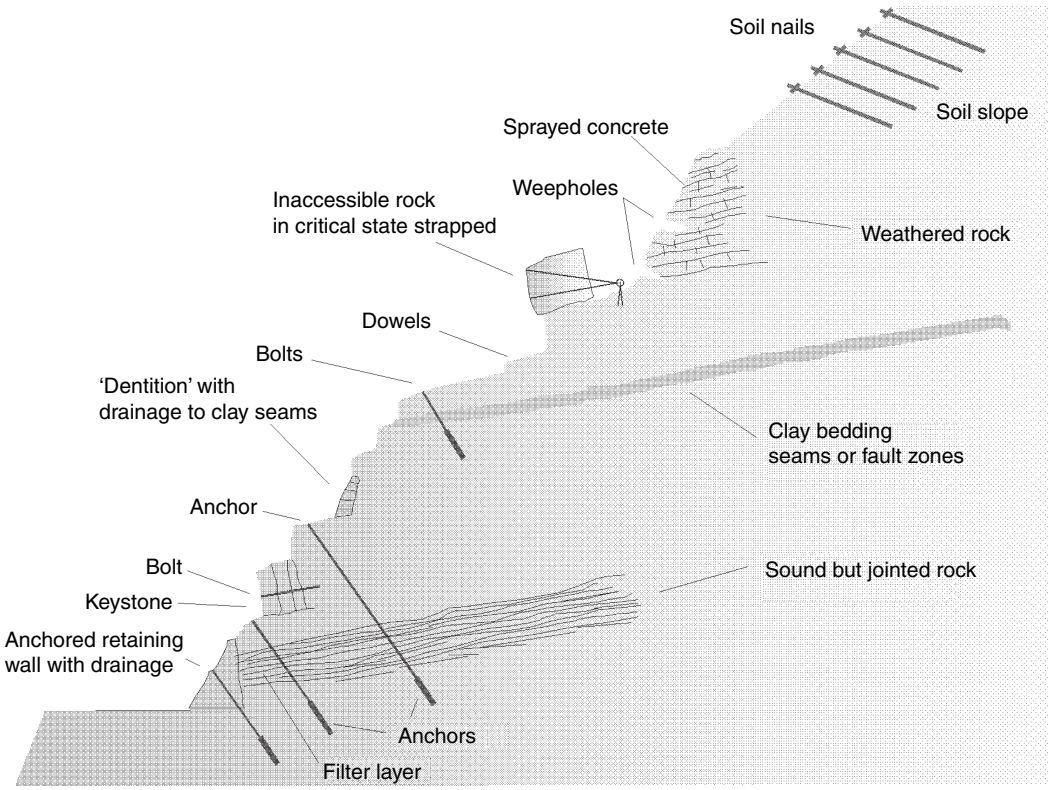


Figure 87.9 Summary of typical engineered solutions
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engineered solutions included the construction of cast *in situ* buttresses, application of sprayed concrete and the installation of rock bolts, dowels and rockfall netting. The works required bespoke drill rigs to facilitate both down-the-hole (DTH) and drifter drilling in sub-vertical locations (see **Figure 87.10**).

87.4 Maintenance requirements

As part of the management of any asset, a maintenance regime should be developed that is specific to the installed stabilisation works. However, in general rock stabilisation works typically require minimal maintenance.

87.4.1 Inspection

It is prudent to instigate a monitoring and inspection regime to ensure that any issues are identified at an early stage. Annual inspections should be undertaken by maintenance staff and comprise a visual inspection from the toe and crest of the rock face. Principal inspections should be undertaken one year following completion of the works and then at three to five year intervals by a geotechnical specialist. These inspections may entail tactile inspection of the rock face and the installed works.

The inspections should identify any significant changes or defects in the face and installed works compared to the as-built photographs and drawings. A written record and appropriate photographs of the inspection and findings should be made and kept in a health and safety or operation and maintenance file.



Figure 87.10 DTH drilling for 100 tonne-capacity multi-strand anchors in Gibraltar
© CAN Geotechnical Ltd

87.4.2 Scaling and rock removal

During a principal inspection, the opportunity should be taken to scale the rock face, removing any rock debris that may have the potential to cause harm to people or infrastructure below.

Rock traps and rockfall netting should be cleared of any arisings to avoid accumulation of rock debris that may damage netting or negate the benefit of the rock ditch.

87.4.3 Vegetation control

Trees and vegetation (including creepers such as ivy) should in general be prevented from becoming established on the face or crest, because roots have detrimental effects on the rock face and vegetation can conceal hazards. Vegetation should be removed and any stumps or roots injected with systemic herbicide.

Where it is desired that vegetation does become established, there needs to be careful consideration of the aspect and drainage of the rock face. Only very shallow-rooted and drought-tolerant species should be promoted.

Shrubs (particularly prickly species such as hawthorn and blackthorn) should be encouraged at the toe of the face as they will deter access to the toe of the face and also act as a catch system for any spalling debris weathering from the face.

87.4.4 Stabilisation works

The maintenance regime should be bespoke to the works installed, with a checklist to ensure a systematic record of repairs is kept. The list below is generic covering typical areas of maintenance only and is not intended for site use:

- Catch fences and rockfall netting should be inspected for signs of impact damage to posts, fraying of cables or damage to the netting.
- Ground anchors should be inspected for signs of corrosion and the tightness of anchor nuts and plates checked. Anchor caps, if installed, should be re-greased and new gaskets fitted. Periodic lift-off tests should also be undertaken on anchors to monitor the residual load applied.
- Condition of sprayed concrete should be monitored so that any damage (concrete spalling or cracking) can be repaired.
- Inclined drains should be flushed and weepholes rodded to ensure that they are free flowing.

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87.5.1 Further reading

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It is recommended this chapter is read in conjunction with

- Chapter 18 *Rock behaviour*
- Chapter 72 *Slope stabilisation methods*
- Chapter 89 *Ground anchors construction*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 88
Soil nailing construction

Philip Ball Keller Geotechnique, St Helens, UK
Michael R. Gavins Keller Geotechnique, St Helens, UK

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Soil nailing is now an established method of providing efficient slope stabilisation and structural retaining wall stability. Soil nails are utilised on a wide range of slope geometries and in variable ground conditions; this has resulted in the development of a variety of innovative materials and construction methods.

The technique involves the installation of a reinforcement bar within a drilled hole which can be grouted before, during or after the reinforcement is inserted. The soil nails are usually arranged on regular grid spacing depending on ground conditions and slope geometry. The surface of the slope is also usually covered with either a hard or flexible facing with a head plate being fixed to the nail reinforcement and bearing on the facing. Soil nail projects also involve the excavation or preparation of the slope face, construction of crest and toe drainage. Quality control includes the load testing of nails.

88.1 Introduction

The soil nail technique may be used to improve the stability of a recently cut slope, an existing unstable slope subject to landslips or a retaining wall formed before or during the excavation process. Soil nails are installed within the soil of the slope providing reinforcement to resist destabilising forces. This chapter outlines the various stages and methods which are required in order to construct a soil nail scheme. The design of soil nails is covered in Chapter 74 *Design of soil nails*.

Detailed planning of the scheme is required to ensure a safe and economical site and construction sequence is established using the appropriate plant. Following the provision of suitable access, the slope needs to be carefully prepared before the nails are installed. Soil nails consist of slender reinforced elements, usually steel bars, and cement grout which are installed on a regular grid within the slope. There are various techniques available for the drilling of the nails, placing of reinforcement and grouting which depend on slope geometry, access and ground conditions. The slope to be treated generally requires a surface facing medium to be placed and the construction of drainage for which there are also several possible solutions. The load testing of the soil nails forms part of the quality control and design verification and requires careful on-site practice.

CIRIA have published a comprehensive report, *Soil Nailing: Best Practice Guidance* (C637) (Phear *et al.*, 2005), which provides detailed information on the design, construction, testing and maintenance of soil nailed walls and slopes and can be read in conjunction with this chapter.

88.2 Planning

88.2.1 Health and safety

Soil nailed slopes tend to have all the ingredients for a hazardous activity: working at height, slippery slopes, manual handling, haul roads close to the face and rotating drill strings. It is essential that the planning phase considers all the concurrent activities and how they impact on one another. Invariably, several different trades are involved and so coordination is required.

On new slopes it is typical for the site/haul road to pass along the face of the works. The drilling rig, and any mobile elevated working platforms (MEWPs), will be positioned close to passing site traffic with potential for clashes. On road widening schemes this is exacerbated by public traffic alongside generating noise and distractions. Adequate separation and management of site traffic is required and personnel must be segregated from the movements as much as possible.

Cut slopes will vary widely but often present wet slippery surfaces that lie between 30 and 60 degrees from horizontal. These are sources of considerable risk if access to nail positions and the crest are not planned adequately. There may be several phases of access required to any one part of the slope. If the slope is excavated in a top-down manner access is required near the crest to install the top row of nails and again at intermediate rows, etc. Then to test sample nails access may be required at random. To fit the facing access will be required from behind/above the crest, down the face and finally, to fit the face plates and tighten them up access will again be required. On each occasion the means of access, the slope surface and the type of work changes. Preparation is vital to ensure these activities are well managed.

One good solution to provide efficient safe access is to use the soil nailing machine to place anchorage points along the crest to which a wire rope can be strung. The operatives can then attach themselves to this with fall-arrest lanyards. Possibly the most overlooked risk is of personnel slipping down the cut face and impaling on the protruding nails.

Where existing faces are to be strengthened or oversteepened similar risks are likely to be present but less severe. Nevertheless, each activity needs to be considered.

88.2.2 Access

Soil nailing is usually best suited to a sequence of tasks from one end of the slope to the other. Consequently, access along the slope should be continuous and have regard for the constraints (if any) imposed by the designers. For instance, it may only be permissible to excavate for one row of nails at a time, which requires a considerable length of site accessible for one shift of work. To allow for curing and the next level of excavation up to three days' production may need to be available along the length of the slope. Therefore, the sequencing, access and deliveries need to be considered to allow efficient working.

Access up the face of the slope is typically achieved using MEWPs. These are currently available with up to 23 m reach and have become the standard tool where the benches are deep or to gain access for the 'finishing' activities once the slope is cut (**Figure 88.1**).

In the case of a high slope where benches are not being cut it may be necessary to resort to specialist drilling equipment that can access the face from the toe. An extreme example is shown in **Figure 88.2**.

Consideration must be given to the means of access for fitting plates, meshing or similar surface covering and testing. Modern test frames need to be located on the slope surface with independent reference beams and access for the technician. The selection of the location of load test and access to it should be planned early. See section 88.9.

88.2.3 Rig types

There are numerous types of rig that can be used for soil nailing, which include:

- tracked hydraulic drilling/mini-piling machines;
- excavator-based drill masts;
- telehandler mounted drill masts;
- hand-held drills;
- air-powered drills – sometimes used with rope access systems.

Provided a hole can be formed that is suitably accurate and straight any drilling machine that can reach will do. Holes can be drilled with purely rotary methods (tri-cone rollers, drag bits or similar) or augered. Where harder ground is encountered percussion is frequently used where the impact energy and a flushing medium are used to penetrate. This is the most common style of drilling for self-drilled hollow bars.

The decisions on type of drilling and type of reinforcement are linked and are sensitive to cost and programme. Occasionally access constraints will also determine the selection. The relatively high cost of self-drilling hollow bars has to outweigh the cost of installing solid bars in pre-drilled holes. With the former, the hole is drilled with the bar that will form the soil nail. On reaching depth it is left in the hole and grouted. Therefore, the speed at which these holes can be drilled is high – typically 180 to 220 linear metres per shift. In the case where the hole is pre-drilled the drilling tools have to reach depth and then be removed before the reinforcing bar is positioned. The extra time is the key factor but the stability of the hole can present a risk. All of the above will influence the choice of drill rigs and equipment.

88.2.4 Working platforms

The working surfaces for new slopes in cuttings tend to be the most difficult to manage. Because the benches cut to provide access to the soil nail entry positions are also the working platforms for the rigs – and are temporary – they can be a source of problems. Trafficking and uncontrolled surface water will soon cause deterioration that poses risks and can adversely affect the quality of the product.

The activities of other trades near the slope have to be allowed for in terms of access and impact on the slope. For instance,



Figure 88.1 Example of long reach personnel access



Figure 88.2 Example of long reach personnel access

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if the temporary slope is only stable over a specific height the excavation of a trench along the toe to provide for services would not be acceptable. It is quite common for communications cables and drainage to run parallel to the toe of the slope and also for gantry positions to require special consideration.

88.2.5 Sequencing

This issue has been referred to above with regard to safety considerations. Each project will need to be considered separately as there are no 'fixed' approaches. When there are repeat visits to a section of slope it can protract the period before the slope is completed. This is particularly true for slopes where the nail installation is followed by acceptance testing, mesh, soil retention geofabrics, topsoiling and then final geofabric covering. In such circumstances the overall stability of the slope and the risk of surface erosion need to be considered. A temporary cover may be beneficial but is obviously extra work.

The application of a mesh, where required, on the surface is a key step. The mesh has to be in place before the nail heads are fitted so therefore, the period between the nails being drilled and the mesh placed can be a 'hold point' preventing further benching.

88.2.6 Spoil/drill arisings

There tends to be little spoil from the drilling of soil nails unless formed in soft clays. Typically bores will be 100–150 mm in diameter and so produce a small volume of waste.

When liquid flush is used to drill the holes there can be a continuous flow from the top of the hole and that requires control and disposal or recycling. In some cases that flush will be grout but frequently it is water. Either way, the final grouting of the hole will cause overspill and so needs to be cleaned off the slope. Such waste needs to be segregated as it is not inert.

88.2.7 Environmental considerations

Most soil nail applications are carried out on slopes with freshly cut faces. Vegetation clearance and reptile control would normally all be in place in advance. Some slopes though may need enhanced stabilisation and so trees, shrubs and surface vegetation remain. Each case needs to be considered on merit to minimise environmental impact and consideration should be given to the aesthetics of the end product.

The issue of spoil from the drilling and grouting is described in section 88.2.6 above.

Some fibrous geofabrics – coir matting in particular – are easily set alight. This can be accidental, deliberate vandalism or inadvertent when cutting the protruding bars from the finished slope. In hot dry periods this has to be considered as it will result in damage to the other facing materials.

88.2.8 Pre-construction approvals

The nature of soil nailing projects means that often any preliminary test is difficult. This may be because the access is not available, the slope can't be cut or even the site is not available. However, the nature of this type of work means that early agreement as to the size, length and construction method is a significant advantage that will avoid delays and aborted work when the main phase starts.

Refer to section 88.10.1 for details of preliminary testing.

88.3 Slope/site preparation

The site access roads, working platform and slope face should be planned in consultation with the soil nail contractor and designer and then constructed and maintained to provide a safe and efficient site for the installation of soil nails.

88.3.1 Topographic accuracy

An accurate topographic survey will be required for both the soil nail designer, setting out engineer and for the soil nail and facing installation contractor. The survey should detail both the overall slope geometry and any localised variations as these can affect nail design, spacing, the setting out and difficulties in placing head facing materials.

88.3.2 Haul roads/platforms

Haul roads to the site should be designed and prepared to allow the safe passage to and from the slope for all the relevant plant that is required to excavate the slope and install the soil nails. The haul roads may have to be frequently moved due to the excavation and construction sequences.

Working platforms should be designed and constructed to ensure the safe operation of the rigs during nail installation. Soil nail rigs can vary in weight from 4 tonnes to 40 tonnes for long-reach machines. It is therefore recommended that platforms are designed, constructed and maintained in line with BR470 *Working Platforms for Tracked Plant* guidelines (Skinner, 2004). The working platforms should be placed to ensure that the rigs provided can reach all the soil nail positions and allow access for nail testing.

Soil nails can also be installed using rope access techniques where the slope topography and access dictates. Consultation with at least one of the several specialist companies that undertake this type of work is recommended during the planning phase.

88.3.3 Slope surface preparation

The method of slope surface preparation depends on whether or not the slope is to be excavated and cut to a steeper angle or whether the existing slope is just to be soil nailed at its existing angle.

Where an existing slope is heavily vegetated and impedes the safe and efficient installation of the soil nails and slope facing, the vegetation should be removed. It should be noted that on some slopes the vegetation is providing a beneficial effect on the stability of the slope and consideration should be given to ensure its removal will not cause instability.

Following removal of the vegetation the slope should be reinstated to provide a uniform surface suitable for the placing

of the facing and head plates. Excavated slopes should be cut as uniformly as possible in order to reduce problems with the placing of facing and head plates. Any very loose material or surface anomalies should be removed.

Guidelines for the excavation tolerances of the slope surface are available and given in CIRIA Report C637 (Phear *et al.*, 2005).

88.3.4 Setting out

Sufficient data for the level and position of the soil nail heads, angle and length of the soil nails should be provided by the soil nail designer to enable accurate setting out and construction. The entry position of the soil nail should be marked accurately using a steel pin in the slope. The slope topography can vary significantly compared to the theoretical design profile: in these cases the setting out of the nails can be difficult and the designer should be consulted to ensure any variations satisfy the design requirements.

88.3.5 Services/land drains

A full survey to establish the existence of services and land drains should be undertaken before the commencement of the works. This is to ensure both the safety of personnel and that the services or drains are not damaged during excavation, drilling or filled with grout up during the works.

88.4 Drilling

The tolerances for setting up and drilling soil nail bores is set out in *Soil Nailing: Best Practice Guidance*, Section 10.6 (Phear *et al.*, 2005). Recommendations given there are summarised below:

- slope face angle deviation = ±2.5°;
- localised tolerance of the slope face = +150 mm/−300 mm;
- positional tolerance at the installation ground surface: ±100 mm horizontally and vertically;
- orientation at the head of the completed soil nail: ±5°;
- orientation of the soil nail along its length: ±2°.

Care is needed in applying these tolerances. Unduly stringent demands can significantly affect the costs and programme but do not provide any benefits to the client. For instance, the accuracy of the cut face in a new slope may not need to be so tightly specified if the design is not too sensitive to local variation and the nail head plate/washer assembly is designed to cope with some variability. Conversely, if a flat sprayed concrete finish is required a variation in the face of up to 450 mm would consume a lot of material. What is essential is that each aspect is considered in the context of the project and specified accordingly.

Aligned to the issue of setting up rigs is the risk of deflection. Thin self-drill bars can readily be deflected on boulders or tree roots and so may not be appropriate if such a deflection were to endanger services or other features. Casings or stiffer bars would have to be used to mitigate the risk.

88.4.1 Open holes (unsupported)

In stable clayey soils, weak rock or similar the bores can usually be formed by drilling rotary techniques or augers. The hole would be drilled to depth, cleaned out and the drill string withdrawn. The bar can then be placed in the hole with requisite centralisers and couplers and finally the hole grouted to the top (see C637, Section 4.0). Alternatively, the bore can be filled with grout first and the bar plunged into it. This is the preferred method if there is any risk of instability.

88.4.2 Cased holes (temporarily supported)

Casing is often used where there is a risk of unstable ground interfering with the installation of the reinforcement (and occasionally where extra accuracy is required). The casing is advanced to the required depth which may only be part way down the hole. The bore is drilled to depth and then the installation of the steel is undertaken as described above. However, the casing then needs to be removed using the drill rig and so productivity is adversely effected. In this instance it would be preferable to grout the bore, remove the casing and thereby release the rig to drill another hole while installing the steel bar.

88.4.3 Drilling with fluids

Some ground will lend itself to drilling using a fluid to support the bore instead of using a casing. The fluid supports the bore's sides but also clears the arisings by flushing them to the surface. The choice of fluid is usually either neat grout or water. Occasionally a polymer may be added. Whatever the fluid it must not be detrimental to the bond of the grout to the ground. That applies equally to some clay soils. If flushed with water they can cause smearing to the sides and prevent a good bond.

88.4.4 Self-drill hollow bars

Usually drilled with grout or water flush the hollow bars form the reinforcement as well as the drill tool. Each bar is drilled into the hole and the subsequent bar added using an external coupler. Drill bits can be attached to cater for a variety of soil types and are available in different diameters. The bar diameter and wall thickness (therefore torsion capacity) limits the size of bit that can be used in any one ground condition. On reaching depth the drill string should be surged up and down the hole a few times to ensure a clear bore. The drill bit is sacrificial in this operation which can prove a costly element, especially in hard ground where tungsten-tipped bits are required.

88.5 Placing the soil nail reinforcement

The essential element for a soil nail is the reinforcing bar. It provides the tension capacity in the ground by countering the strain along its length and by resisting the loads on the head plates. To do this over the design life the grout/bar/soil bond must be effective and the bar must survive any corrosion.

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The head plate and the connection details must be equally resilient.

It is essential that the bore is clean and free of debris and water. The grout, however placed, must form a constant column in the ground completely surrounding the bar. It is not actually critical that the bar lie on the axis of the bore but it must be surrounded by a sufficient thickness of good grout to provide a bond to the soil and in most cases corrosion protection. For example, a plain bar will have been designed to have a thick grout cover (40 mm or so), whereas galvanised or stainless steel nails may not rely at all on the grout for corrosion protection. They still need the bond provided by the grout but not its protection. Refer to Chapter 74 *Design of soil nails*, on design.

Given the variables described above judgement can be made on which elements of the quality are critical and which are not. If it is essential to maintain the bar on the axis of the hole adequate centralisers must be provided at appropriate intervals. In the case of self-drilled hollow bars it can be difficult to achieve a consistent result due to the methods normally used. A series of radial splines arranged around a circular collar are typically used at each coupling. These are loose fitting and allow the bar to rotate inside the collar. When drilling down the centralisers are pushed up against the coupler. However, on completion of the drilling any surging of the bar to flush the hole or any particularly soft areas may allow the centralisers to move and or dig in thus compromising the grout cover. Consequently consideration needs to be given to the finished product and whether the method of installation is entirely appropriate.

Where bars are not self-drilled the centralising can be more controlled and is generally more reliable.

In the event that bars have been specifically coated for their durability care must be taken not to damage that protection system during installation.

88.6 Grouting

In almost all cases soil nails are grouted with a conventional neat Portland cement. The characteristic strength rarely needs to exceed a UCS of 30 N/mm^2 because soil nails are usually used in weak ground. The composition can vary but a water/cement ratio between 0.4 and 0.5 is commonly used for the final article. Frequently though, while drilling, a 'lighter' grout mix will be used to support the bores and flush the arisings effectively.

In particularly aggressive ground other cement types may need to be considered.

The mixing of the grout is important. A properly blended grout needs to be mixed in a high-shear mixer for about three minutes before use. Other types of mixer, such as those that mix dry cement immediately before pumping, must be verified before use to ensure thorough blending is achieved. This can be done by passing the mix over a fine mesh filter and looking for 'lumps' of dry cement.

Bleed, the loss of excess water, has the potential to be a significant problem. When grout bleeds the particles of cement settle, the free water separates and there is a loss of volume. The problem is not at the surface where the effects can be seen (column of grout recedes and may crack on the surface) but below ground where the effects cannot be seen. The results of excessive bleed will include the loss of grout cover, deterioration of its durability and at worst loss of cross-section. Regular bleed testing should be an integral part of the quality control on a project.

The portion of the hole immediately behind the head plate is often the main cause of concern. The grout may settle or shrink at the face while curing. This can be due to bleed as described but also filtration into the host ground. The grout surface is usually sub-horizontal in its liquid form and so a separate operation is required to top up the grout to form a surface parallel with the ground surface (see **Figure 88.3**). In this location the plate and grout need to be in close contact and the bar must be adequately protected against near-surface moisture that would readily corrode an unprotected bar. Obviously, as discussed above, the degree to which this issue becomes critical depends on the environment, the protection system and the design life.

88.7 Completion/finishing

The issue of the final preparation of the grout column near the surface has been covered above.

The head plates form an integral part of the soil nail and so need to be placed and positioned properly to work well. There are few problems with plates (provided the protection is not compromised) but frequently the slope surface is not trimmed well enough to permit a flush fitting of the plates. This is particularly true where weak rock or mixed ground cause an uneven surface to be cut. In such instances the plate is only in partial contact and may put the bar into bending. Hemispherical nuts and tapered washers can provide some flexibility but it is limited. Where necessary the face should be levelled after trimming and patched to provide a suitable bearing surface for the

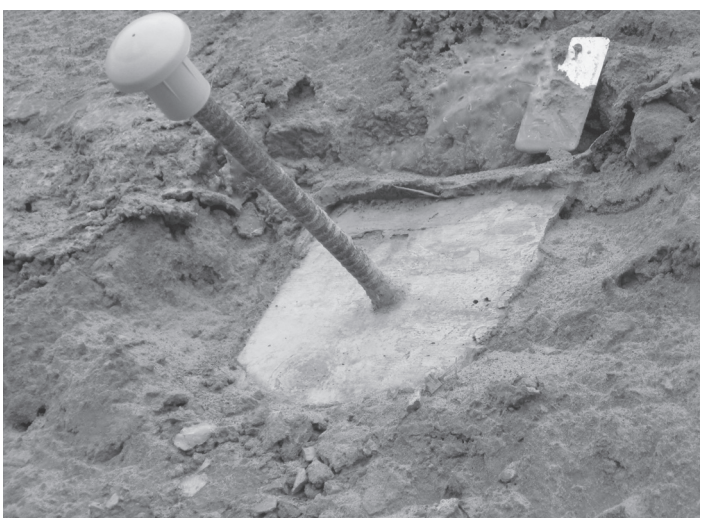


Figure 88.3 Finished grout around a plain solid bar nail

plates. In the worst cases this could require small shutters to hold grout on the slope.

88.8 Slope facing

Slope facing is required to provide resistance to any localised failure or ravelling between the soil nails and also to provide additional bearing support for the nail head plates. In addition slope facing can provide topsoil retention and promote vegetation growth. The slope facing should be specified by the soil nail designer. Various types of facing and methods of installation are available and are dependent on the site topography and design requirements.

88.8.1 Types

The three main categories of slope facing are:

- **Flexible structural facings** – These generally comprise of a metallic mesh which may be coated for corrosion protection and also may incorporate fibrous matting to assist in the growth of vegetation (see **Figure 88.4**).
- **Hard structural facings** – Hard facings comprise reinforced concrete which can be formed using pre-cast panels or more often utilising the sprayed concrete method into which the soil nail heads are embedded or fixed on the external face (**Figure 88.5**). Hard facings may also be adopted to prevent soil erosion or for purely aesthetic reasons. These may consist of solutions such as timber crib walls or stone-filled gabion-like baskets.
- **Soft facings** – The most common forms of soft facings are coir mats or geotextiles which promote the growth of vegetation and help to prevent soil erosion. In addition, the use of cellular topsoil retention facings is becoming frequent as they have the aesthetic benefit of completely covering the nail head plates and providing a good depth of topsoil to promote vegetation.

88.8.2 Installation and joining details

88.8.2.1 Flexible structural facings

The most widely used flexible structural facings consist of a coated wire mesh. The mesh is placed following the installation of the soil nail bar reinforcement and grouting. The mesh is usually supplied rolled and is installed in a top-down sequence. It should be initially fixed at the crest of the slope above the top row of nails and then rolled vertically down the slope. The mesh is fixed to the slope using galvanised hooked pins at regular grid spacings. At the top of the slope it is good practice to provide sufficient facing so it can be anchored above the crest of the slope. The spacing of the pins and pin length (400 to 600 mm long) should be specified by the designer or specialist contractor and will depend on slope geometry and soil conditions in the near surface and are installed by hand. The soil nail head plates will then be placed over the mesh on to the nail reinforcement and secured by a locking nut. This helps to fix the mesh.

Where the slope is to be cut top-down in staged excavations it may be necessary to install the facing in the same sequence

(a)



(b)



Figure 88.4 (a) Nearly finished slope showing nail plate and fibrous matting under mesh; (b) Galvanised hollow bar with plate over mesh with coir matting below

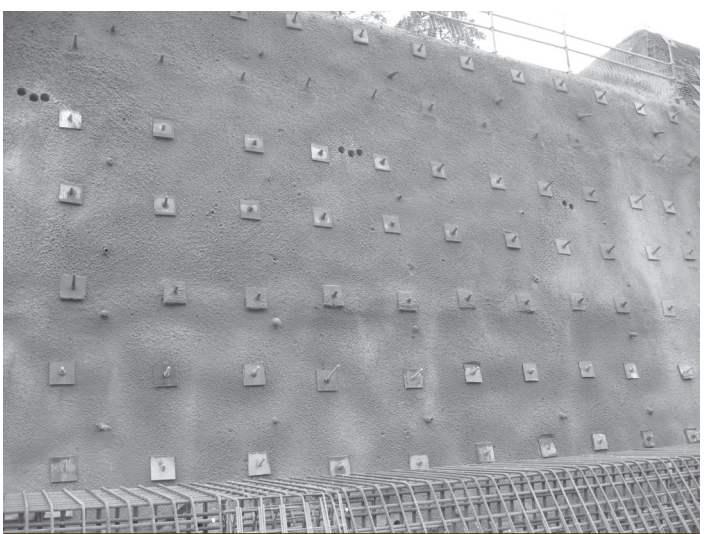


Figure 88.5 Near vertical hard facing (note the cored UCS core sample holes)

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also leaving the roll of mesh securely fastened above the next row of nails to be installed.

At joints to adjacent strips of mesh a suitable overlap should be provided and the meshes joined using ties or clips to provide a structural join.

The use of MEWPs can simplify the procedure and provide a safe access platform for the personnel installing the meshing and head plates (see **Figure 88.1**)

88.8.2.2 Sprayed concrete

Sprayed concrete involves the placing of fresh concrete on a slope surface using a high-pressure spraying technique. The slope surface is first cut to the required angle and this should be cut as accurately as possible as filling voids with sprayed concrete can be uneconomical. Following installation of the soil nails the facing material, usually steel mesh, is placed against the slope surface ensuring spacers are placed at sufficient intervals to provide the required cover. The soil nail head plates can be installed before or after the sprayed concrete is placed depending on whether they are to be incorporated into the concrete or left exposed on the surface.

Concrete is then sprayed onto the surface around and through the reinforcement and head plates (**Figure 88.6**). Depending on the thickness of the concrete facing specified this can be done in one or more layers. Sprayed concrete can either be placed using wet or dry mix methods. It is recommended that a specialist sprayed concrete contractor is consulted during both the design and planning stages to ensure a practical solution is specified.

Sprayed concrete is generally used on steeper or even vertical cuttings (**Figure 88.5**). The stability of the slope is therefore often more critical and the sequencing of the excavation, nailing and sprayed concrete should therefore be given careful consideration. It may be necessary to cut the slope in discrete panels on a hit and miss basis and install the soil nails and sprayed concrete before excavating the adjacent section. In this case an allowance for the lapping of the sprayed concrete reinforcement should also be made.

88.8.2.3 Facings

Various facings can be installed in conjunction with soil nails including timber crib walls, modular blocks and gabion basket facings and these will be constructed following the completion of the excavation and soil nails. The facing is normally connected to the soil nails and is installed from the base of the slope. Specialist suppliers and contractors will provide facing materials, guidance on installation and design and if required full construction for these types of facings (**Figure 88.7a** and **88.7b**)



Figure 88.6 Applying sprayed concrete to vertical face (note the double mesh and drainage holes)

(a)



(b)



Figure 88.7 (a) Construction of timber crib facing; (b) Soil or stone panel facing
Courtesy of Phi Group

88.8.2.4 Soft facings

Soft facings are installed in a similar manner to the flexible facings as described in section 88.8.2.1.

88.8.3 Soil retention systems

Cellular soil retention systems are installed following the completion of the soil nail head plates and may be placed over flexible structural facings. They are fixed to the slope using steel pins in a similar fashion to the facing (**Figure 88.8**). Alternatively a narrow steel mesh cage, or soil panel, can be used to contain the topsoil. Topsoil can then be placed by normal excavators before *hydro seeding*.

88.8.4 Vegetation: hydro seeding

Hydro seeding (the placement of a seeded solution by spraying) is generally carried out by specialist landscaping companies. This operation should be carried out following the placing of all the slope facings and topsoil if required.

88.9 Drainage

The stability of a slope is usually highly dependent on the groundwater levels within the slope strata and it is therefore essential that any drainage systems specified are constructed properly to ensure they are fully operational.

88.9.1 Crest and toe drainage

If crest or toe drainage channels or trenches are specified they are usually installed using standard excavation and drainage methods. There are, however, increased risks working above and below a slope whilst excavating and precautions should be taken to ensure the slope stability is not compromised.

88.9.2 Slope surface drainage

For most soil nail schemes the slope surface drainage occurs naturally through any meshing and vegetation. Where an impermeable hard facing such as sprayed concrete is specified drainage measures can consist of weep holes and/or a drainage fabric placed between the facing and soil on the slope. Weep holes can be formed using plastic pipe and a filter membrane and fixed to the mesh before sprayed concrete is placed. Weep holes can also be drilled through the sprayed concrete after it has been constructed and hardened.

The drainage fabric should be placed and pinned to the slope surface before the placing of any mesh reinforcement and sprayed concrete.

88.9.3 Horizontal or subsurface drainage

The design of the slope may also require the installation of horizontal drains which can assist in the control of groundwater pressures. The drains usually consist of a slotted uPVC pipe with a geotextile membrane which is placed inside a pre-drilled bore. The drains should be inclined 5° to 10° upward to allow the water to drain to the slope face. The drains can be installed by the same rigs and personnel as the soil nails especially for cut slopes where the excavation is undertaken in stages (**Figure 88.9**).

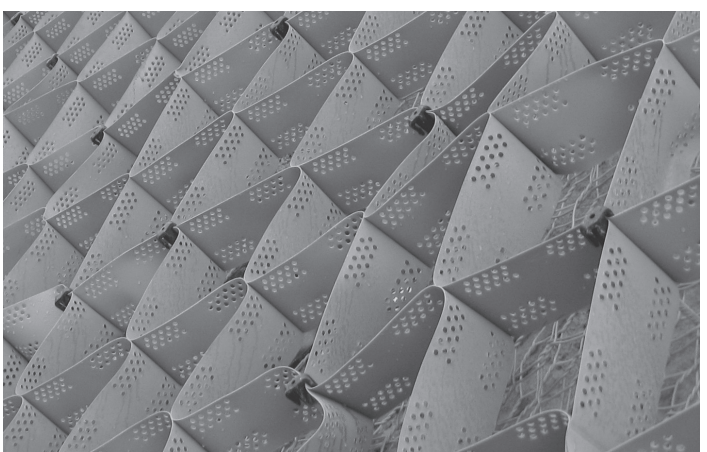


Figure 88.8 Soil retention system on 60 degree slope (note the holding down pins)

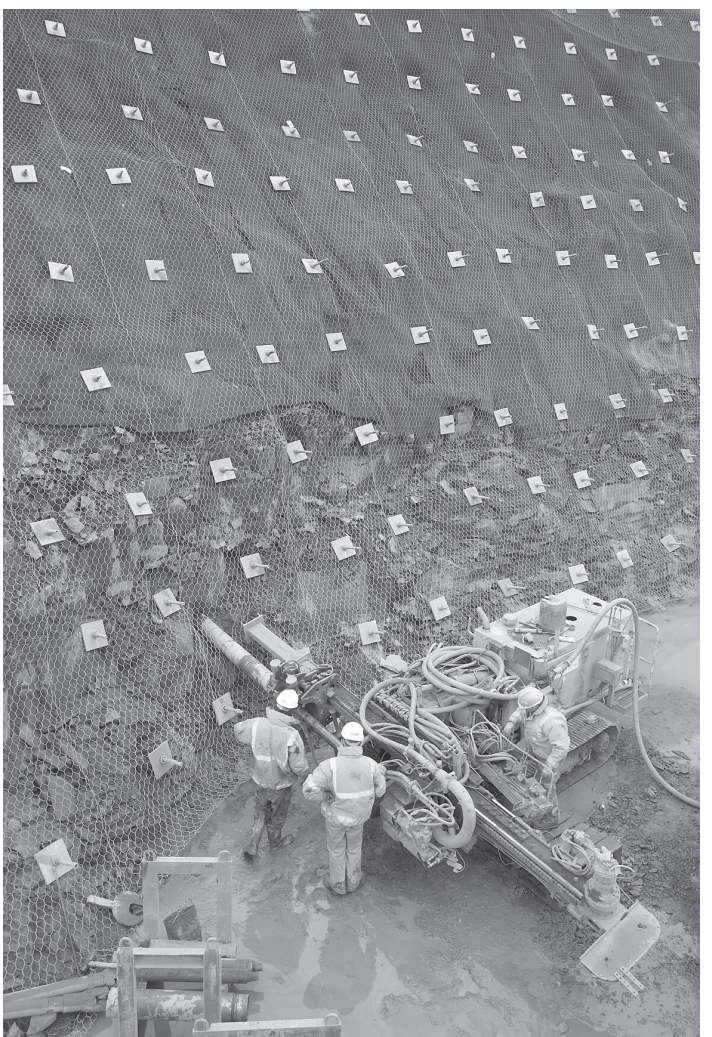


Figure 88.9 Installing deep drains into rock face

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A recent innovation by Ischebeck™ involves the use of a horizontal drain installed using self-drilling hollow techniques and a permeable grout mix.

88.10 Testing

Soil nail performance can be affected by variations in ground conditions and soil nail installation methods. Soil nail testing is therefore required to verify that the nail–grout–ground interaction performance is in accordance with the design assumptions.

88.10.1 Preliminary and acceptance testing

The type, frequency, location and loading schedule of the load test should be specified by the soil nail designer (see Chapter 74 *Design of soil nails*). Guidance on this can also be found in Section 11.3 of CIRIA 637.

Preliminary soil nail tests are carried out in advance of the main works usually on sacrificial nails installed using the same technique as the proposed working test nails. Depending on the site conditions and topography it may be necessary and more efficient to undertake vertical nail tests which can still demonstrate the nail–ground bond in the same strata as the working nails.

Acceptance or working nail tests are undertaken on production or working nails which form part of the permanent works. These tests are carried out during or after the main working nails are installed using similar testing methods as for the preliminary nails.

88.10.2 Reaction system

A suitable reaction frame should be provided to transfer the stressing loads into the slope or ground adjacent to the nail. The actual detail of the reaction frame will depend on the slope topography, ground conditions and test load to be applied. These should be considered carefully for each project. A typical beam test set up is shown in **Figure 88.10**.

The reaction system design should take into account the test load and condition of the slope surface. It is essential to ensure the reaction system is safely secured to the slope.

88.10.3 Monitoring

The extension of the soil nail during the load test should be accurately monitored. The actual method of measuring the extension will vary depending on the load test location and the topography of the slope (**Figure 88.11**). Ideally a displacement (dial) gauge or transducer should be placed on the exposed soil nail bar end and supported on a frame which is not subject to movement from the stressing operation or other factors (see **Figure 88.11**).

The stressing load should be measured using a calibrated pressure gauge connected to the jack.

88.10.4 Free length formation

The designer may specify that the soil nail to be tested should be de-bonded over part of its upper length in order to prove the nail capacity in a certain strata or level in the resistant zone. The de-bonding can be carried out using a plastic sheathing between couplers which should be sealed against grout intrusion.

88.10.5 Approvals

For preliminary tests sufficient time must be allowed in the construction programme for the contractor and nail designer to be able to review the soil nail testing and make any required changes to the design before the main works commence. The working test results should be reported, reviewed and approved as soon as practically possible after the tests so any consequential actions can be implemented minimising possible delays and costs.



Figure 88.10 Load testing frame mounted on a slope

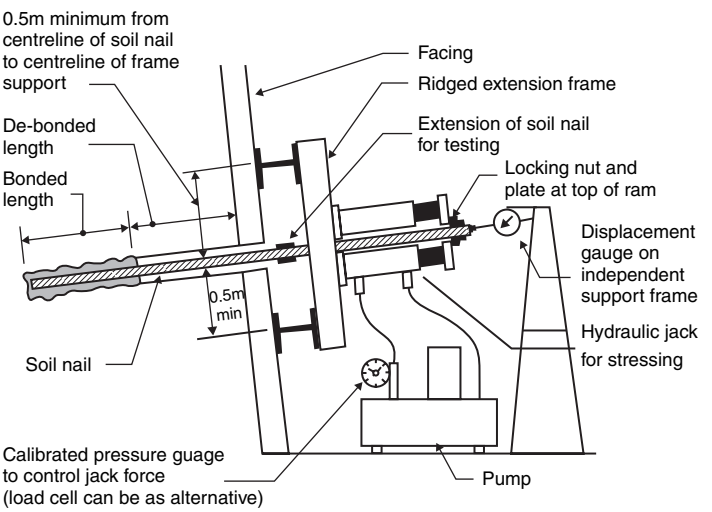


Figure 88.11 Schematic layout of a soil nail testing system
Reproduced with permission from CIRIA C637 Phear *et al.* (2005), www.ciria.org (originally developed from Pr EN14490:2002)

88.11 References

Phear, A., Dew, C., Ozsoy, B., Wharmby, N. J., Judge, J. and Barley, A. D. (2005). *Soil Nailing: Best Practice Guidance*. CIRIA Report C637. London: Construction Industry Research and Information Association.

Skinner, H. (2004). *Working Platforms for Tracked Plant: Good Practice Guide to the Design, Installation, Maintenance and Repair of Ground-Supported Working Platforms* (BR470). London: BRE Press.

88.11.1 Further reading

British Standards Institution (2010). *Code of Practice for Strengthened/Reinforced Soils and Other Fills*. London: BSI, BS 8006-1:2010.

British Standards Institution (1995). *Code of Practice for Strengthened/Reinforced Soils and Other Fills*. [partially replaced]. London: BSI, BS 8006:1995.

Bye, G., Livesey, P. and Struble, L. (2011). *Portland Cement* (3rd Edition). London: ICE.

Pr EN 14990. *Execution of Special Geotechnical Works – Soil Nailing* [Draft].

88.11.2 Useful websites

Deep Foundations Institute (DFI), USA; www.dfi.org/
Federation of Piling Specialists (FPS), UK; www.fps.org.uk/

It is recommended this chapter is read in conjunction with

- Chapter 72 *Slope stabilisation methods*
- Chapter 74 *Design of soil nails*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 89

Ground anchors construction

John Judge Tata Steel Projects, York, UK

Ground anchors are an important geotechnical component in the resisting of destabilising forces in structures and slopes. The construction process is of fundamental importance in the performance of the ground anchors and the industry offers many contractor-designed systems. The history and experience of anchoring contractors is critical in terms of the use of drilling and grouting systems and their effect on the surrounding ground conditions. The long-term corrosion resistance of the anchor system is important, together with the long-term monitoring and maintenance of the anchor heads.

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89.1 Introduction

Ground anchors are tensile members that allow load transfer from a structural face to a discrete zone of ground. The particular ground where the load transfer is carried has been previously examined and defined through good ground investigation and survey works, and targeted as part of the preliminary anchor design process. This transfer of load to a particular stratum of ground makes the ground anchor approach different from systems such as the rock bolt, soil nail and tension micropile (where load is generally carried along the full length of the structural member).

Ground anchors can be installed for loads from 10 kN to in excess of 5000 kN. However the normal range of loads tends to be 100–1000 kN for permanent systems. The ground anchor is commonly used to support retaining structures, resist anti-flotation and earthquake loads, and for slope stability schemes. The ground anchor is also used to improve rock face stability with similar techniques used for soil nail installation (see Chapter 88 *Soil nailing construction*). The design of these systems is closely linked to the construction process, hence good understanding is required between the designer, contractor and specialist anchor contractor when using this system within a project design.

This chapter reviews the following aspects of ground anchors:

- applications of ground anchors;
- types of ground anchors;
- ground anchor tendons;
- construction methods in various ground types: cohesive and granular materials, and weak and strong rock;
- ground anchor testing and long-term maintenance of anchor systems.

89.2 Applications of ground anchors

Ground anchors in the United Kingdom are generally installed based upon the requirements of BS 8081:1989 *Code of Practice for Ground Anchorages* (BSI, 1989). However the BS EN1537: 2000 *Execution of Special Geotechnical Work* (BSI, 2000). *Ground Anchors* also reviews construction practice.

Ground anchors (or anchorages) consist of an anchor head, free anchor length and fixed length (see **Figure 89.1**). Each part requires a detailed design based upon the structure type and the expected ground conditions (see Chapter 66 *Geotechnical design of ground anchors*). The design of the fixed anchor length (bonded length) where the load transfer occurs has a particularly important link to the construction process.

One of the key aspects of ground anchor design is that specialist ground anchorage contractors must be consulted at an early stage to advise on construction issues and how they can influence the proposed design. In addition, where specialist ground anchorage contractors are proposed, the design engineer and main contractor must be satisfied with the experience of the installer. Failure to follow this approach may lead to construction and performance problems with the ground anchors.

The design of fixed lengths is summarised within BS 8081: 1989 (BSI, 1989) and is covered in detail in Chapter 66 *Geotechnical design of ground anchors*, and is based upon the construction process. This is summarised as follows:

Type A anchor	Straight-shafted anchors where uniform bond stress is developed
Type B anchor	Low-pressure grouted anchors where enhancement of the ground strength occurs; increase in diameter of fixed lengths achieved through permeation and compaction of the granular material
Type C anchor	High-pressure grouted anchors (tube à manchette systems) where the ground is hydro-fractured causing grout rooting into the ground
Type D anchor	Under-reamed (or multi-bell) systems where the fixed length drill diameter is enlarged through a system of bells

The construction of the ground anchor involves one or more of the above processes and these are referenced throughout this chapter.

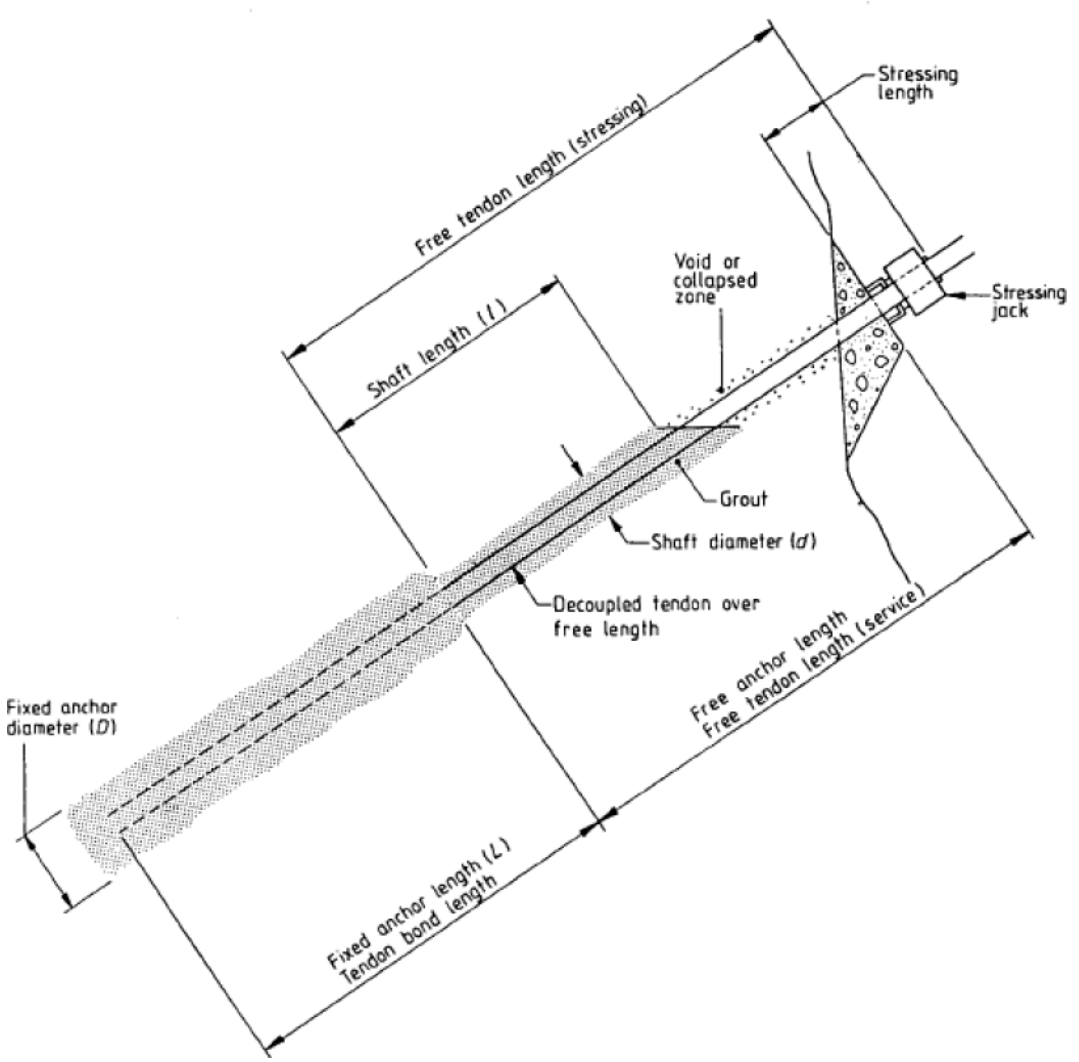


Figure 89.1 Ground anchor nomenclature for a grouted anchor
Reproduced with permission from BS 8081 © British Standards Institution (1989)

89.3 Types of ground anchors

Ground anchors can be divided into either mechanical or grouted-type installations.

89.3.1 Mechanical ground anchors

Mechanical anchors are generally used for low capacity anchors. The resistance is generated by driving in the anchor tendon to depth and forming a load transfer mechanism with a plate system; two examples are the Platipus system (see **Figure 89.2**) and the Duckbill system.

The advantage of these systems is fast installation, no requirement for curing of grout and instant load application. However the design method for these systems is semi-empirical with the load capacity based upon the expected ground conditions and the structural loads required by the designer. These systems are suitable for small-scale installations such as masonry wall repair, foundation anchors and slope support.

However, caution needs to be exercised with these systems as confirmation of acceptance does not conform to BS 8081:1989 (BSI, 1989) in terms of testing and corrosion protection requirements. The use of such systems in permanent works therefore needs careful consideration with regard to the long-term performance and the factor of safety involved. Typical systems are designed for working loads of 10–120 kN; however, it is rare that these are tested to the full three times working load required by Table 2 of BS 8081: 1989 (BSI, 1989). Actual factors of safety on the system would therefore require preliminary tests on site to give confirmation of the levels of safety achieved.

Mechanical anchors are generally driven into the ground to the required length using pneumatic hammer systems. Due to this method of installation, they are particularly susceptible to obstructions within the ground. The key difference with other geotechnical products is that the only section of the ground anchor that can be shortened is the fixed anchor length

(following detailed site trials). The ground anchor must achieve the free length detailed by the designer to maintain overall stability of the structure (see Appendix D of BS 8081:1989; BSI, 1989). Hence, if obstructions are encountered, pre-drilling works may be required to allow installation. Due to these problems, these systems tend to be installed in relatively low strength fine-grained soils of a homogenous nature. It is important that specialist suppliers and installers of these systems are consulted if this method is to be employed.

89.3.2 Drilled and grouted ground anchors

Drilled and grouted ground anchors are generally the most common form of ground anchor installation for large civil engineering projects. These systems allow the designer to use developments in drilling technology as well as specialist grouting techniques. Both these areas of construction have a significant influence on the load carrying capacity of the ground anchor in its final condition (and to achieve the factors of safety required by the Design Codes).

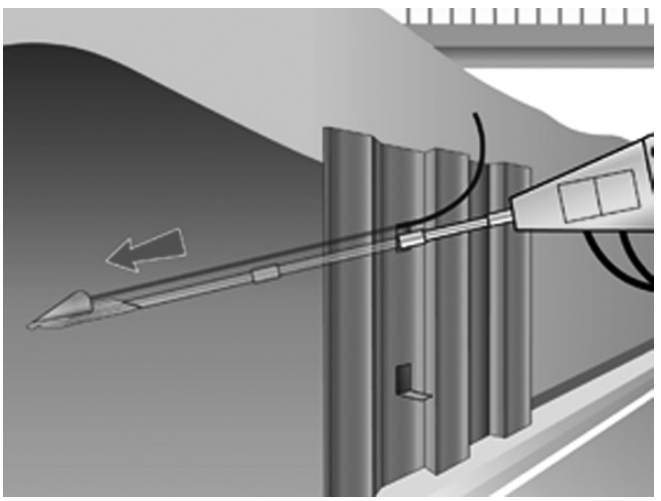




Figure 89.2 Platipus mechanical anchor system
Courtesy of Platipus Anchors Ltd

The drilled and grouted system generally involves the formation of the anchor borehole using the following: open hole drilling techniques, rotary duplex cased systems, rotary percussive cased systems, or down-the-hole hammer systems. Significant variation and overlap of these systems can be used and the type of drilling system generally depends on the type and strength of ground expected over the anchor length. These systems are explored further within this chapter in relation to different types of ground encountered.

On completion of the borehole, an anchor tendon is installed to the base of the borehole and grout introduced using a tremie system. This allows full grouting of the borehole and prevents the occurrence of voids within the anchor bore, which could lead to failure of the anchor. Where temporary casing has been utilised, this is gradually withdrawn whilst continually topping up the grout level.

Based upon the tendon arrangement, and the drilling and grouting methods, a number of ground anchor systems have been provided by specialist contractors and suppliers. Examples of these types of developments are single-bore multiple anchor (SBMA) systems, multi-stage multi-bell/under-reamed ground anchors and tube à manchettes. All these systems allow modification and efficiencies in the load transfer mechanism compared to conventional anchors, and in particular allow high anchor loads to be obtained in poor ground conditions.

89.4 Ground anchor tendons

The anchor tendon generally consists of either a high strength steel bar or woven wire pre-stressing strands (see **Figure 89.3**). The tendon system is factory fabricated to avoid contamination of the units on site. This is especially important with regard to the corrosion protection system (for permanent anchors) and the greasing and free movement over the anchor free length.



Figure 89.3 Factory fabricated double plastic corrosion protected ground anchors
Courtesy of Keller Geotechnique



Figure 89.4 Storage frame
Courtesy of Keller Geotechnique

The factory-fabricated anchor tendon is delivered to site in a secure storage system to allow installation (see **Figure 89.4**). Any corrosion damage to the anchor tendon during transportation or installation could affect its long-term performance.

89.5 Construction methods in various ground types

89.5.1 Construction issues and risks

The key area to consider in ground anchor construction is the effect the installation method will have on the load carrying capacity of the ground anchor. The preference with most ground anchors is to use a construction method that allows formation of the fixed length to the diameter of the borehole. Although enlargement of the fixed length (by techniques such as grout enhancement or under-reaming) is accepted, great control of these processes is required.

Where ground conditions are such that actual grout loss can occur, this can be detrimental to the construction and performance of the ground anchor. Grout loss should be carefully considered within materials such as chalk (where solution features are expected) and Coal Measures (due to past mining activity and fissured bedrock). Methods to deal with this problem include the use of thixotropic grouts or pre-grouting works prior to construction, i.e. a two stage process.

The effect of the flushing medium on the ground and the knock-on effect on the load transfer mechanism can also be detrimental. Careful consideration of the flushing medium and its control is required – not only from a design point of view, but also from an environmental and control perspective. In certain situations, poor control of flushing systems can cause severe effects on adjacent structures and ground types.

Careful consideration is required regarding the tolerances and location of as-built anchors. Ground anchor construction is now routinely used in areas where retaining walls support structures that are supported on piles. The ground anchor system can be designed to pass between piles and be installed at inclinations (both vertical and horizontal) that pass with safe clearance to the deep foundations. Tolerances quoted by BS 8081:1989 (BSI, 1989) are:

- entry point ±75 mm;
- alignment at entry point ±2.5°;
- borehole deviation during construction 1 in 30.

These tolerances are generally achievable, but can often be improved to allow greater control of anchor location. The location of as-built anchors is also important in terms of construction design and management (CDM) requirements, and for future construction works causing disturbance and possible failure of the ground anchors. This has been especially common in the Docklands area of London where new developments utilising piling techniques have had to be carefully monitored in relation to the effect on existing ground anchors to the river walls. Where concern is raised regarding anchor location and possible interference, accurate borehole alignment monitoring techniques can be used as part of the construction works. A precise understanding of the accuracy of these techniques is fundamental in the choice of system and data management.

89.5.2 Construction of fixed anchor lengths within clays

Ground anchor construction in clays should be viewed with some caution, and detailed assessment of strength development with depth should be carefully considered at the design stage. Design of fixed lengths within clay deposits tend to start with the Type A anchor system approach. This is very common within over-consolidated clays such as London Clay, Oxford Clay and Gault Clay found in the southeast of the United Kingdom.

To support loads in excess of 100 kN, the clay needs to be of firm (but heading towards stiff) consistency (BS 5930:1999). Clay strengths less than 75 kN/m^2 should be treated with caution in terms of anchor design and construction, although some success has been achieved in softer soils with membrane expansion-type anchors. The limitations on fixed anchor lengths due to the effect of progressive de-bonding mean that they are generally limited to less than 10 m. This limits the load carrying capacity of the ground anchor, hence the development of the under-reamed system. Littlejohn (1977) details the under-reamed anchor (or multi-bell) system which gives a load carrying capacity of up to five times more.

Even with this increase, under-reamed anchors have limited use outside of the London area. This is due to the requirement of careful quality control of the under-ream and demonstration of good construction. The influence of granular material within clays or low strength clays can cause instability of the bore during construction. However, the system should not be discounted as it has major advantages where issues with land take and boundary constraints on anchor length exist.

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One system that has allowed higher capacities to be achieved with clay deposits is the SBMA system (Barley, 1997). This system allows the fixed length construction to exceed the maximum of 10 m used by single encapsulated lengths, and hence fully utilise the increase in strength with depth of clay deposits. The disadvantage of this system in clays is the large anchor lengths and the effects on boundary constraints. A schematic of the differences between conventional and SBMA fixed length load distribution is shown in **Figure 89.5**. Another similar system is the DYWIDAG (acronym for Dyckerhoff & Widmann AG) multi-stage anchor tendon.

The most common form of anchor construction in clay is the use of auger drilling systems (see **Figure 89.6**) where the drill string helical causes the material to pass to the surface for disposal. This system is quick and allows the borehole to be constructed with minimal use of temporary casing. However, problems can develop where groundwater is encountered within clays, an example being mudstone bands in London Clay. The drilling system does not allow for removal of the water and this can create a slurry which can rapidly reduce the clay strength and grout ground bond stress properties. This can also prevent installation of the ground anchor as the borehole cannot be cleaned to allow anchor tendon installation.

For clays that can contain granular material and possible groundwater, rotary duplex or rotary percussive systems are often employed (see **Figure 89.7**). These are commonly used in the boulder clays and glacial tills encountered within the northeast of the United Kingdom where full length casing can be employed, together with a water flush to maintain stability of the borehole. Water-flushed drilling techniques ensure that a positive hydrostatic head is maintained within the bore which will prevent 'blowing' conditions.

For applications where drilled and grouted anchors do not achieve the required factors of safety on working load



Figure 89.6 Auger drilling in London Clay
Courtesy of A. D. Barley/Geoserve Global Ltd

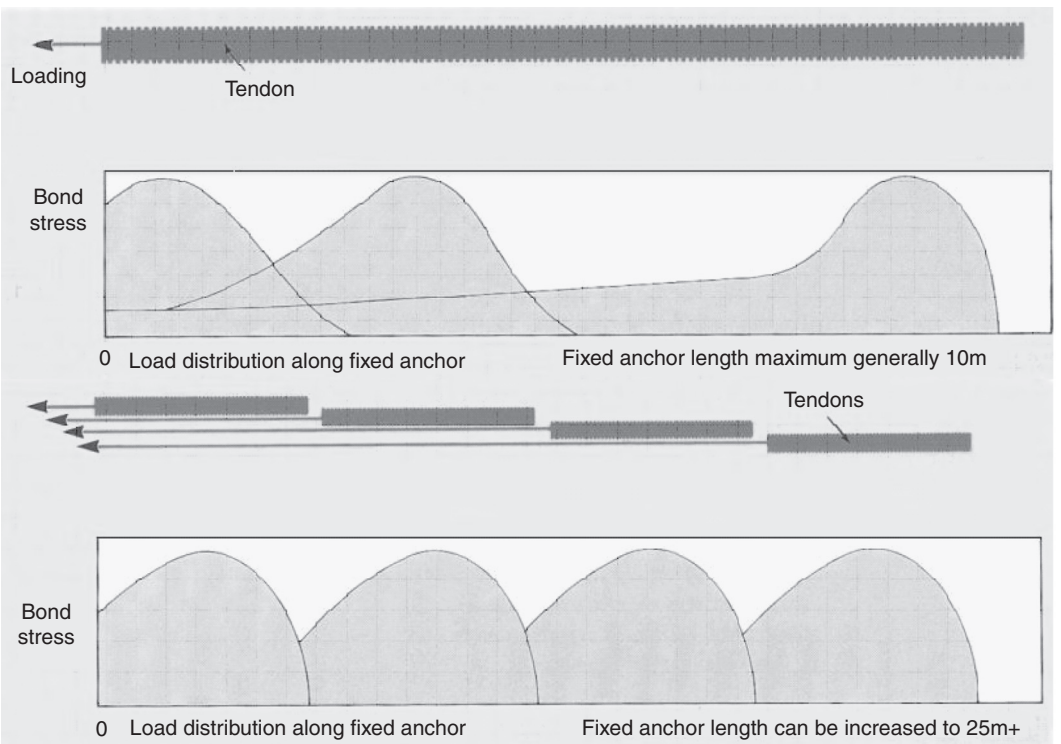


Figure 89.5 Load transfer of conventional and SBMA systems
Courtesy of A. D. Barley/Geoserve Global Ltd

requirements, high pressure grouting techniques have been used. This involves the use of tube à manchette systems with high pressure grouting, using packers. This system can cause hydro-fracture of the clay and allow some grout penetration into the clay (see **Figure 89.8**).

89.5.3 Construction of fixed anchor lengths within sands and gravels

The construction of fixed anchor lengths within sands and gravels are almost all exclusively based upon low pressure grouting techniques (Type B anchors). Low pressure grouting of the high strength grout uses the fully cased system to increase the borehole diameter of fixed lengths through permeation and compaction of the granular material (see **Figure 89.9**). With limitations of fixed lengths of 7–8 m due to progressive de-bonding, permanent anchor loads of 350–450 kN can be achieved within marine sands commonly found within the United Kingdom in coastal areas. However, for coarser deposits which are proven to be predominantly gravels (e.g. terrace gravel), permanent anchor loads of 600–800 kN can be achieved. Utilisation of the SBMA system allows these loads to be exceeded for permanent anchors with working loads of up to 1200 kN – achieved in the Cotton Beds of Great Yarmouth.

The key installation method for ground anchors within granular materials is to have the casing closely follow the drill bit, with a good control on the flush system. The use of water flush is common as this minimises disturbance to the material, and is a safe flushing system in terms of re-circulation (with the use of Siltbuster technology). However, careful monitoring is required to stop 'water jetting' where the soils are loosened and allowed to collapse – creating loose zones. This can cause settlement of the ground surface and affect adjacent structures. The use of air flush is not recommended in granular soils due to the aggressive nature of the flush and the migration of the air through the ground – which again may affect adjacent structures.

Ground anchor construction within alluvial and marine sand deposits generally involve river wall and harbour wall support. Typical installation plant used is illustrated in **Figure 89.10**.



Figure 89.7 Rotary duplex drilling in Lias Clay
Courtesy of Keller Geotechnique



Figure 89.8 Post-grouted fissures in stiff clay sands
Courtesy of A. D. Barley/Geoserve Global Ltd

89.5.4 Construction of fixed anchor lengths within chalk

The United Kingdom contains large areas of chalk where major infrastructure works have been carried out. This generally is around the southeast of England, Norwich and Humberside. Anchoring in chalk carries a greater degree of risk compared to other soils due to the high variation in weathering, and the strength and presence of solution-related features (causing excessive grout take).

Barley *et al.* (1992) reviewed works completed at Castle Mall in Norwich where pre-grouting works were carried out using a sand cement grout prior to re-drilling and anchor installation. Grout volumes of three to seven times the theoretical borehole volume were recorded for the test anchors using this technique, due to the existence of solution features. Careful



Figure 89.9 Low pressure grouted anchor. The casing shoe on the left was used to drill the section on the right using anchor tension. The exhumed grout body on the right shows the expansion of the grout body through pressure grouting
Courtesy of A. D. Barley/Geoserve Global Ltd

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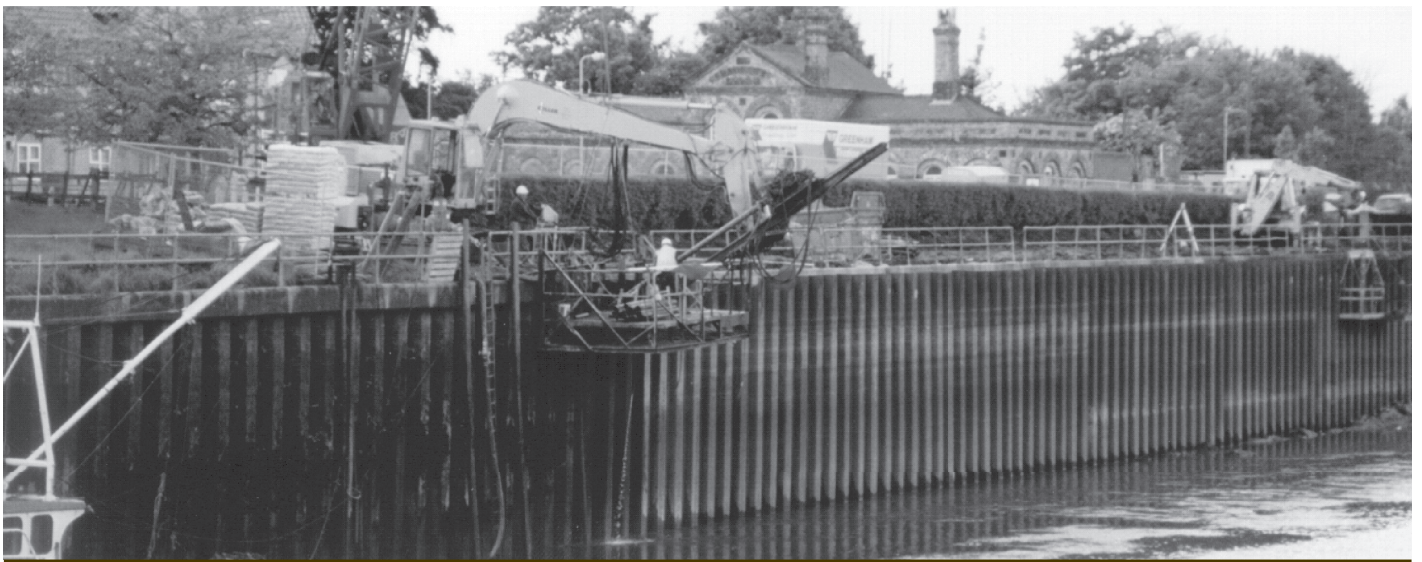


Figure 89.10 Hymac and caterpillar machine modified for under-slung drilling
Courtesy of Keller Geotechnique



Figure 89.11 Exhumed grout body within chalk using Type B installation – showing grout penetration
Courtesy of Geoserve Global Ltd

drilling and pre-grouting is required within the chalk as the use of water flush can trigger ground subsidence from collapse of the solution feature.

Construction methods used within chalk are generally end-of-casing grouting techniques (Type B), utilising rotary duplex or rotary percussive drilling methods. The use of these techniques, the structure of the chalk and the increased grout take cause expansion of the fixed length diameter (see **Figure 89.11**).

89.5.5 Construction of fixed anchor lengths within rock

Ground anchors are commonly associated with rock anchors used within rock cutting support or anti-flotation anchors. The question 'when is a soil classified as a rock?' is very much open to debate, especially in ground anchor construction. This is always a borderline discussion in terms of chalk type, based upon the weathering profile of the material. Ground anchors within the United Kingdom can be sub-divided by its strength and geological history.

Problems of anchor construction within weak bedrock in the United Kingdom generally revolves around mudstones and siltstones such as Coal Measure deposits, Mercia Mudstones and Lias Clay deposits, which tend to cross the country from the Severn Estuary to the north of England. For very weak mudstones, the drilling process can have a major impact on the bond stress properties of the material, hence caution should be taken with regard to over-estimated designs. It is recommended that very weak mudstones are designed based upon a clay soil approach.

Because of the unpredictable nature of weak rocks, the use of water, air or auger drilling techniques should be carefully considered regarding the effect they have on the structure and bond properties of the rock type. Auger drilling can create a smear effect on the ground interface (especially in mudstones) which can rapidly reduce load carrying capacity of the ground anchor. Air flush can cause migration along open fractures and affect adjacent structures. The use of air can also cause disintegration of the bedrock, reducing bond stress properties.

Water flush drilling can cause localised softening, especially where boreholes are left open for a period of time. The use of anchor contractor experience and trial anchor testing are important parts in demonstrating constructability in weak rocks.

One major problem with forming ground anchors within weak, weathered rock, is the inter-bedded nature with stronger, more competent materials. This is especially common with Coal Measures where bands of strong sandstone can be encountered. Also, the Mercia Mudstones can be inter-bedded with strong bands of limestone. The drilling system in these conditions is generally chosen for the strongest element, hence when anchors are formed within these materials, variation in the positioning of the fixed length in the rock type can be a problem. In these materials, careful review of construction methods compared to design is required. Detailed review of testing within mudstones is covered by Barley (1988).

For stronger, more competent bedrock, i.e. igneous deposits in Scotland, Wales and southeast England, limestone, Sherwood Sandstone and metamorphic rocks, down-the-hole hammer systems are generally used to form the anchor boreholes. These systems are efficient because the energy of the drill is directly applied at the drill bit (and not at the drill head, as with rotary percussive drilling). This is especially important in drilling strong rocks as the rate of penetration is quicker than 'top' drive systems.

Down-the-hole hammer systems use air flush to return the drill spoil to the surface, hence the calculation of uplift velocity compared to depth drilled is extremely important. These drilling systems rely on air compressors to have the capacity to return the spoil, so the diameter of hole compared to the rod size needs careful review. If the air gap between the rods and the hole diameter is too great, the returns will not be expelled at the surface and will affect the ground anchor construction. These systems have one key problem compared to water flush systems – control of the spoil is difficult due to the airborne nature of the returns. Head collecting systems or reverse circulation systems can be used to control the material. Symmetrix and MaxBit down-the-hole hammer overburden drilling systems are becoming increasingly popular due to their speed and ability to bore through difficult ground.

Drilling within igneous deposits that have a high quartz content can cause extreme wear on the drill bits, which is accelerated when drilling in rock that contains saline ground water. Careful review of these systems is required. Large high capacity ground anchors are used within dam construction and dock wall remediation where earthquake loading is required; such systems are very common as these structures are often constructed on bedrock (see **Figure 89.12**). High tolerances of construction are required for these structures, and hammer and rod systems can be flexible in their nature (similar to auger drilling). In these situations, the hammer is often connected to a strong casing member over the initial 10 m to give a stiff drill string. Using this method can give a high degree of drilling accuracy and tolerance. These techniques are reviewed in Cavill (1997) and Shields (1997). Centralising rods can also be used, but again, trial drilling and measurement should be carried out to confirm that the tolerances can be achieved as required by the designer.

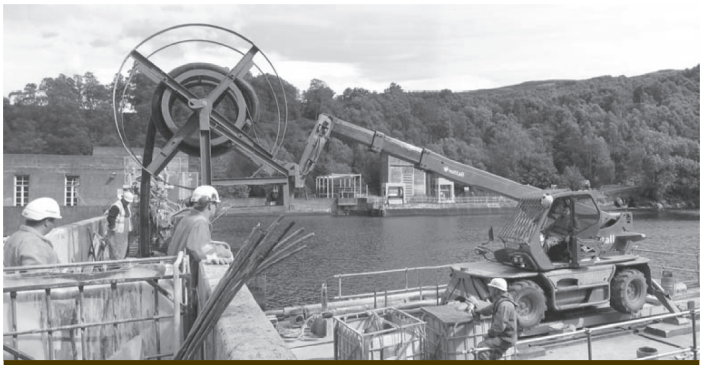


Figure 89.12 Installation of permanent multi-strand anchors at Clunie Dam, Perthshire
Courtesy of BAM Ritchies

89.6 Ground anchor testing and maintenance

Ground anchors are probably the most tested component in geotechnical works. Many ground anchors that are installed are active (carry pre-stress load). Some ground anchors are considered passive – where a nominal load is introduced, based upon the use of a torque wrench or jack system. These passive anchors are always bar systems and are often used to resist hydrostatic uplift forces.

Ground anchor testing is covered in some detail by BS 8081:1989 (BSI, 1989) and BS EN 1537:2000 (BSI, 2000) and the reader is directed to these references. The testing regime is covered by preliminary trial anchor tests, on-site acceptance tests and suitability tests. The anchor testing and lock-off is the most important part of the anchor commissioning process. The designer and contractor should make sure any specialist ground anchor contractor has the necessary experience and history of installing ground anchors. Indeed, if there is concern of the contractor's experience, a contract should be let on a preliminary trial anchor program to confirm the contractor's competency and design proposals.

Anchor testing is generally carried out using a mono- or multi-jack system (see **Figure 89.13**), hence proper quality control and calibration certificates for the jacks are of paramount importance. Also, the necessary access is required for the stressing crews in terms of anchor locations and temporary platforms, as the stressing jacks can be large and heavy.

One of the most important areas of ground anchor construction, which is commonly overlooked, is the long-term maintenance of the anchors. These tendons are often critical in terms of the performance of the structure, hence it is important to include a long-term maintenance program for the duration of the design life. Strong evidence exists that long-term anchor failure and/or poor performance is due to problems at the anchor head, hence load checks and visual inspection by an anchor specialist is very important. Some engineers consider the use of load cells for

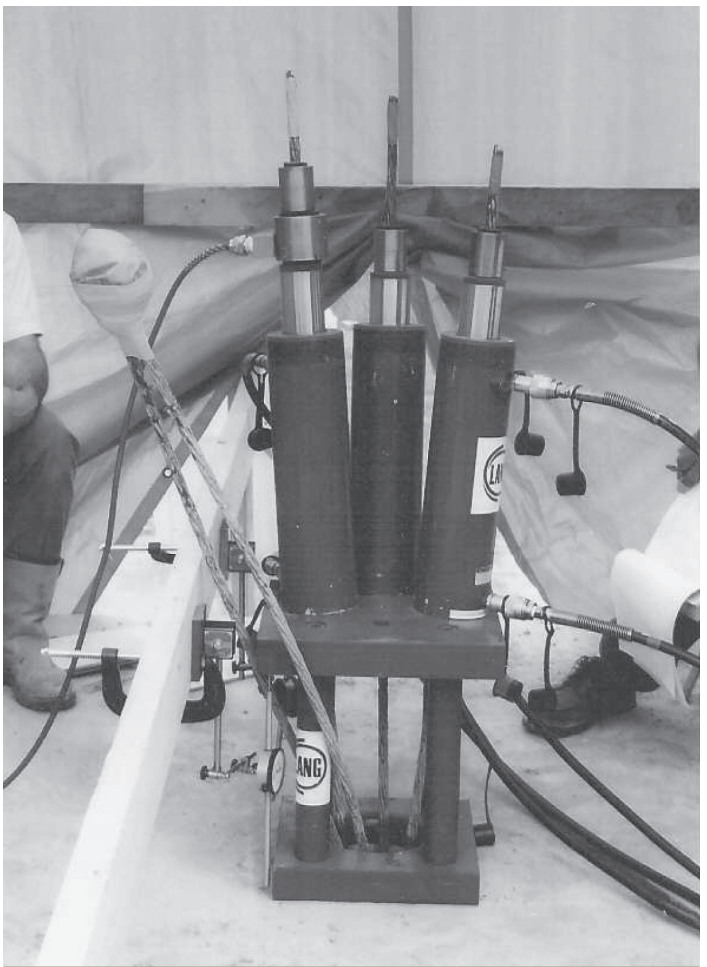


Figure 89.13 Hydraulic multi-jack system for SBMA tendons
Courtesy of A.D. Barley/Geoserve Global Ltd

long-term monitoring (BD90/64) as an option, as remote data collection becomes more popular. However, caution should be exercised as load cells can have long-term performance issues and the use of load cells enlarges the anchor head. Their use does not review the state of the anchor locking system and corrosion protection compound in the anchor head. Therefore a proper anchor maintenance program must be considered when designing and constructing ground anchors.

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It is recommended this chapter is read in conjunction with

- Chapter 66 *Geotechnical design of ground anchors*
- Chapter 72 *Slope stabilisation methods*
- Chapter 79 *Sequencing of geotechnical works*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 90

Geotechnical grouting and soil mixing

Alan L. Bell Keller Group Plc, London, UK

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Grouting and soil mixing are a range of processes that involve the injection of wet or dry materials into the ground to provide improved engineering properties. Common aims are to increase strength or stiffness or to reduce permeability within the mass of ground treated. The simplest process in concept is the permeation of the pore spaces with a fluid grout which then sets, and provides the desired properties. Soilfracture and compaction grouting methods overcome some of the limitations of permeation and employ displacement to insert volumes of grout into the ground. Compaction grouting can be used to densify cohesionless soils, and soilfracture can be used to improve cohesive soils by consolidation. Both compaction and soilfracture processes can be used to arrest or even control ground movements. Jet grouting employs erosion and mixing using high energy jets, to attack a wide-ranging set of soils and applications. Soil mixing employs powerful mixing tools to mix soils with grouts or dry binders *in situ*. A range of shapes and applications are in use. Finally verification to ensure that the intended performance is achieved is necessary, and the main methods are briefly described.

90.1 Introduction and background

This chapter covers methods using the injection or mixing of wet or dry materials to improve the performance of the ground. Mechanical improvement employing deep vibratory and dynamic compaction methods are covered separately in Chapter 84 *Ground improvement*.

Geotechnical grouting is the controlled injection of a fluid grout mix into soil or rock. The grout is typically designed to harden with time to secure improvement in the strength, stiffness or watertightness of the ground so that temporary construction or permanent specified performance can be realised. Special applications of grouting include the restoration of levels to badly settled foundations and the control of ground movements caused by underground works such as tunnelling.

Soil mixing, as the term implies, is a related set of processes whereby the ground is improved by mixing grouts in the ground, so forming a new material which hardens to deliver the desired properties.

Success in these processes depends on four crucial and related issues:

- the type, state, properties, profile and variability of the soil or rock to be treated;
- the formulation of the grout and its preparation;
- the method for placing or injecting the grout in the ground;
- the intended properties or performance required of the treated ground and how these will be verified.

These will determine the selection of which process to use; the level of improvement possible; and the costs.

Early attempts at direct injection of simple cement grouts into the ground were often hampered by failure to permeate the ground, leading to an appreciation of the limits of this grout type for permeation, and the development of silicates and more complex chemical formulations to extend the range of soils treatable, but with significant limitations.

Subsequent grouting development concentrated more on the means of delivery of simple cementitious grouts. This has opened up the range of soils and processes available, and has considerably broadened the range of applications. By the early 1990s jet grouting, compaction grouting, soilfracture grouting and soil mixing methods were all being widely and successfully applied in addition to permeation grouting (e.g. see Bell, 1994; Rawlings *et al*., 2000). Since then there has been increased use of soil mixing, and continued development of the other grouting methods particularly in relation to electronic monitoring and control of the processes on site.

Today, many of the processes require only fairly simple grouts consisting of suspensions of water and cement, or microfine cement, with the water : cement ratio being a key variable. Additives can be used to modify either the temporary fluid grout behaviour such as longer gel times, or hardened properties such as unconfined compressive strength. Reactive additives such as PFA, Bentonite, Lime, GGBS (blast-furnace slag) or Gypsum may be included in the grout mix. Indeed some of these materials are sometimes used without cement. Inert materials such as sand, rock flour or talc can be added as fillers. Chemical solutions and emulsions may also be used as grouts. Silicates are used for permeating finer-grained sands, and epoxies are sometimes used in sealing, e.g. fine rock fissures.

Figure 90.1 shows the main methods and their usual ranges of application in soils as a function of grain size. However, this figure does not allow for the effects of layered soils, highly

variable or mixed-grain soils, and the soil strength, density or stiffness, and is for general guidance only.

A summary of the key physical effects and property changes of the various grouting and soil mixing processes is included in **Table 90.1**.

The methods are extensively used to solve a range of geotechnical and foundation problems. All have been used for improving foundation-bearing capacity and for reducing total or differential settlements. Permeation, jet grouting and soil mixing have been applied in forming barriers to minimise the flow of water or waterborne pollution. Soilfracture and compaction grouting have been used to minimise foundation movement or surface displacements caused by adjacent excavations or tunnelling, and for re-levelling previously tilted or distorted foundations. However, matching the problem to be solved with the appropriate technique requires careful consideration.

It can be seen from the above that grouting and soil mixing are a range of processes, which depend on the ground profile, the type of grout selected, the method employed and the end objective. They do not produce homogeneous volumes of improved ground, but depend instead on the mass properties produced, with necessary variations in properties within the mass. This reality must be accepted in design and verification approaches to ensure technical and economic success.

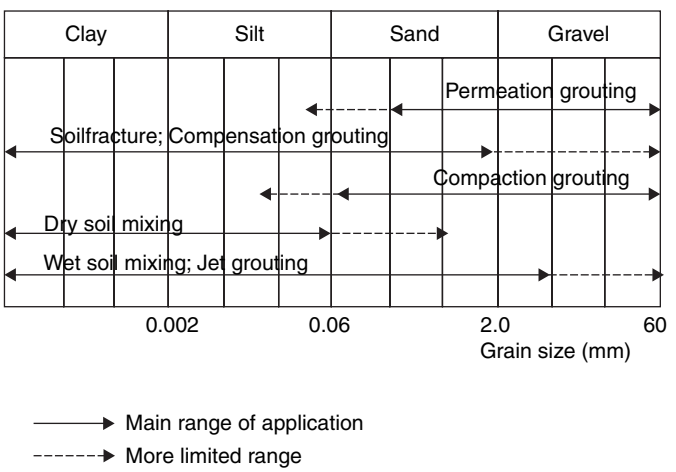


Figure 90.1 Grouting processes applicability according to grain size

It is important to ensure, early in project planning, that the ground investigation is designed to include relevant ground properties for the prospective approaches. Pre-contract or preliminary trials may be necessary to optimise the approach, or in some cases to confirm the viability of the process. Use of an observational approach, requiring changes to the quantities or injection parameters as the project proceeds, has also been found to be helpful for difficult sites. However, a detailed consideration of the limitations and considerations in grouting and soil mixing is beyond the scope of the present chapter, and those requiring further information should refer to the references and reading lists, notably CIRIA reports C572 (Charles and Watts, 2002), C573 (Mitchell and Jardine, 2002) and C514 (Rawlings *et al*., 2000). Specialist advice needs to be sought in all cases.

90.2 Permeation grouting in soils

Permeation grouting is achieved by grouting to partially fill the pores in cohesionless soils, largely displacing the water or air that is present naturally in these spaces, and without altering the structure of the ground. Groutability is a key issue, ensuring that permeation is possible in the target soil or rock mass. In soil it is influenced by the grout type and grain size distribution, and the soil permeability, grain size and grading, fines content and variability.

Soil pore size is theoretically more relevant than grain size, but establishing the pore size, shape and distribution, even in cohesionless soils is difficult. An additional issue is that small grout particles become wedged together to form a filter cake thus effectively preventing further penetration. Groutability based on soil and grout particle gradings can be developed, and a rough guide for acceptability is given by Mitchell and Katti (1981) as:

$$D_{15(soil)}/D_{85(grout)} > 24$$
$$\text{or} \quad D_{10(soil)}/D_{95(grout)} > 1$$

where the D values relate to the percentages passing a given size. This kind of approach is useful in a general sense, but care is needed as grout particles flocculate after hydration so changing the apparent particle sizes. Decisions about groutability are usually taken on the basis of experience coupled with detailed information on the variability and true grading range of the soils being considered. It should be appreciated that many

Method	Major physical effect	Key properties for application
Permeation grouting	Infills voids and fissures with grout	Reduces permeability; increases strength or stiffness
Soilfracture grouting	Opens network of fissures using grout	Increases strength or stiffness; controls ground movement
Compaction grouting	Forces a body of grout into the ground	Increases density; controls ground movement
Jet grouting	Erodes the ground, replaces with grout	Increases strength or stiffness; reduces permeability
Wet soil mixing	Mixes grout with the ground *in situ*	Increases strength or stiffness; reduces permeability
Dry soil mixing	Mixes dry binders with the ground *in situ*	Increases strength or stiffness

Table 90.1 Summary of physical effect and properties for application

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ground investigation sampling methods may not recover all of the fines in cohesionless soils, and this may distort the view taken on groutability or variability of the ground. Advice on the number and quality of samples for grading, and the interpretation of gradings, and similar care with rock discontinuity assessment, must be sought as part of the early planning process. Greenwood (1994) provides a more detailed consideration of the issue.

Generally, experience indicates that low-pressure permeation grouting using simple cement grouts is usually limited at best to gravels containing perhaps some coarse sands, or to fissures > *c*. 0.2 mm in rock. With microfine or ultrafine cements (more finely ground cements) and/or silicates, described below, it is possible to extend permeation into the coarse sands and even fine to medium sands with silicates.

Key properties of simple cement grouts as a function of water/cement ratio are shown in **Figure 90.2**.

Sandy cohesionless soils may be treated using sodium silicate – the process developed in 1926 by Dr Hugo Joosten using a two-stage approach to combine waterglass and calcium chloride to form precipitate silicates *in situ*. By the late 1950s a single-shot approach was developed by mixing an organic hardener and waterglass before injection, which then gels after predetermined periods in the ground. This principle remains applicable in soil permeation, and various proprietary reagents are available as hardeners, with widely differing properties and durability.

Other single fluid low-viscosity chemical grouts capable of penetrating finer-grained soils such as silts have also been developed, but these are expensive, and the strength, stiffness and durability often more limited than with simple cements. Several are also highly toxic leading to restrictions or bans on their use, and are not considered further here as they now see little use and are for very specialist applications only.

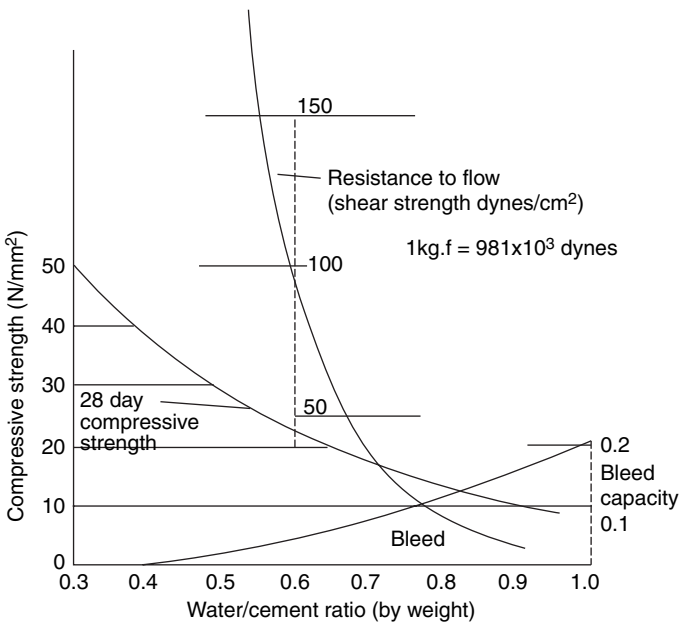


Figure 90.2 Properties of simple cement grouts
Reproduced from Littlejohn (1982)

The mass properties of permeated ground are quite different from the neat grout properties, and **Table 90.2** provides an indication of the strength and permeability limits expected. The expected properties of treated ground depend on the grout type, the grading and state of the ground and the details of the permeation grouting method employed. Considerable variation, particularly in strength and stiffness, is normal.

Equipment employed in permeation grouting is wide-ranging and to some extent dependent on the grout selected and the method of injection. Drilling boreholes is the normal precedent to grouting, necessary in order to get access to the ground to be grouted. A description of drilling processes is beyond the scope of this chapter (see Warner, 2004 and the suggested reading list). Only relatively small diameter holes are needed, less than 100 mm diameter, and while vertical holes with long masts in open sites are cheapest, inclined holes, including drilling from shafts, galleries and tight corners are perfectly feasible if the application demands it.

For low-cost treatment in highly porous ground, grouting can be performed directly from the end of borehole casings or lances or through slotted pipes installed in the boreholes. High pressures are not effective due to grout surfacing, and control of permeation is relatively crude. However, such methods can be relevant for improving marginal situations in open ground, and particularly for temporary works.

Soil grouting was considerably improved with the invention of the Tube à manchette pipe by Ischy in 1933, or TaM pipe (**Figure 90.3**). These pipes, typically 25 mm to 50 mm o.d. are installed in lengths which screw together in a borehole to attain the depths required, and are grouted in place using a relatively weak sleeve grout. Grout ports closed with cylindrical rubber seals are spaced at regular intervals, ranging from 250 to 500 mm. A packer placed inside the pipe isolates one set of ports at a time, and grout can be injected into the surrounding ground, after it expands the surrounding seal and breaks the sleeve grout.

The TaM pipe method provides much more control for permeation and related forms of soil grouting because the injection position can be selected, repeated injections from any port location can be made, and grout pressure or volume can be measured or controlled. The TaM method is now almost invariably used in all but the least demanding soil permeation applications. Borehole spacings for TaM installation depend on the grout and permeability of the soil, and the application, but range typically from 0.6 m to 2 m.

	Cements	Microfine	Silicates
Unconfined compression strength UCS (N/mm²):	< 15	< 10	< 7
Coefficient of permeability k (m/s)	> 10^{-6}	> 10^{-7}	> 10^{-7}

Table 90.2 Properties of cohesionless soils treated by permeation grouting

Permeation grouting is a flexible process because discrete layers or zones of soil can usually be treated, even when beneath buildings and structures as only small diameter drilling is needed to reach them, and the shape of the treated volume can be made flexible to suit. Minimal spoil is generated, and ground movements likewise, if the grouting is properly selected, specified and executed (Greenwood, 1994).

Major applications are as follows:

- underpinning foundations;
- sealing cracks, joints and other rock discontinuities;
- temporary support for excavations, shafts or tunnel construction;
- temporary cutoffs;
- reducing permeability below dams;
- improving stability of retained ground;
- support for services, such as pipelines on poor ground.

Perhaps the major use of permeation grouting worldwide is to reduce soil or rock permeability. Many dams have cut-offs formed in this way (**Figure 90.4**). Another very broad range of applications concerns the provision of temporary stability to facilitate the construction of tunnels, shafts and excavations. Underpinning has also been a key use of permeation, where ground conditions allow (e.g. **Figure 90.5**).

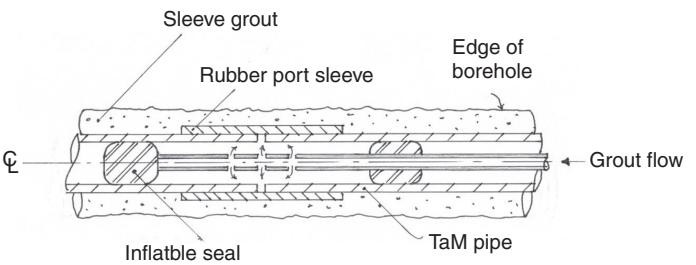


Figure 90.3 Principle of operation of Tube à manchette pipe

90.2.1 Permeation grouting in rock

Permeation grouting in rock is usually applied to infill the discontinuities in otherwise sound rock. It is mainly used for sealing, or less frequently for strength or stiffness, in fissured or jointed rock: for example cut-off curtains or blankets beneath dams; sealing rock near nuclear installations; to enable tunnel construction or for permanent improvement around or near tunnels. Site investigation needs to identify the volume and size of the rock fissures and voids so cores and *in situ* tests will be needed to properly explore the rock mass.

Grout hole diameters are small, typically 35–75 mm, and spacings typically 1–2 m with opportunity for infill holes if needed. For curtain grouting the holes may be spaced in two or more rows, depending on the specification and rock openness, to improve the effective thickness of the curtain. The rock is often grouted within the bores in stage lengths of 3–5 m. Each stage is isolated by down-the-hole packers which seal the upper and lower boundaries of the stage. Depending on the rock mass state, grouting may proceed from the top (descending stage) or from the base of the borehole (ascending stage).

Seepage through grouted rock can be determined using the Lugeon unit (1 Lugeon is the volume of water (litres) lost per metre length of hole per minute at 10 bars pressure). However, this does not necessarily relate well to groutability, so the GIN (grouting intensity number) method has been developed, which

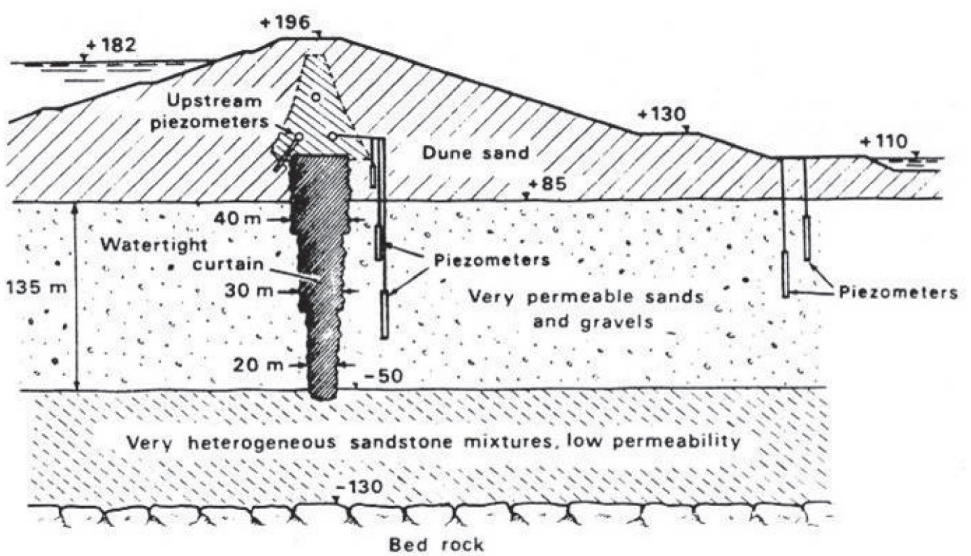


Figure 90.4 Grout curtain for high Aswan Dam
Reproduced from Littlejohn (1983)

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avoids the need for Lugeon testing, and is increasingly being used in the USA, Europe and the UK. The method was introduced by Lombardi and Deere (1993), and is described in Warner (2004).

Cavities in rock are also frequently backfilled with grouts, particularly in areas of old underground mineworkings in the UK. Low-cost grouts are used to fill old shafts and working tunnels to prevent migration of voids to the surface and thus prevent or minimise any consequent surface subsidence. Methods are described in CIRIA SP32 (Healy and Head, 2002) and on the websites of the specialist contractors.

90.3 Soilfracture and compensation grouting

Fracture grouting, or soilfrac grouting, employs a similar equipment and approach as permeation grouting in soils but uses higher pressures to deliberately cause 'hydrofracture' which opens up a network of fissures in the ground, thus enabling the injection of grout. This widens the range of soils applicable, and enables treatment to extend even into the clay range. Significant control issues in regard to pressures and volumes injected, and the nature of the grout itself, are needed. Compensation grouting is a particular application of fracture grouting which enables the control of ground movements to compensate for deformations arising from construction.

Fracture grouting was often correctly seen as an undesirable effect of overpressuring in soil permeation grouting, but was first developed in France as a positive measure and is thus sometimes referred to as 'claquage' grouting. Controlled hydrofracture is achieved by injecting high-viscosity grout under high pressures so inducing cracks in the ground which are opened up by the flowing grout in them. By repeated injections of relatively small amounts of grout a network of veins and lamellae form a dendritic structure in the ground (**Figure 90.6**). Steel TaM pipes are often used to ensure the repeatability of the injections, especially for compensation grouting applications which can go on for many months and sometimes years. Grouts employed are commonly cements but often contain additives to assist in achieving early strength or other required characteristics.

It is important with hydrofracture grouting that experienced specialists are employed who understand the process well, as uncontrolled hydrofracture at best will be ineffective and at worst can create deleterious effects for the ground and may cause grout to travel into such structures as drains, sewerage systems or basements, creating blockages or damage.

The technique can be applied in mixed and cohesive soils for straightforward ground improvement for foundations, although applications are limited due to time and cost. Generally the process causes consolidation or tightening between the grout veins, and stiffness increases of between two and five are typically possible.

However, perhaps of more interest is that, particularly in cohesive or more dense granular ground, the volume of the soil mass has to increase to accommodate a significant proportion of the extra volume of the injected grout. Consequently a powerful application is to employ the technique to arrest excessive total or differential settlement beneath existing buildings and structures.



Figure 90.5 (a) Underpinning using permeation grouting; (b) Thames Gravel section treated using silicate grout permeation
Courtesy of Keller Ground Engineering

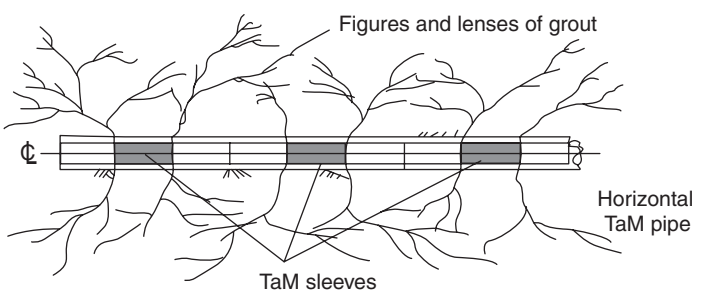


Figure 90.6 Formation of dendrictic structure of grout inclusions using soilfracture grouting

In such cases it is advantageous to have the pipes drilled in at low angles to the horizontal or even from specially constructed shafts in order to get the array of pipes installed. Since the movements induced in the ground are locally very small and are attenuated over a larger area when transmitted underneath a foundation, the technique is kind to the structure when properly managed. In any case the existing state of the structure and its foundations, and any adjacent structures or buried services, must be well understood prior to design and grouting operations.

The volume increase caused by soilfracture, if injection persists, can migrate to the surface as heave. This effect, when controlled by the layout of the TaM pipes, the grout type, and the sequence and intensity of injection, can be used not only to arrest settlement, but to take the process further in order to relevel previously settled buildings and foundations – a major application of the technique (e.g. **Figure 90.7**).

This application was first employed in Germany, and not only led to high accuracy in many cases, but also to the development and application of soilfracture grouting to compensate in real time for ground loss (Raabe and Esters, 1990). This takes concepts of re-levelling even further and attempts to balance the ground loss arising from tunnelling with the volume increases in the ground caused by the soilfracturing grout injection. Basic principles of compensation grouting are outlined in **Figure 90.8**.

The technique has been applied in the UK where it is known as compensation grouting, and the first major applications were in connection with protection of many key structures during construction of the running tunnels for several of the Jubilee Line Extension contracts in London (Linney and Essler, 1994), and is now a common technique across Europe and further afield (Moseley and Kirsch, 2004; Mair, 2008).

90.4 Compaction grouting

Developed in the USA in the late 1950s, compaction grouting is now used extensively there, and has been adopted in the UK (e.g. see Crockford and Bell, 1996) and increasingly in Europe and Asia. It offers a very different approach from permeation or soilfracture. The method employs low slump mortar-like grout (slump typically 25–75 mm but can be more), which is forced into the ground under high pressures to form bulb-shaped masses. These can be employed to compact loose granular soils, or can be used to displace cohesive and mixed soils in order to create heave, close voids or control ground movements during construction. (It should be noted that in the USA recent terminology distinguishes between 'compaction grouting', used only in applications to compact soils, and other applications of the technique, referred to as 'low mobility grouting'. In Europe all applications come under the 'compaction grouting' umbrella.)

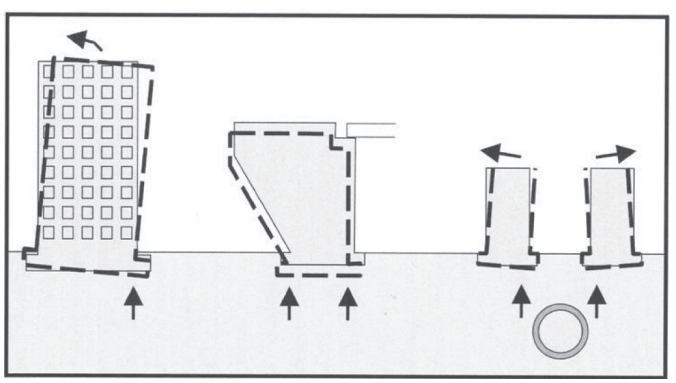


Figure 90.7 Applications for soilfracture grouting
Modified from Raabe and Esters (1990)

A key requirement of the grout mix is that it does not permeate or mix with the soil, or cause hydrofracture as a fluid, but remains a plastic growing mass during injection, with enough internal friction to allow flow but not so much that segregation or filter blocking occurs. Mixes consist of a wide range of aggregate sizes, often a silty sand, with silt sizes accounting for 10–25% of the total by weight, with cement or cement substitutes such as fly ash. Further details are included in Rubright and Bandimere (2004) and Warner (2004). The grout is often batch-mixed on site in simple pugmills, or is supplied from readymix plants. More expensive continuous batching set-ups can be effective for large or high-volume demand sites.

Generally the grout bulbs are formed from the end of casings installed in boreholes or pipes of about 50 mm to 100 mm max dia. driven directly. Gaining access under structures can be facilitated by the use of inclined borings. A typical set-up for ground improvement using compaction grouting is shown in **Figure 90.9**. Typical pressures needed are often in the 40–60 bar range, and concrete pumps with capabilities of 80 bar or more are typically specified. Pressures at which further grout intrusion refuses are not usually obtained before surface heave is reached, which is helpful for applications in which surface movements are required.

Lateral spacings between the pipes, forming the bulbs upstage or downstage, the vertical spacing between bulbs, and the order in which adjacent casings are injected are interrelated and depend on the objective of the process, the ground conditions and the site environment. Spacings between pipes vary for compaction purposes from about 1 m where overburden pressures are low, to about 4.5 m or so where deeper layers at 10 m or more are being addressed.

To enable efficient compaction, pipes are often installed in primary and secondary grid arrangements. Upstage grouting is the most common and least costly approach, with each bulb injected at intervals as the pipe is withdrawn from the maximum required depth. Downstage grouting can be useful where compaction is needed only at shallow depths to provide some near-surface confinement for subsequent grouting beneath. Compaction grouting applications usually proceed by injecting target volumes, to ensure relatively even distributions of grout in the ground. Experience indicates grout takes

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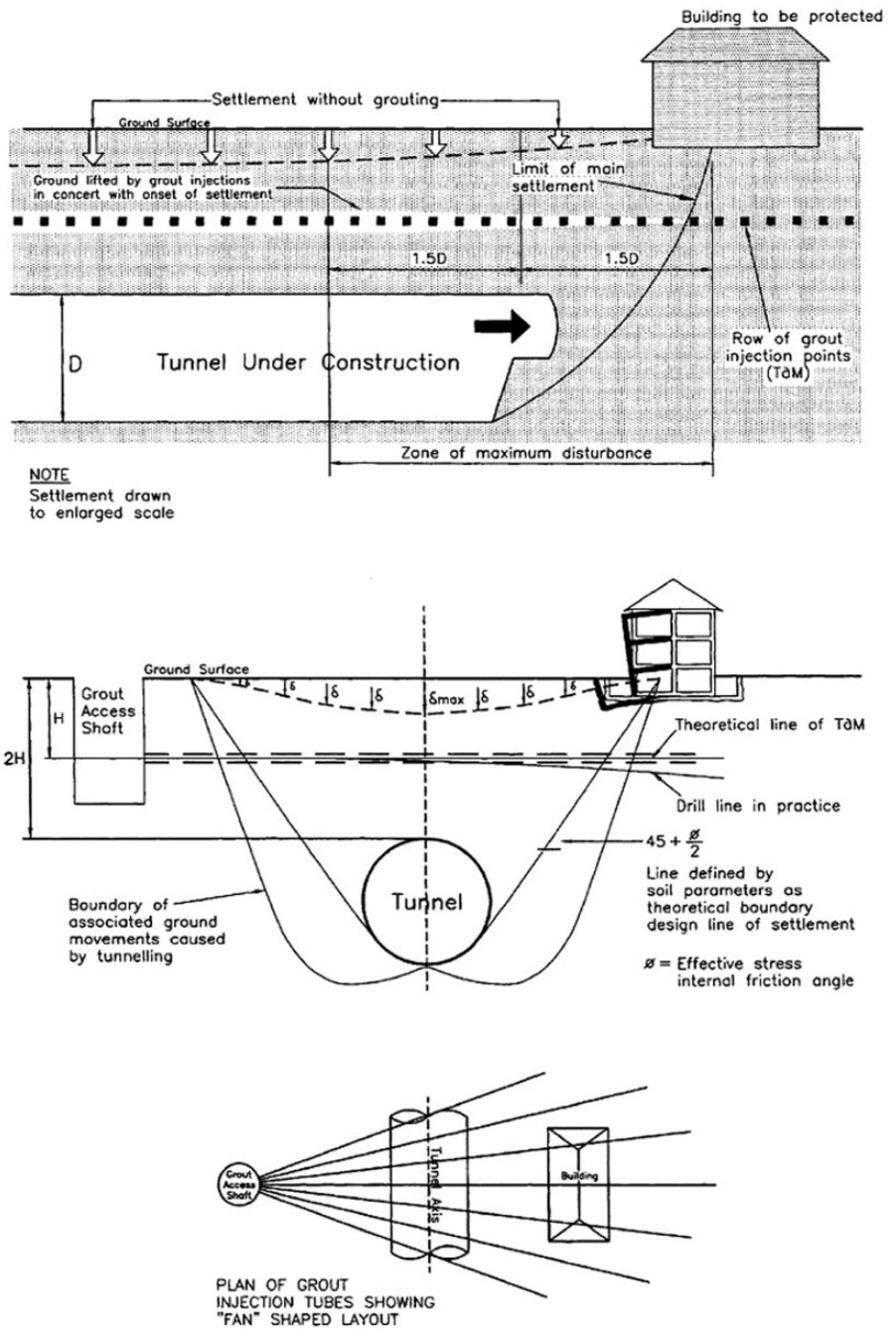


Figure 90.8 Principle of compensation grouting for tunnels
Reproduced with permission from CIRIA C514 Rawlings *et al.* (2000), www.ciria.org

of 3–12% of ground volume, more frequently at the upper end, are needed to achieve intended levels of densification or movement in non-voided soils.

Compaction grouting can be used as a ground improvement technique to densify loose natural or voided filled ground (e.g. **Figure 90.9**) to be employed as bearing material for new or adjacent construction; or to deal with features such as loose material in chalk cavities. The latter is not uncommon in southern England and compaction grouting has been used for several foundations for structures and the preparation of waste disposal sites, as well as remedial works beneath existing buildings. It has also been applied in the densification of potentially liquefiable zones for earthquake protection, and has been applied retrospectively to provide seismic protection

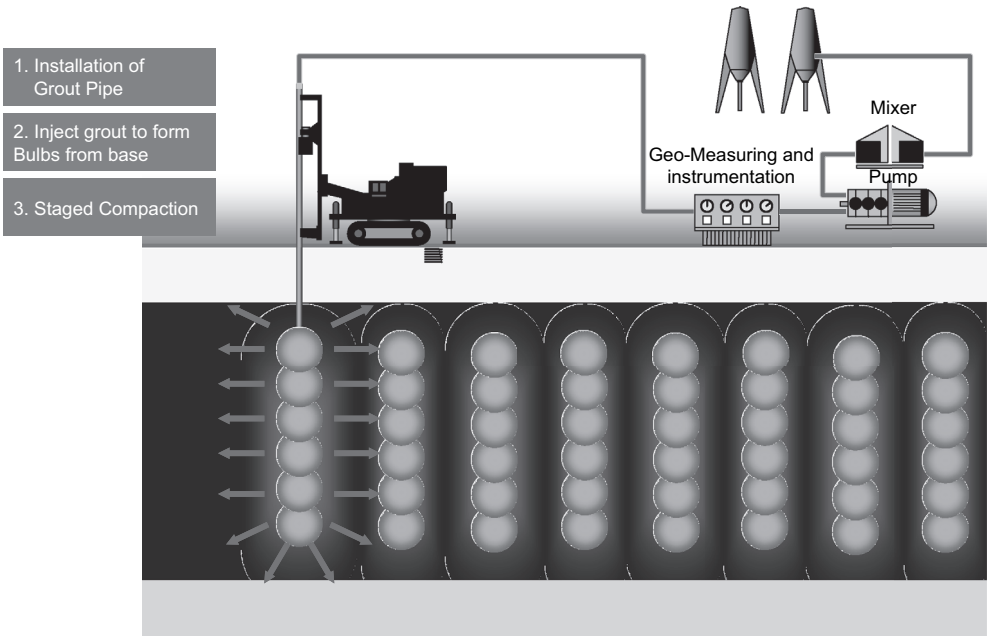


Figure 90.9 Equipment and procedure for ground improvement using compaction grouting
Courtesy of Keller Grundbau

for pre-existing structures. Warner (2004) and Moseley and Kirsch (2004) provide many examples.

However, because it can create heave, like soilfracture grouting, a key application area is in re-levelling structures (**Figure 90.10**) and to compensate for ground movements caused by tunnel or other excavations. Indeed the first compensation grouting in the USA employed compaction grouting and predates the use of soilfracture grouting in Europe for the same purpose (Baker *et al*., 1983)

90.5 Jet grouting

Jet grouting uses erosion to enable grouting to proceed across a very wide range of soil types. High-pressure jets of water or grout break down the soil structure, and remove varying proportions of the resultant soil particles. The grout replaces the eroded material and becomes mixed with the residue, which can be a very small proportion depending on the method employed and its ability in discharging spoil to the surface.

There are now several forms of jet grouting in use, and the majority stem from three basic variants: single, double or triple systems (**Figure 90.11**). Examples of the drilling equipment and high-pressure pumps required are shown in **Figure 90.12**. Compact crawler rotary drills with rotary heads are most often used to reach the zone identified for treatment by jet grouting. Where possible, long masts should be used to enable the jet grouting for each element to be completed in one lift, so avoiding costly re-entry and joints in the treatment. The erosive pumps (grout for the single and double, and water for triple) need to operate consistently at high pressures of up to 600 bar or more. Grout is usually mixed and batched on site and held in tanks before injection.

In all cases the work commences by drilling a small borehole of *c*. 100 mm diameter to take the hollow jetting rods containing the relevant jetting fluids to the maximum treatment depth. A column is formed by simultaneously withdrawing and rotating the rods at constant predetermined rates, while the erosive jets cut the ground and the grout is placed with predetermined volumetric injection rates. In some cases panels of grout are formed as this geometry can be useful in sealing applications. The jetting parameters, i.e. lift and rotation speeds, fluid flows and pressures, are always monitored very closely and in many cases are controlled automatically to ensure highest quality. Columns are the most commonly required shape, and are usually intersected to form a treated mass of ground with properties determined in part by the nature of the ground being eroded, as well as the injected grout. Some examples of column and panel arrangements for various applications are shown in **Figures 90.13**, **90.14** and **90.15**. The grout mix is typically a simple water/cement grout, sometimes with additives such as bentonite. It must be free from impurities and sufficiently fluid to flow easily through the jetting nozzles.

The single and double systems use a grout jet to erode and place the grout, shrouded in the case of the double system to improve efficiency. The triple system employs an efficient air-shrouded water jet for erosion and the grout is separately placed below the erosive jet. Precutting using water or air–water jets may be useful in stiffer cohesive layers or with very mixed soils, to improve the efficiency and uniformity of the final column.

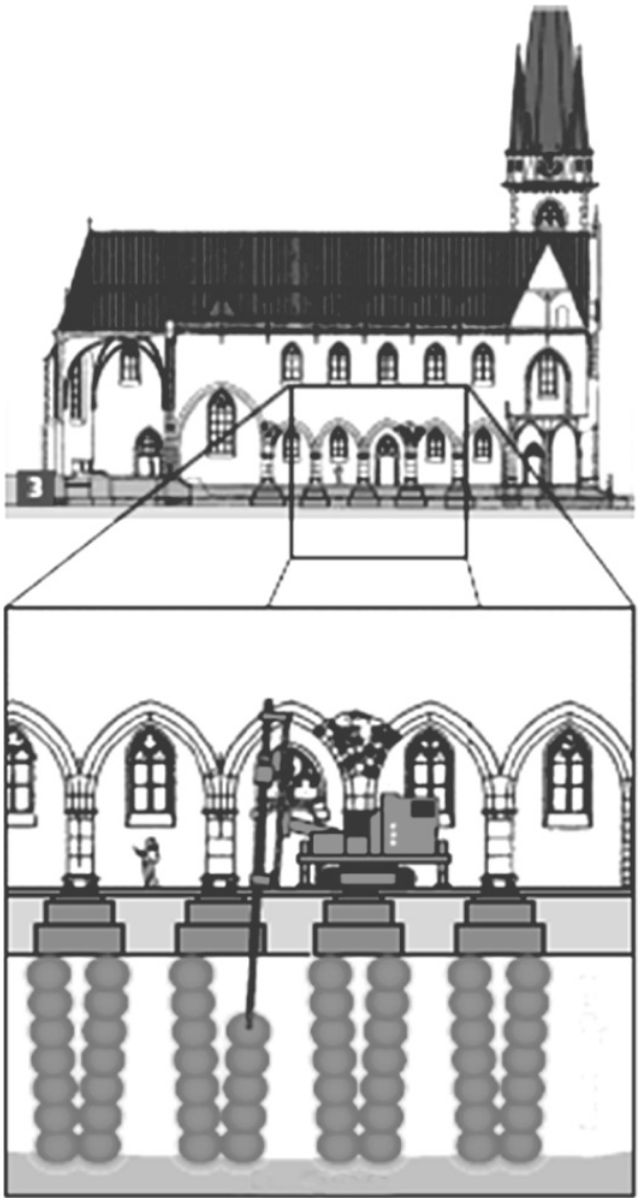


Figure 90.10 Re-levelling and protection of old foundations
Courtesy of Keller Grundbau

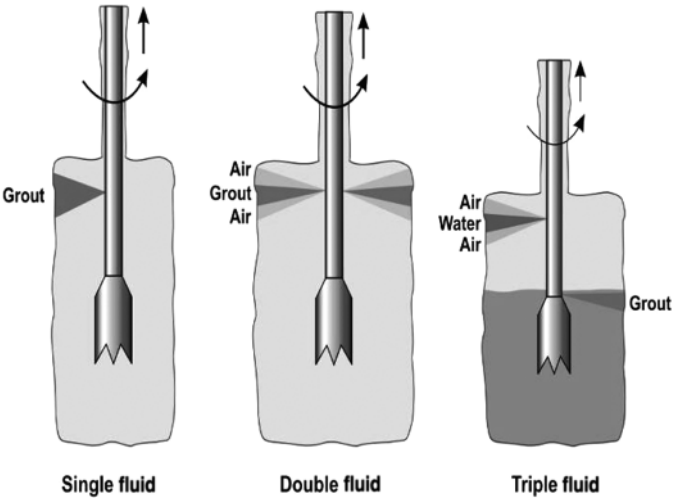


Figure 90.11 The main variants of jet grouting processes

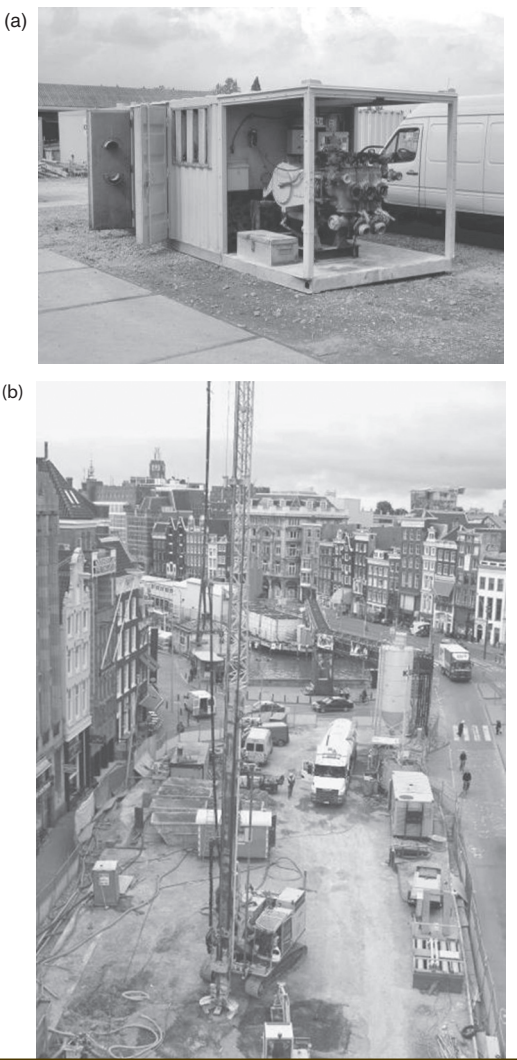


Figure 90.12. (a) High pressure pump for water or grout; (b) Jet grouting drill rig
(b) Courtesy of Keller Grundbau

It is important to ensure the spoil has an unhindered passage to the surface to prevent excess ground pressure, which can result in unwanted heave at the surface. Similarly, planned and efficient spoil handling and disposal, possibly by recycling, will be necessary. In many applications, notably underpinning, a thorough survey of the existing structural integrity of any structures likely to be influenced will be needed. Monitoring of structural movements and ground heave will be necessary, and proficient real time systems of communication established between the monitoring and the jetting process and personnel.

Since jet grouting can erode large volumes of soil, attention also needs to be paid to temporary stability, the gain of grout

strength with time, and the sequence of forming interlocking columns. Diameters of up to 5 m or more are possible with higher-energy air-shrouded grout jet systems using double opposed nozzles (i.e. the 'superjet' process) and these can be very efficient in such applications as forming basal cut-off slabs in basements and excavations. However, large diameters are not always appropriate or economic depending on the application, for example beneath adjacent sensitive structures which may have limited ability to span large unsupported volumes of ground in the temporary condition. Column diameters in most UK applications tend to be much less than 3 m. Erodability of the soils being treated is also highly relevant to the diameters that can realistically be achieved, and variation in soil type or state in the treatment zone must be understood in advance to ensure success.

Table 90.3 provides a rough guide to the range of mass properties expected, but actual properties will depend on the ground properties and variability, the process and jetting parameters used, and the properties of the grout injected.

Jet grouting has the ability to treat discrete layers in the ground, along with other forms of grouting, but is able to treat a wider range of soil and rock and has additional advantages in that the geometry can be more flexible, the properties more predictable and able to meet demanding specifications, with good performances available in a wide range of soils. Indeed jet grouting enjoys wide application also due to its flexibility of geometry in the ground, and the broad range of ground conditions treatable.

A key further advantage is the intimate contact which jet grouting can make with overlying structural foundations, or with sealing sheet piles, particularly with the double and triple systems as the erosive jet cleans existing structural surfaces prior to the grout being placed (see **Figure 90.13**). This makes jet grouting suitable, with appropriate consideration, for sealing and underpinning applications. It can also be used in weak rock, in cleaning out softer zones in weathered rock, in debris-filled fault zones, fractures, fissures or joints.

Underpinning of existing buildings needed for new adjacent construction is one of the most common applications of jet grouting. This is frequently done in conjunction with basement construction, as the jet grouted mass not only provides support of the foundations of the existing building, but also support for the excavation and formation of the new basement, which can be taken close to the existing building line thus saving space over other methods. It has been used frequently in many cities around the world including London. Typical arrangements for jet grouting underpinning are shown in **Figure 90.14**.

As well as underpinning, jet grouting has been widely used in both permanent and temporary cases for providing basal seals and plugs and in forming new cut-offs or extending or remediating existing cut-offs beneath dams and other water-retaining structures. Due to its capacity to be bonded to existing structures *in situ* it has been used to seal leaking coffer-dams and shafts. It is

UCS (Mpa)	3–28	(sands and gravels)
	2–6	(silts and silty sands)
	1–5	(inorganic clays)
	< 2	(organic clays, silts and peats)
Permeability k	> 10^{-8} m/s	granular and cohesive

Table 90.3 Typical properties for jet grouted soil masses

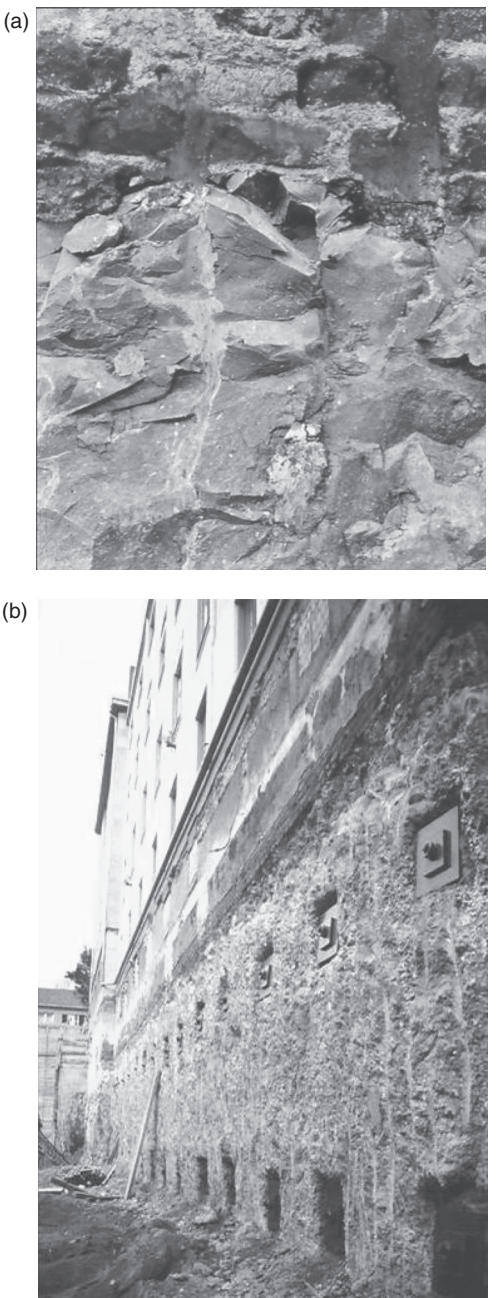


Figure 90.13 Showing (a) intimate contact between underside of footing and jet grout; (b) exposed and trimmed jet grout columns with anchor plates

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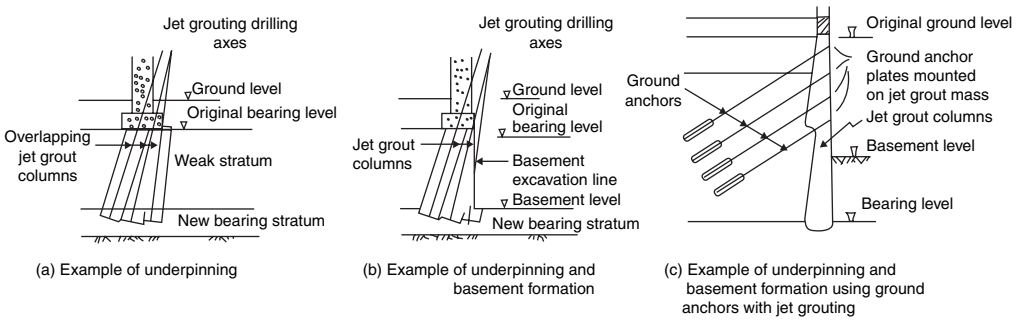


Figure 90.14 Examples of underpinning and basement formation using jet grouting

also used directly and indirectly to provide support, for example improving the capacity of existing foundations and abutments. It is also extensively employed in creating temporary stability for tunnel construction or for break-in/break-out blocks. Some examples of such applications are included in **Figure 90.15**.

Figure 90.16 shows a sheet-piled excavation supported by a jet grout base plug. After driving piles to support the eventual structure, the base plug is formed using interlocking jet grout columns, drilling from the surface and jetting only over the depth range needed for the plug. Sheet piles to support the sides of the excavation are then driven, and a structural high-level propping system installed during initial excavation. Further excavation proceeds to final excavation level with no need to install further support as the jet grout seals against water ingress, stabilises against heave and acts as a basal prop. This approach to constructing excavations, with some variations in sequencing of the elements illustrating the flexibility of jet grout construction, is widely applied and has been employed on several projects in the UK.

90.6 Soil mixing

Deep soil mixing has developed in two major ways. Dry soil mixing uses dry powders, often cement or lime-cement, which is usually blown with compressed air into the mixing train, and the soil moisture is sufficient for the properties of the mix. Dry soil mixing was developed in Sweden in the 1960s to address the compressible soft soils found there. The method, initially employing lime binders, is now widespread in Finland and Norway as well as Sweden, and more recently in other parts of Europe including the UK and the USA. Cement and cement-lime binders are now more common.

In contrast, wet soil mixing uses wet grout, usually a cement suspension, prepared and mixed at the surface, before being pumped into the ground through the mixing tool for subsequent mixing in with the soil. Water in the grout mix and water in the ground contribute to the final mix. Wet soil mixing was introduced in the USA in the 1950s, and has been developed steadily since the 1970s notably in Japan with depths of 50 m or more having been treated, and with some very sophisticated equipment developed for near and offshore work. The method is now well accepted in the USA and the Far East as well as Japan, and is increasingly being employed across Europe, notably in Poland.

Soil mixing is being applied in a wide range of applications, often using columns to provide an improved mass of ground similar to jet grouting. Soil mixing needs to be performed from the surface in each case, lacking some of the flexibility of jet grouting, but is nevertheless usually competitive due to its generally lower cost per cubic metre. Applications of soil mixing include:

- improvement of bearing capacity;
- reduction of total or differential settlement;
- improvement of slope stability;
- support in temporary or permanent case for excavations or tunnels;
- reduction of earth pressures behind retaining structures;
- cut-off barriers (reduction of permeability);
- immobilisation or confinement of *in situ* soil contaminants;
- improvement of seismic (liquefaction) resistance in new and retrofit construction.

A more complete description of the methods may be found in Topolnicki (2004, 2009), and in the proceedings from two recent international conferences at Stockholm in 2005 and Okinawa in 2009 (see references below).

90.6.1 Dry soil mixing

Dry soil mixing is typically applied to soft soils in which the natural moisture content is close to or above the liquid limit. It is increasingly being used for more organic soils and even some peats. Typical dosage rates are 75–200 kg cement per m^3 of natural soil, or more for very soft or organic soils but depend on the soil and binder types, the intended performance of the mixed soils, and the capability of the equipment. Undrained shear strengths attainable are up to 1000 kPa or better at 28 days in inorganic silts, but are typically much lower, and especially so in the case of organic clays and peats.

In any case, it is normal to conduct pre-contract laboratory mixing tests with a range of prospective binder dosages on representative soil samples, to confirm the expected level of improvement. Often, and in particular where experience with

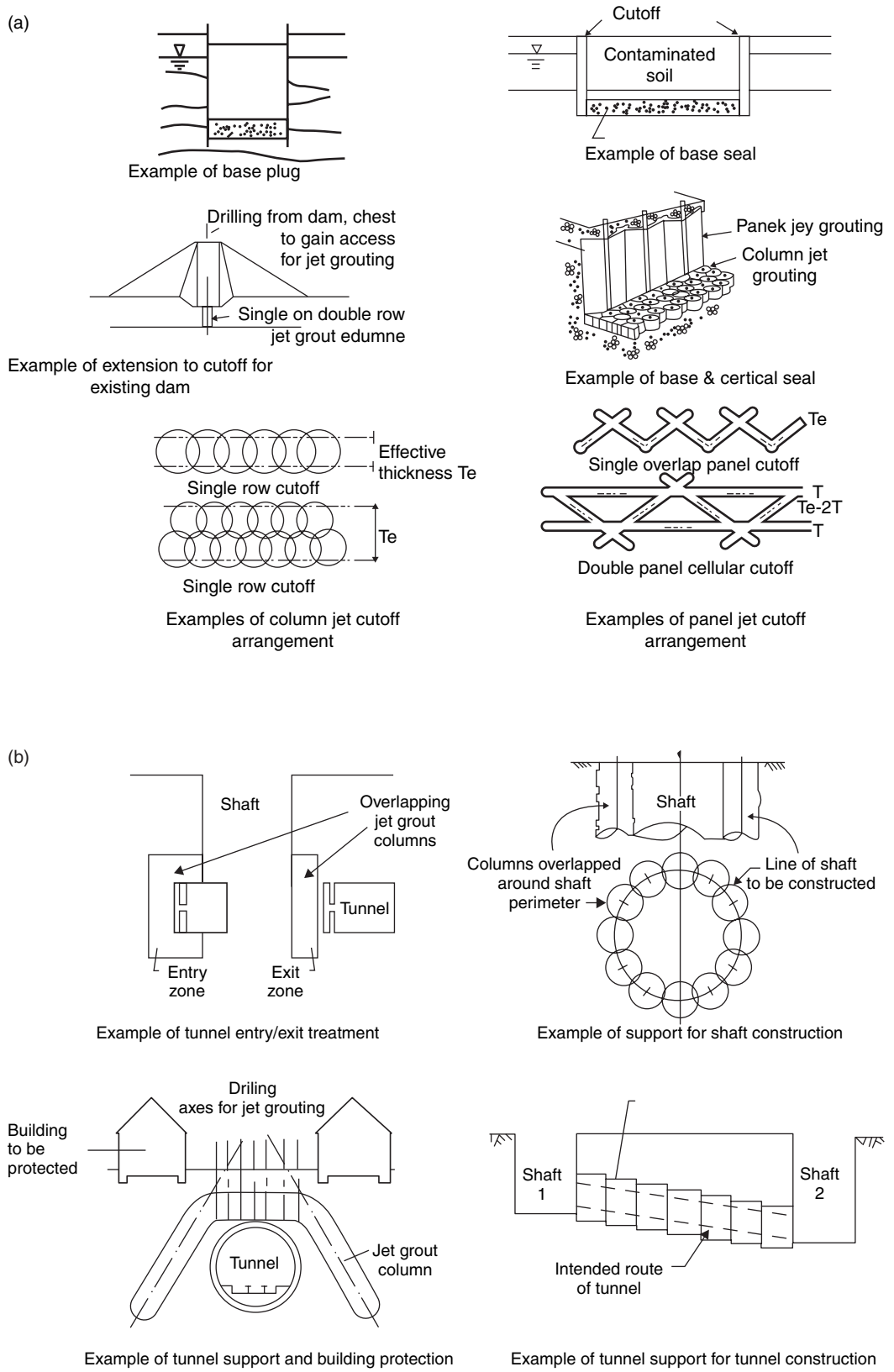


Figure 90.15 Some examples of jet grouting (a) in sealing applications; (b) for shafts and tunnels (Bell, 1993)

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(a)



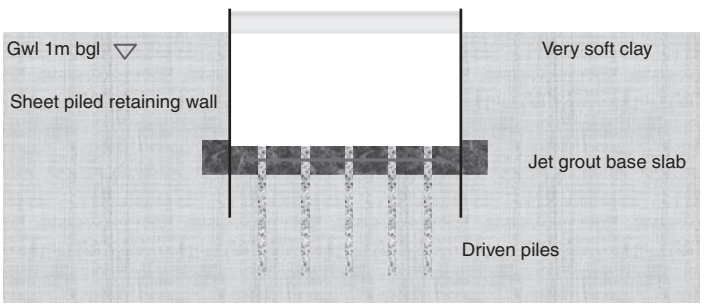


Figure 90.16 Jet grouted base slab
Courtesy of Keller Ground Engineering

the soil type or area is minimal, it will be necessary to conduct preliminary site trials as well, as laboratory mixing can never be truly representative of the ground profile or the mixing efficiencies achieved in the field.

The mass of ground to be improved is often formed using columns of mixed soil, most designed to interlock. After the initial set of the columns a small surcharge can be applied to optimise the settlement performance in embankment and foundation applications. Mixing tools to form columns range in size from 500–1000 mm diameter, and commonly 600–800 mm diameter, to suit the binder injection equipment. Most designs employ vertical columns, with construction tolerances of up to 1 in 100, but small inclinations of up to 10V:1H may also be used especially beneath embankments.

Equipment (**Figure 90.17**) consists of a relatively lightweight hydraulic base machine of up to 40 tonnes or so, with wide tracks to assist with operating on working platforms formed over soft ground. The machine operates in tandem with a self-propelled binder delivery unit, a pressurised air vessel known as a 'shuttle'. The base machine rotates a Kelly bar with a mixing tool attached to its base, into the ground to the desired depth. It is then withdrawn at a steady rate and the tool rotated at a high rate, typically 150–200 rpm, as the binder is injected using compressed air, so forming the mixed column. Columns



Figure 90.17 Dry soil mixing equipment: dry binder shuttle and mixing machine
Courtesy of Keller Ground Engineering

can be constructed to about 20 m depth or beyond, given due consideration of machine stability and enhanced air pressure. Dosage rates can be modified to some extent during withdrawal to suit significantly differing layers in the ground. However, the horizons of such layers must be well established in advance of construction using, for example, CPT equipment, as the dry soil mixing process is not intended for ground investigation. Binder wastage is under 5% in most cases.

It is important to ensure even distribution and mixing of the binder into the ground. The shape, numbers of blades and nozzle spacing on the tool are set to optimise binder injection across the diameter of the column. Distribution with depth is dependent on steady binder delivery and tool rotation and withdrawal, and it is common to automatically measure the binder delivery, and show this against depth and tool parameters, and to display this information in the cab of the mixing machine. Some machines are now using such information to not only measure, but also control binder delivery, thus improving the quality of the mixing and minimising waste.

A useful broad indication of mixing efficiency is provided by the blade rotation number (BRN), defined as the total number of mixing blades passing during mixing over a 1 m depth interval for a single shaft, whence:

$$BRN = (NR)/V$$

Where N = nr of blades
R = rpm of tool
V = withdrawal velocity (m/min)

Experience from Sweden indicates that BRN values for adequate dry mixing in normal soft soils range from 220 upwards, and in practice is often 300 or more.

At relatively shallow depths, typically 3–5 m in depth, mass mixing rather than column mixing may be used. Here, tools

rotating in a horizontal axis, or larger than normal vertical axis tools, are mounted on the booms of basic hydraulic diggers. These are used to form large volumes of improved ground relatively quickly, by repeated mixing within designated zones. Care is needed with site control to ensure appropriate binder distribution and mixing, as there is no means of accurately measuring binder dosage due to the large overlaps used. However, provided the applications for mass mixing, such as highway sub-grades, are carefully selected to match this approach it can represent a cost-effective solution.

Like jet grouting, blocks of treated ground can be formed by interlocking dry soil mixed columns together to attack a wide range of geotechnical problems. Dry soil mixing has extensively been applied to support road and rail embankments over soft ground (see **Figure 90.18**). It has also been employed beneath housing and other structures; to provide side and base support to excavations; to buttress retaining walls; and in shaft and tunnel construction.

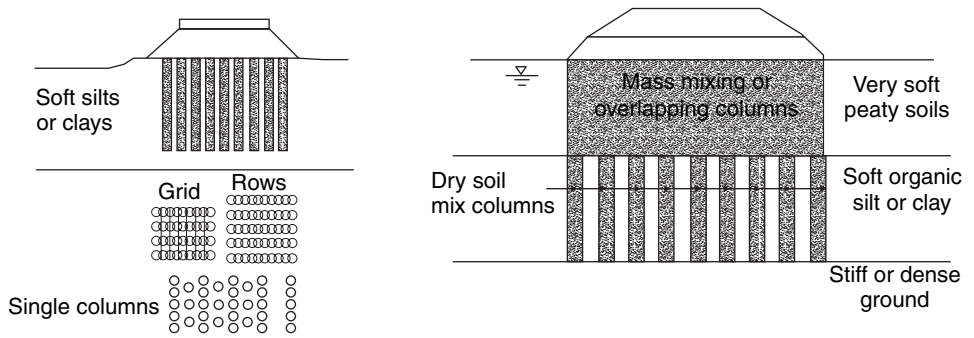
90.6.2 Wet soil mixing

Wet soil mixing differs from dry soil mixing, not only in that wet grout is injected, but in that the equipment, range of applications and soil types treatable are very much broader. Columns can be formed using single vertical axis systems, or with double, triple or more axes constructing interlocking columns in one cycle (**Figure 90.19**). The multi-axis systems tend to be more efficient when constructing linear structures in the ground such as cut-off walls or long earth-retaining structures. Single-axis soil mixing is used to construct columns typically of 600–2500 mm diameter and multi-axis systems have a wide range of geometry, with nominal diameters again of 600 mm and greater. Japanese development has gone even further and large multi-axis mixing machines are available for working off barges to depths of 60 m or more from sea level.

The key equipment for wet soil mixing consists of silos for storage of the cement and other materials needed, high shear mixers, water supply and pumps capable of delivering sufficient slurry grout to ensure that each cycle of mixing in the ground is not held up – for quality as well as production reasons. The grout is pumped through delivery lines into the hollow drilling stem and thence through ports placed in the mixing tool at its end. The stem or Kelly bar is normally mounted on the masts or leaders of cranes, and rotated using high torque heads. The base machines tend to be heavier than with dry soil mixing reflecting the wider range of soil types and larger column diameters, and multi-axis approaches that are employed, and additional consideration for good working platforms needs to be made. Mixing tools vary widely in design depending mainly on the soil type being treated, most with multi-level blades.

Construction for single axis and many multi-axis cycles commences with penetration to the maximum treatment depth, and unlike dry mixing, grout is usually injected both during this phase and the withdrawal phase. The wet grout assists penetration in drier cohesive soils and in sandy and mixed soils, and in the uniformity of the mixing. Penetration speeds and withdrawal speeds normally are different, as are the injection rates during penetration and withdrawal. Some approaches employ a further complete cycle or partial repenetration to ensure a higher mixing efficiency. Rotation rates are lower than for dry soil mixing, but the BRN, described above, has been found to be helpful also for wet soil mixing. Topolnicki (2009) showed that the BRN should exceed 430 for a reasonable consistency in relation to the variation in strength of the mixed ground in single-axis mixing using cement grout in clay soils. It is expected this should also apply for multi-axis methods. In practice most wet mixing approaches in cohesive soils provide at least this level of BRN. Different considerations apply for wet soil mixing in cohesionless soils such as sands, and lower BRN values may apply.

As with dry soil mixing, electronic measurement and reporting is commonplace, and is also now increasingly being used to control the mixing cycles. The reporting and control provide confidence in the dosage rates with depth, and assist in minimising variability and waste.



Example of column arrangements

Example of mass and single column arrangements

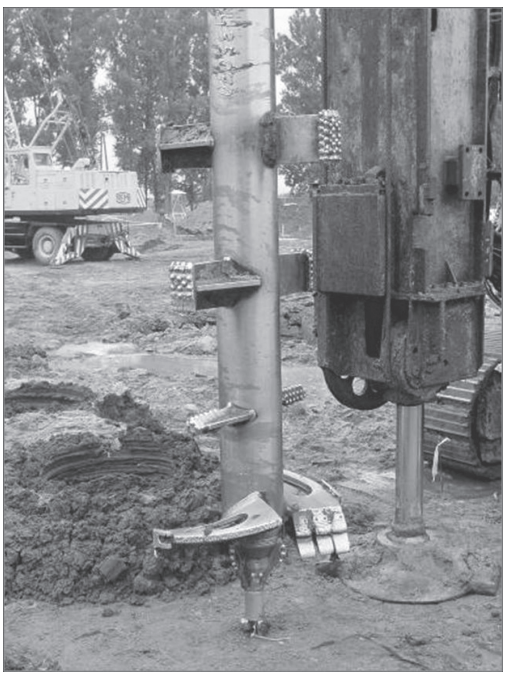
Figure 90.18 Methods for supporting embankments on soft ground using dry soil mixing

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Dosage rates vary typically from 100 to 350 kg cement or more per cubic metre of natural soil, with the higher dosage rates used in the softer and more organic soils, or where very high strengths are required by the particular application (see **Table 90.4**). Stiffness in the form of elastic modulus is less often required, but where measured is typically in the range of 75 to 100 × UCS (Filz, 2009). Mass coefficient of permeability, k, may be as low as 10^{-8} m/s or better where the grout mix, method and application demand, as shown below, but can be at least one or two orders of magnitude higher.

Wet soil mixing, like jet grouting, has found applications in a broad range of geotechnical problems. Given that relatively high strengths and consistency can be achieved, wet soil mixing can be used to provide foundations not only for embankments and the like, but also for relatively large loaded structures (see **Figure 90.20**). Wet mixed columns can be arranged to form gravity walls or alternatively slimmer forms can be reinforced with H section steel beams or the like, to provide additional bending capacity and stiffness and an excellent example is shown in **Figure 90.21**. Likewise in suitable ground conditions wet soil mixing can provide low permeability and has been used to provide new or remedial cut-off elements for dams and other barriers to water or contaminated flows. Triple-axis mixing is often more efficient than single-axis for this range of applications.

(a)



(b)



Figure 90.19 (a) Single axis mixing; (b) double axis mixing
Courtesy of Topolnicki

Soil type	Cement factor (Kg/cu m)	UCS at 28 days (Mpa)	Permeability k (m/s)
Sludge	250–400	0.1–0.4	1×10^{-8}
Peat, organic silts, clays	150–300	0.2–1.2	5×10^{-9}
Soft clays	150–300	0.5–1.7	5×10^{-9}
Medium/hard clays	120–300	0.7–2.5	5×10^{-9}
Silts and silty sands	120–300	1.0–3.0	1×10^{-8}
Fine to medium sands	120–300	1.5–5.0	5×10^{-8}
Coarse sands and gravels	120–250	3.0–7.0	1×10^{-7}

Table 90.4 Typical field strength and permeability data for wet soil mixing
Data taken from Topolnicki (2004)



Figure 90.20 Prepared foundation excavation showing 0.9 m diameter wet soil mix columns
Reproduced from Topolnicki (2004)



Figure 90.21 Reinforced soil mix columns to form retaining structure; Music Academy, Posnan, Poland
Courtesy of Topolnicki

90.6.3 Soil mixing using trench cutting

Another approach to soil mixing has been provided by the recent development of equipment which enables mixing of wet grout whilst cutting trench structures in the ground. Originally developed and used in Japan there are now several approaches available and in use in Japan, in the USA and across Europe. The major applications are in forming linear *in situ* structures such as cut-off barriers and retaining walls. Reinforcement in the form of steel sections can be used to provide additional structural strength where needed. Advantages of these systems, given the right conditions, are a reduction in the number of joints over competing methods, less waste for any required wall thickness as overlap is minimised, so leading to reduced costs and improved quality. Wet grout mixes are generally used, with similar mix proportions and final *in situ* mixed characteristics as with other methods of wet soil mixing.

The TRD equipment (**Figure 90.22**) consists of a large machine of about 100 tonnes and 8 m tall and effectively enables cutting and mixing by means of a chainsaw concept. An initial starter trench is formed into which a post is assembled and lowered into the ground to the required depth. Depths of 30 m or more have been treated. The post holds the cutting chain and injection proceeds as the machine crawls forward cutting a full depth face and providing a uniform mix in place material. Joints in the soil mixed trench material only occur if production is stopped, for example if only day shifts are being used. In this way joints are few and the system is particularly efficient for long cut-off barriers.

The CSM (cutter soil mixing) method (Stoetzer *et al.*, 2006) is increasingly being used world-wide. It uses combination cutter wheel arrangements attached to Kelly bars or suspended from ropes (**Figure 90.23**) to form panels up to 2.8 m in length or greater. An advantage of the method is that both fresh-in-fresh and fresh-to-hard panel construction joints can



Figure 90.22 TRD cutter soil mixing machine
Courtesy of Hayward Baker

be facilitated. Depths up to 55 m or so can be achieved with the rope suspended CSM. In relatively uniform soils, or for retaining structures up to about 20 m deep, mixing is conducted during cutting (penetration) and withdrawal. For deeper cut-off walls and for less uniform soils a two-phase approach is adopted using bentonite for temporary trench support during cutting. As with conventional diaphragm wall construction, the bentonite slurry is recirculated and cleaned by passing through de-sanding equipment. Grout is injected during the withdrawal phase and mixed with remaining soils.

90.7 Verification for grouting and soil mixing

It is important to verify the mass properties of the grouted or mixed ground in some way to ensure that the treatment has delivered the intended specification.

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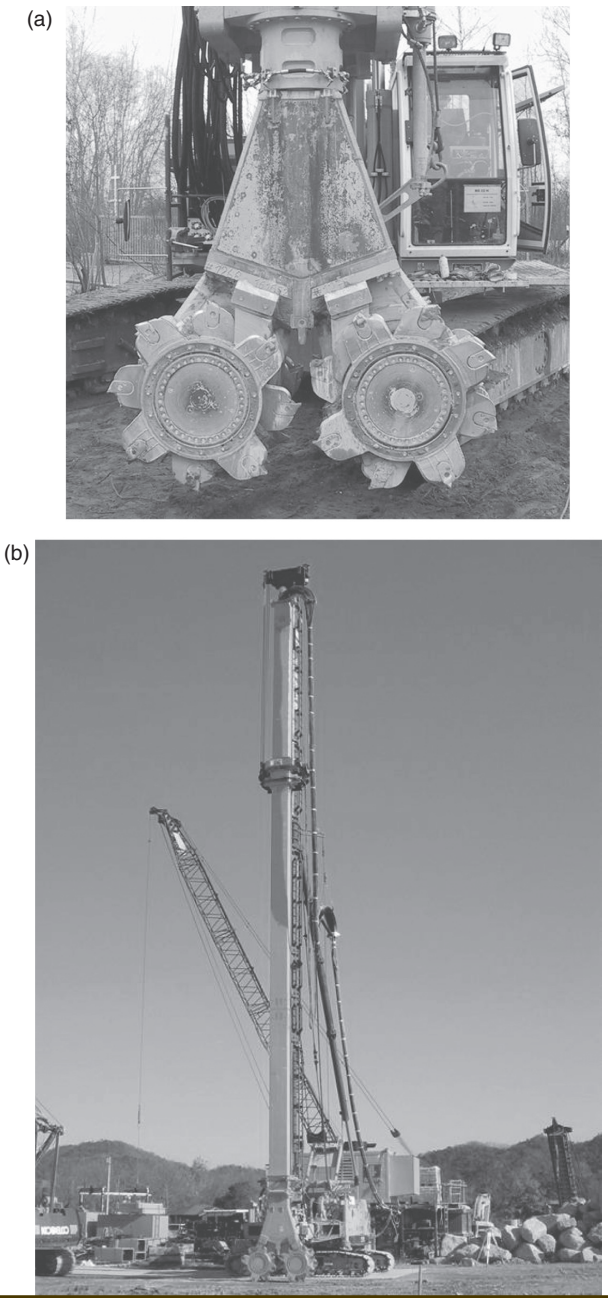


Figure 90.23 (a) Mixing wheels and housing; (b) Kelly guided CSM unit
Reproduced from Stoetzer *et al*. (2006)

Verification in fact commences with the careful execution of the work within the specified tolerances to an appropriate level of quality control. Further verification using field trials, laboratory or *in situ* tests or some other means is usually necessary. Attention should be paid to guidance available in the CIRIA reports C572 (Charles and Watts, 2002), C573 (Mitchell and Jardine, 2002) and C514 (Rawlings *et al*., 2000), and the European execution codes listed below. As with the processes themselves it is essential that advice from experienced specialists is taken.

Considerable care is needed in the selection, execution and interpretation of these tests, and all parties to the work need to be clear on the verification strategy and the interpretation of the results before testing commences. It is clearly wise to select a verification programme which has had a proven track record. **Table 90.5** indicates the main means of verifying treatment, but the reader should be aware that there are several other means, some specific to individual processes, which may be particularly relevant to their project.

Pre-contract trials at full scale are often useful, and sometimes essential, in order to establish the properties or even the viability of the prospective process. In jet grouting, for example, evidence of the diameters and associated properties may require the formation of full-scale test columns, which can then be examined in detail to provide confidence in conditions for which previous experience is small or unavailable (**Figure 90.24**).

For both dry and wet soil mixing, laboratory trial mixes are particularly useful, but cannot directly take account of variability in the ground, and lab mixing efficiencies will inevitably be different from *in situ* mixing. Careful interpretation of such tests is therefore necessary.

Tests on cores, or more rarely on block samples, recovered from the treated zone are often considered and can be good guides to performance. However, low sample disturbance, accuracy in aligning the coring tool, and high core recovery to properly represent the mass properties are needed. Realism in assessing the prospects of achieving good-quality cores, together with the employment of specialist crews experienced in verification coring, is essential to achieve a successful outcome. In wet soil mixing and in jet grouting, triple tube coring methods have shown good efficiency where the treated mass is suitable.

Another option possible in some situations for soil mixing and jet grouting consists of the recovery of 'wet' samples soon after completion of the process and before setting takes place. These samples can be placed in cylindrical or other approved moulds and tested at the specified time intervals.

A range of tests carried out from within boreholes can be conducted. Again it is important to ensure the positional tolerance of these, and take account of the possible effects of disturbance on the properties being measured. They are particularly helpful in measuring permeability as some feel for the mass behaviour is gained. Other common *in situ* tests such as SPT or CPT are commonly considered for verification tests. It should be appreciated, nevertheless, that these were developed for soil investigation and they can be unpredictable in assessing the properties of what in effect are artificial cemented materials produced by one of a range of grouting and soil mixing processes. The author is aware of sites for which the CPT grossly underpredicted the apparent *in situ* shear strength of cement bentonite injections, jet grouted columns and soil mixed masses, by comparison with tests on cores and wet samples. These systems can therefore only be recommended when there exists a good body of relevant information for the processes

	Permeation	Jet grouting	Soil fracture	Compaction	Dry soil mixing	Wet soil mixing
Permeability						
Pre-contract full-scale trials	Possible	Possible	N/A	N/A	Possible	Possible
Laboratory tests on trial mixes	Unlikely	Unlikely	N/A	N/A	Possible	Possible
Laboratory tests on *in situ* cored or block samples	Possible	Possible	N/A	N/A	Possible	Possible
Laboratory tests on *in situ* wet samples	Unlikely	Possible	N/A	N/A	Possible	Possible
Tests from boreholes in the treated ground	Possible	Possible	N/A	N/A	Possible	Possible
Full-scale assessment from piezometers	Possible	Possible	N/A	N/A	Possible	Possible
Unconfined compressive strength						
Pre-contract full-scale trials	Possible	Possible	Possible	N/A	Possible	Possible
Laboratory tests on trial mixes	Unlikely	Unlikely	N/A	N/A	Possible	Possible
Laboratory tests on *in situ* cored or block samples	Possible	Possible	Possible	N/A	Possible	Possible
Laboratory tests on *in situ* wet samples	Unlikely	Possible	Unlikely	N/A	Unlikely	Possible
in situ SPT or static CPT tests	These can only be recommended for situations in which good experience and calibration exists Note: they are useful for assessment of density change in the ground around compaction grouting bulbs					
Stiffness						
Pre-contract full-scale trials	Possible	Possible	Possible	Possible	Possible	Possible
Laboratory tests on trial mixes	Unlikely	Unlikely	N/A	N/A	Possible	Possible
Laboratory tests on *in situ* cored or block samples	Possible	Possible	Possible	Possible	Possible	Possible
Laboratory tests on *in situ* wet samples	Possible	Possible	N/A	N/A	Unlikely	Possible
in situ SPT or static CPT tests	These can only be recommended for situations in which good experience and calibration exists					
Pressuremeter or dilattometer tests	These can only be recommended for situations in which good experience and calibration exists					
Full-scale trial loading tests	Possible	Possible	Possible	Possible	Possible	Possible

Table 90.5 Guide to main verification methods for grouting and soil mixing

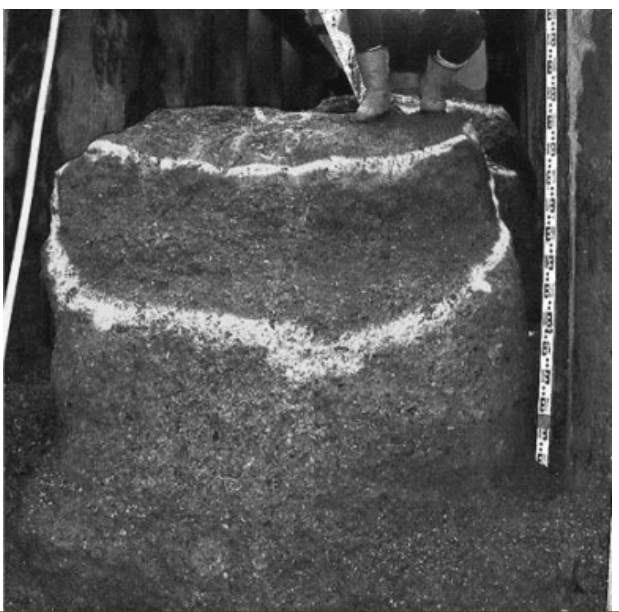


Figure 90.24 Example of jet grouted column formed as part of a pre-contract trial

and specific soils to be treated, or where sufficient pre-contract trials are specified in order to calibrate such tests with the required properties.

Finally, testing at or near full scale can be a powerful means of verifying the performance of the treated mass. However, this is often expensive and can suffer from lack of coverage of the whole volume of treatment. In assessing permeation, for example, such as for cut-off walls, piezometers suitably placed either side at the key levels can be used to check the overall cut-off performance. However, the timescales involved in assessment may be considerable and well beyond the construction programme. Similarly, foundation settlement under load can be assessed to some extent by full-scale loading on the improved ground, although usually needing further interpretation to cover the likely timescales and loading complexities imposed during the design life of the structure being supported.

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It is recommended this chapter is read in conjunction with

- Chapter 25 *The role of ground improvement*
- Chapter 59 *Design principles for ground improvement*
- Chapter 84 *Ground improvement*
- Chapter 94 *Principles of geotechnical monitoring*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

Chapter 91

Modular foundations and retaining walls

Cliff Wren Independent Geotechnical Engineer

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The introduction of the *Code for Sustainable Homes* (DCLG, 2009) has seen an increase in the use of modular off-site foundation systems. This is particularly noted in the case of house foundations and other lightly loaded structures. The benefits of using modular foundations are only just being realised and as the trend for 'off-site superstructure fabrication' continues to progress, the use of such sustainable options will increase. The emphasis of many of the leading ground engineering specialists has therefore been to explore and develop new systems that assist building contractors to achieve a higher code rating than would be realised by using traditional 'dig and dump' foundations.

91.1 Introduction

Structures that utilise a form of modular foundation have been with us for decades. The British landscape is peppered with excellent examples of just how forward-thinking our ancestors were. Sustainability was not high on their agenda. They had not heard of lean construction, Egan, or Latham, and they did not appreciate the effects of global warming and the lasting legacy of their actions. They built foundations for the structures like the ones shown in **Figures 91.1** and **91.2** because they had practical reasons to do so.

The methods they employed depended on the skills and equipment of the time, but were always tried and tested techniques. The drivers for their construction were purely practical. In the case of **Figure 91.1**, the requirement was to store grain off the floor in a dry environment away from rodents and other vermin, and in the case of **Figure 91.2** the need was to live above water. They were not designed using mathematical formulae and did not employ the concepts of soil mechanics or geotechnical engineering. They were built using techniques passed down from generation to generation and, more importantly, the methods they employed worked.

As engineers we seek to improve on the 'traditional' methods employed by our ancestors. The Egan report *Rethinking Construction* (Construction Task Force, 1998) brought to the attention of the construction industry the concept of 'lean construction'. One of the cornerstones of this concept is the use of modularisation, off-site techniques and 'just-in-time' delivery. The concept was not really new to the foundations industry – it had been installing pre-cast driven piles (a factory-produced product delivered 'just in time') for decades and had been fabricating pre-cast concrete walls and foundation beams since the early 1930s. Many of these early modular buildings still survive and are in good serviceable condition (BRE, 1984). However, the principles behind *Rethinking Construction* require the piling and foundations industry to take a more holistic view and move towards the use of a truly modular foundation. Generally,



Figure 91.1 16th century grain store foundation and staddle stones
Courtesy of Nigel Rake, all rights reserved



Figure 91.2 House on stilts
© Mikhail Nekrasov

this is not a practical solution – for instance, in the case of rotary bored or driven cast *in situ* piles. However, in the particular case of house foundations and lightly loaded structures, there have been recent (contractor-led) developments towards the use of modular off-site foundation *systems*.

91.2 Modular foundations

We can define a *module* as being 'a separable component, frequently one that is interchangeable with others, for assembly into units of differing size, complexity or function', and *modular* as being 'composed of standardised units or sections for easy construction or flexible arrangement'. What is therefore meant by a modular foundation? The historic 16th-century grain store foundation shown in **Figure 91.1** provides us with an excellent example of how a simple practical design concept of the past can be translated into modern concepts. The foundation *system* is modular; the staddle stones provided the support mechanism in a similar way to a foundation system patented some 450 years later.

Perhaps the most well-known example of a modular foundation is the pre-cast pile (**Figure 91.3**), a concrete unit manufactured in a factory environment and delivered to site, where it is subsequently driven into the ground. Individual pre-cast concrete sections (modules) can be joined together during the driving process and the final length of pile is therefore determined during the actual installation process. In the strictest sense the system is truly modular. A more in-depth review of modular piling systems is covered elsewhere in this publication: Chapter 81 *Types of bearing piles* provides further information on the construction of these and other displacement pile types; Chapter 82 *Piling problems* outlines some of the commoner problems that may be encountered; and finally, suitable design methods are discussed in Chapter 54 *Single piles*.



Figure 91.3 Pre-cast piles
Courtesy of Roger Bullivant Ltd, all rights reserved

In house building, the trend for 'off-site superstructure fabrication' continues to progress. There is now a diversity of approaches based on steel or concrete systems, as well as timber frame or panels. Timber-framed buildings had a bad press in the 1980s due to poor construction and maintenance issues, but they have recently made a comeback due to better product quality control and their green credentials. The influx of light steel and other panel systems from the Continent has huge implications for traditional cladding and roofing products, as well as site practices. This applies across the whole housing and offices sector. It is only in recent years that the foundation sub-structure has moved into an off-site environment. Designs based on modular foundation units (fabricated in a factory environment and assembled on site) have been around for many years, but it is only recently that they have been viewed in a more holistic way – the sub-structure becoming an integral part of the superstructure design.

91.3 Off-site manufactured solutions – the rationale

There is now strong evidence that our climate is changing and our planet is warming up (Stern, 2007). The 24 million homes in the UK account for over a quarter of the UK's carbon usage emissions (DCLG, 2007). As responsible contractors/engineers, the need to change our approach to construction has never been greater. The standards set out in the *Code for Sustainable Homes* (DCLG, 2009) (an environmental assessment method to measure the sustainability of new homes) will need to be achieved if we are to protect our future. We need a revolution in the way we build new homes – both to cut our carbon emissions and to respond to our changing climate – and we need that revolution to begin with this generation of designers, engineers and builders. Chapter 11 *Sustainable geotechnics* provides a holistic overview of sustainable geotechnics and also contains some excellent references which outline the subject in greater detail.

Homes account for around 27% of UK's carbon emissions (compared to air travel, which currently contributes 3% of the global carbon emissions, and cement production which contributes 2%). The emphasis of many of our leading ground engineering specialists has therefore been to explore and develop new systems that assist building contractors to achieve a higher code rating than would be realised by using traditional 'dig and dump' foundations. The contributing issues that modular foundations can affect in respect of the *Code for Sustainable Homes* are detailed in **Table 91.1**.

Several proprietary examples of modular foundation systems are currently marketed in the UK and all offer differing but significant advantages to prospective developers. The most efficient modular foundation systems provide major sustainable benefits (NHBC Foundation, 2010). To give an example,

Categories	Issues
Energy and CO_2 emissions	Building fabric
Materials	Environmental impact of materials
Responsible sourcing of materials – building elements	
Surface water run-off	Flood risk
Waste	Construction waste management
Pollution	Global warming potential of insulates
Health and well-being	Sound insulation
Management	Considerate constructors scheme
Construction site impacts	
Ecology	Protection of ecological features

Table 91.1 Contributing factors for modular foundations to Code for Sustainable Homes (DCLG, 2009)



Figure 91.4 Factory construction of foundation components
Courtesy of Roger Bullivant Ltd, all rights reserved

consider the following: 'traditional' trench-fill foundations for an average house (having a footprint of 80 m^2) will release 45 tonnes of CO_2 into the atmosphere. A modern composite reinforced concrete and galvanised steel foundation system will release 11 tonnes, a reduction of 75%. Even greater percentage reductions are calculated for water and raw materials usage (Arup, 2009).

Given the major impact that construction (and house building in particular) has on the environment, these long-term benefits could have significant implications for the building industry as a whole. However, they are only just starting to be realised in the UK. To a large extent, it is future legislation that is currently driving the use of such systems by national house builders; however, small-scale developers and individual self-builders are the ones who seem to be eager to embrace modular building techniques. They are pushing the boundaries by increasingly using these systems for bespoke one-off developments (www1, 2010).

Most modular foundation systems require no trench excavation, thus avoiding the hazardous practice of working in excavated ground. Components are manufactured off-site (**Figure 91.4**) and installed on-site (**Figure 91.5**) with a significant time saving on traditional house foundation construction methods. These advantages are more pronounced when the systems are installed in large volumes.

Significantly, most of these patented proprietary systems have been developed by specialist geotechnical contractors rather than general building contractors. The most common types are detailed below.

91.4 Pre-cast concrete systems

Modular pre-cast concrete foundation systems have been used successfully for many years, particularly in the housing sector where they form one of the larger component sub-assemblies for either conventionally built or modern methods of construction (MMC) dwellings. A variety of proprietary pre-cast suspended flooring systems are available found on either



Figure 91.5 Pre-cast concrete beams for modular foundation system
Courtesy of Roger Bullivant Ltd, all rights reserved

conventional strip or trench fill foundations for good ground conditions, or on piled supports in poorer ground. The more common types of system are:

(i) *Beam and block*, which is suitable for use at both ground and upper floors and is used in conjunction with standard format building blocks to provide a fast and cost-effective floor deck.

(ii) *Insulated beam and block*, which utilises pre-stressed beams in conjunction with expanded polystyrene blocks to provide inherent thermal insulation for ground floors.

(iii) *Hollowcore*, comprising pre-stressed and reinforced hollow-cored slab elements which easily accommodate the longest spans for domestic applications.

(iv) *Lattice girder*, which is a pre-cast concrete permanent formwork used in conjunction with a composite concrete topping. These floor types easily satisfy the requirements of *Part L* of the Building Regulations (BRE, 2006).

91.4.1 Composite reinforced concrete and galvanised steel

This type of foundation system is a lightweight, modular, composite, fully insulated foundation and ground floor system which will support any low-rise building in all ground conditions at an affordable cost and with significant environmental benefits. The system comprises fibre-reinforced high-strength screed on expanded polystyrene insulation which is incorporated into galvanised steel channel section floor beams. This floor is placed on composite reinforced concrete and galvanised steel primary support beams (**Figure 91.6**). Manufactured predominantly off-site in a factory environment, the system is assembled on-site, by hand, and is connected together with an appropriate amount of reinforced steel and fibre-reinforced *in situ* concrete (**Figure 91.7**).

The modular system is value-engineered to eliminate waste. It contributes to the sustainable design in new construction by substantially reducing the use of natural resources and subsequently the embodied carbon footprint of the building when compared to more traditional foundations. These types of modular foundation system generally have A+ ratings in the BRE green guide (BRE and Oxford Brookes University, 2009). Three types of floor can be used, depending on the level of code requirement. These deal with the Current Building Regulations (as at 2011), a floor to Code 3 (requiring the home to be 25% more energy efficient than one built to the 2006

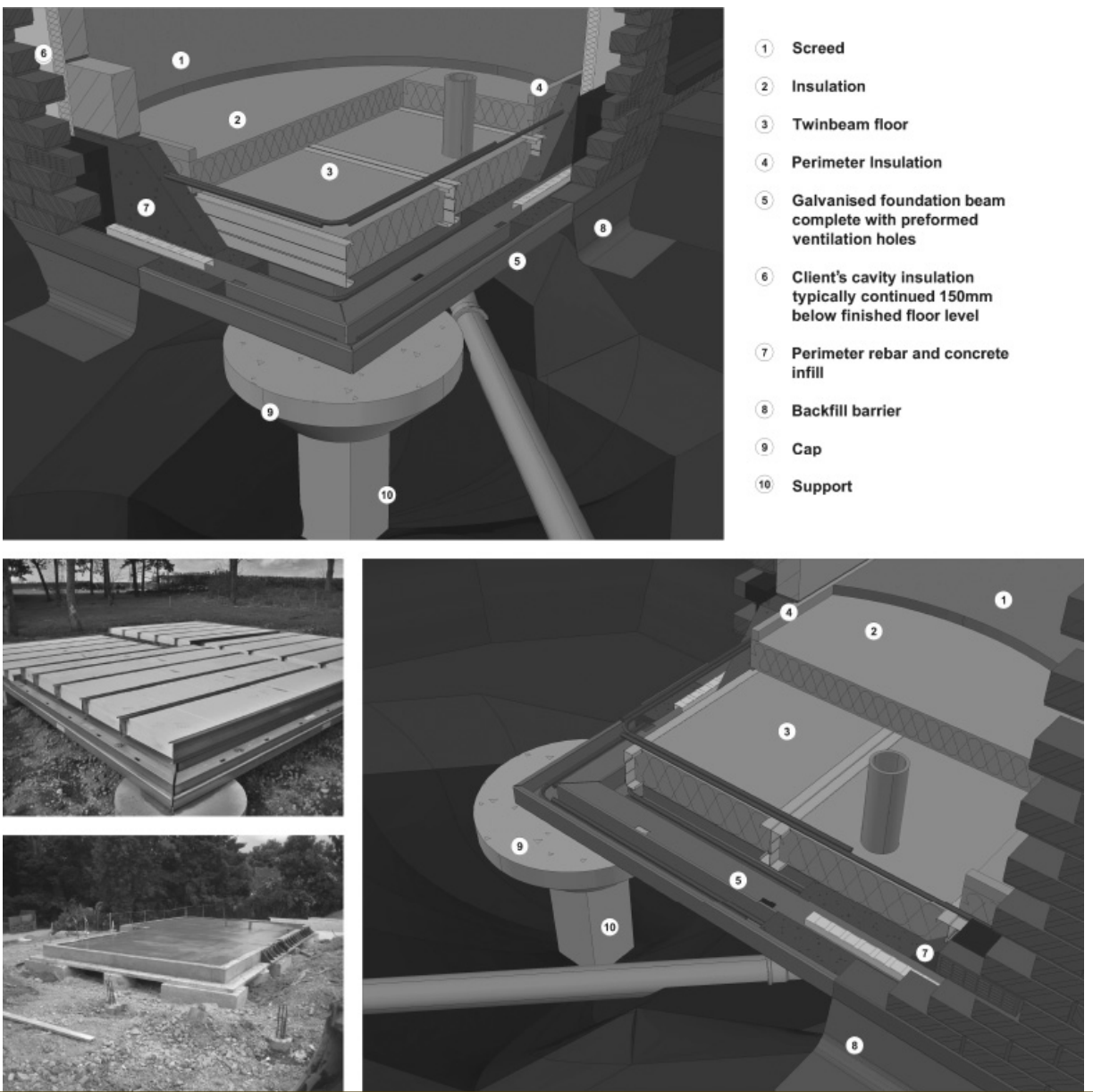


Figure 91.6 Composite foundation floor sections
Courtesy of Roger Bullivant Ltd, all rights reserved



Figure 91.7 Composite foundation floor assembly – SystemFirst™
Courtesy of Roger Bullivant Ltd, all rights reserved



Figure 91.8 Composite floor assembly supported on CFA piles
Courtesy of Roger Bullivant Ltd, all rights reserved

Building Regulations Standards) or a floor to Code 6 (the code requiring the home to be completely zero carbon by 2013). The manually assembled construction process eliminates the requirement for heavy plant, which significantly improves on-site health and safety. Each of the modular units weighs less than 25 kg, meaning that they satisfy the safety requirements for manual handling.

These types of composite modular system are 'flexible', in that they can accommodate superstructure designs based on the following forms of construction: timber frame, traditional masonry, structural insulated panel (SIP), lightweight steel, composite concrete panels, and insulated concrete formwork (ICF). The last two forms of construction require major modifications to the foundation system, which may prove uneconomical.

As with any foundation system, the site-specific geotechnical conditions will dictate the support to be used and it is therefore a fundamental requirement that a sufficient ground investigation is provided at design stage. There are a range of support products and techniques available to suit most modular foundation systems, such as pads, piers, displacement cones, vibro-concrete columns or piles. Although these systems have been developed with low-rise, domestic-type construction in mind, they can be adapted to suit other building categories by increasing the number of supports, limiting the floor and ground beam spans, as well as increasing the ground beam capacity.

91.4.1.1 Case study 1 – Bucks Green, West Sussex

On this small housing development, the client was determined to construct five detached dwellings to a very high standard and planned to achieve the highest possible rating with regard to the *Code for Sustainable Homes*.

The underlying geology of the sloping site, in West Sussex, was Weald Clay with a high plasticity index, necessitating the use of anti-heave precautions in the piles and below the pile caps, as well as a 250 mm void beneath the floor slab. A total of 178 (300 mm diameter) continuous flight auger (CFA) piles, drilled to a depth of 8 m, were required to support the dwellings (**Figure 91.8**).

To assist in achieving the desired rating, the client selected a composite reinforced concrete and galvanised steel system as the most appropriate floor system. The floor was designed to include under-floor heating throughout the ground floor to provide a U-value of 0.12 w/m^{2}K (a U-value is the measure of heat loss through an element expressed as watts per square metre for every degree Kelvin difference in heat either side of the element).

The standard foundation was modified at the design stage to accept brick plinths and a bespoke timber frame. The system proved to be flexible enough to include a 1.5 m level change between one of the dwellings and an attached garage; the level change was required due to the nature of the sloping site.

Many modular foundation systems also contribute greatly to the thermal efficiency of a building. A U-value of up to 0.12 W/m^{2}K can be achieved from the floor construction. As a comparison, values currently achieved with traditional block and beam floors are in the region of 0.18 W/m^{2}K.

91.4.2 Reinforced concrete piled raft

The Housedeck® foundation system is a form of piled raft and offers similar benefits over traditional foundations. The system in its basic form is shown in **Figures 91.9** and **91.10**.

There are many variations available; for example, a void can be incorporated to overcome the potential problem of clay heave. Apart from speed of installation, Housedeck® is particularly useful where trees are located adjacent to the development. The system can guarantee that the piles and construction will not sever crucial life-supporting roots. This is achieved by hand augering at pile locations and then moving the piles to avoid any roots that would otherwise be damaged by the piling



Figure 91.9 Reinforced concrete piled raft foundation – Housedeck®
Courtesy of Abbey Pynford Ltd, all rights reserved



Figure 91.10 Suspended reinforced concrete slabs, Abbey Pynford, Housedeck system, Waltham Abbey, Essex
Courtesy of Abbey Pynford Ltd, all rights reserved

process. Geotextile membranes and lightweight rigs are also used wherever possible to avoid crushing the roots.

Proprietary systems in themselves offer greater comfort to the customer as they are often subject to a quality management system.

91.4.2.1 Case study 2 – Waltham Abbey, Essex

The project involved the design and construction of a reinforced concrete raft foundation system for 48 no. residential units, incorporating two- and two-and-a-half-storey houses/apartments (**Figure 91.10**). The ground conditions of the site



Figure 91.11 Post-tensioned concrete modular foundation – Smartfoot®
Courtesy of Van Elle Ltd, all rights reserved

comprised Made Ground composed of stiff fissured clay up to 2.7 m in depth; alluvium, a soft grey silty clay down to 4.0 m; loose to medium dense well-graded sand and gravels (Kempton Park Gravels) 7.0 to 7.5 m below existing ground level; and London Clay, stiff to very stiff fissured clays with medium to high shrinkability. The piling contractor undertook the design and construction of the piling platform in accordance with construction design and management (CDM) regulations and the Federation of Piling Specialists recommendations, and then designed and constructed 460 no. 300 mm diameter continuous flight auger piles to depths varying from 9.0 to 14.0 m. In this particular example, the specialist sub-contractor also designed and constructed the 2 240 m^2 of suspended reinforced concrete slabs which, because of the shrinkable clay strata, had to incorporate a full 225 mm void to overcome potential heave.

91.4.3 Post-tensioned concrete modular systems

A post-tensioned pre-cast concrete modular foundation system offers similar benefits to the systems already outlined above for a wide range of structures varying from traditional build, modular structures, housing, hotels, prisons and schools (**Figure 91.11**).

Like the preceding modular systems, this type of foundation provides many benefits over traditional strip or trench-fill foundations. It is cost-effective and fast (up to 400 m of beam can be installed in one day), and can be installed to high tolerances (±3 mm). The system is suited to traditional and modular type construction and can be installed with or without piles.

91.4.3.1 Case study 3 – Houghton-le-Spring, Sunderland

The client wanted to develop a site of affordable housing for the local community. Due to the nature of the project, the development had to be both time- and cost-efficient – the developer wanted the houses up as soon as possible. In view of the client's overriding need for the foundations to be installed in the shortest



Figure 91.12 Post-tensioned concrete modular foundation system, Houghton-le-Spring, Sunderland
Courtesy of Van Elle Ltd, all rights reserved

possible time and at the most affordable cost, the project was therefore scoped using a post-tensioned pre-cast modular foundations system and concrete pre-cast driven piles (**Figure 91.12**). There were 58 plots on the site and 600 no. 200 mm × 200 mm piles, and around 2 500 m of ground beams were installed. The job was completed within eight weeks. The scope of works turned out to be a great example of off-site construction methods. Both the piles and ground beams were manufactured off-site to site-specific designs. What followed was a clean site, accurate installation and fast completion, proving all of the benefits provided by the use of a modular foundation system.

91.5 Modular retaining structures

A retaining structure or retaining wall is used to provide lateral support for a soil mass and may be used to carry vertical load, for example bridge abutments or basement walls. The varieties of structural arrangement that may serve as a retaining wall are almost unlimited. Their classification is based on their method of achieving stability; the most common are the gravity wall, cantilever wall, counterfort wall, buttress wall, diaphragm wall, sheet pile wall and reinforced earth wall (Clayton *et al.*, 1993). For a more in-depth review of retaining walls, reference should also be made to Chapter 62 *Types of retaining walls* and Chapter 64 *Geotechnical design of retaining walls* and to Chapter 67 *Retaining walls as part of complete underground structure* as part of complete underground structure.

The use of modularisation is beneficial in many retaining structures, both in terms of cost and productivity. For these reasons, the use of repeating modular units to form structural retaining systems has long been favoured by foundation specialists. Gravity walls, for example, which rely on their own weight and rigidity for stability, can be constructed using smaller interlocking modular units. Examples include gabion walls, crib walls and pre-cast reinforced concrete walls. Gabions are rectangular baskets made from galvanised steel mesh or woven strips filled with stone rubble or cobbles. They are aesthetically pleasing and provide a quick method of installing free-draining wall units. Crib walls are formed with interlocking pre-cast concrete units (or timber). Stretchers run parallel to the wall face and headers are laid perpendicular to the stretchers. The space formed by the 'cribs' is then filled with free-draining material such as rock, cobble or gravel. RC walls are the commonest modern form of gravity wall and are suitable for walls up to 6 m in height.

Interlocking sheet piles are commonly used for embedded retaining walls, both for temporary works and permanent structures. A factory-produced product, these steel (or plastic) 'modules' are designed to provide the maximum strength and durability at the lowest possible weight. The design of the sheet pile section interlocks facilitates pitching and driving, and results in a continuous wall with a series of closely fitting joints (ArcelorMittal, 2008). Steel sheet piling has traditionally been used for river control and flood defence structures, and is a tried and tested material for the construction of quay walls for ports and harbours. It is also an ideal material for constructing basement-retaining walls as it requires minimal construction width. One example where steel sheet piling has been found to be particularly effective is for the creation of underground car parks (SCI, 2001). The reader is directed to the following references which provide further detail on many aspects of the use of modular foundations.

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91.6.2 Useful websites

Association of Specialist Underpinning Contractors; www.asuc.org.uk

Federation of Piling Specialists; www.fps.org.uk

Precast Flooring Federation; www.precastfloors.info

It is recommended this chapter is read in conjunction with

- Chapter 11 *Sustainable geotechnics*
- Chapter 99 *Materials and material testing for foundations*
- Section 5 *Design of foundations*
- Section 6 *Design of retaining structures*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Section 9: Construction verification

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Chapter 92

Introduction to Section 9

Michael Devriendt Arup, London, UK
Michael Brown University of Dundee, UK

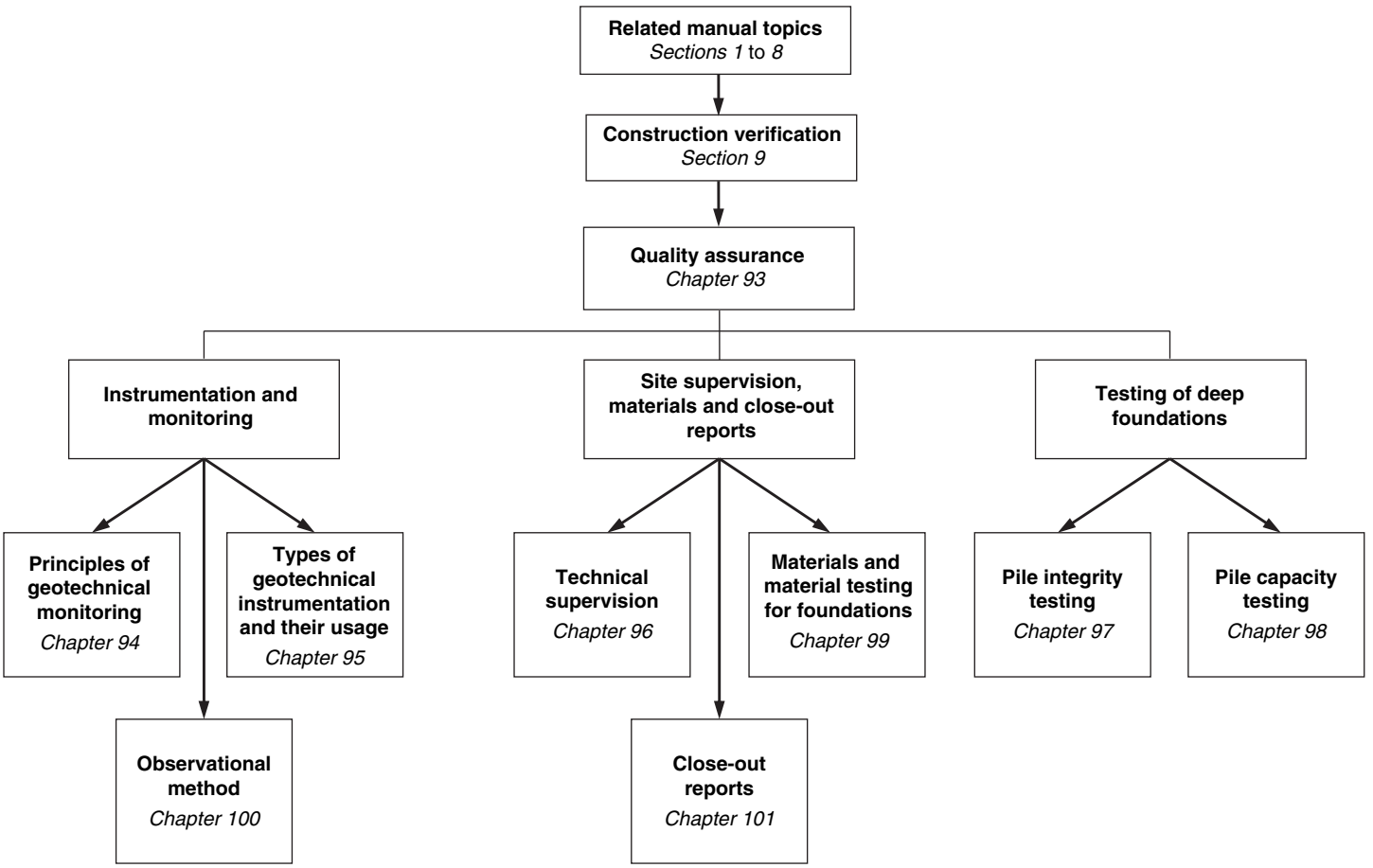


Figure 92.1 Layout of chapters in Section 9

Construction verification can be described as the process of providing test or monitoring results to contribute towards design or management of the works, or to confirm that construction work is being carried out in accordance with the proposed design, specification and appropriate standards.

In no other area of construction is the inter-relationship between design and construction so pronounced. Ground is inherently variable so the main construction material can never be known in advance with enough precision. Additionally, design parameters are often dependent on the methods of construction, so can be easily invalidated by minor changes in technique or in soils encountered. Hence construction verification has a much higher significance in geotechnical design than it does in most other areas of construction. Additionally, there is much to be learnt from observing construction and most innovations in geotechnical engineering have their roots in watching and understanding construction taking place. Hence there is a long-term virtuous circle in improving the links between construction and design. Designers must understand how geotechnical works will be enacted and constructors must understand the significance of the design assumptions underpinning the methods they are using.

This section is composed of nine chapters (see **Figure 92.1**) and begins with Chapter 93 on quality assurance (QA) which outlines the processes and management systems that may be put in place to ensure a minimum quality of the final construction product. One of the most important elements of a QA process is construction verification which is particularly important for geotechnical works as completed elements are usually buried and difficult to inspect subsequently. They are

also amongst the most heavily loaded structural elements and their construction is invariably on the project's critical path. This results in a need for early identification and remediation of non-conformances.

The remaining chapters in the section can then be divided into three general themes:

Instrumentation and monitoring

- Chapter 94 *Principles of geotechnical monitoring*
- Chapter 95 *Types of geotechnical instrumentation and their usage*
- Chapter 100 *Observational method*

Construction verification and close-out reports

- Chapter 96 *Technical supervision of site works*
- Chapter 99 *Materials and material testing for foundations*
- Chapter 101 *Close-out reports*

Testing of deep foundations

- Chapter 97 *Pile integrity testing*
- Chapter 98 *Pile capacity testing*

Geotechnical monitoring provides confirmation that works are being carried out in a safe, controlled and efficient manner. By setting pre-agreed limits, monitoring also provides information on when and how contingency measures should be adopted. Comparisons of the monitoring data can also be made with design calculations or results can be back analysed to improve calculations on future projects with the potential for significant cost savings. Although monitoring may be viewed as being beneficial, it is important that systems are installed with a clear objective and a plan of how the measured data will be implemented. Monitoring is also very useful for avoiding claims from third parties on the effects of construction on them and their activities – movement, noise and vibration monitoring can be particularly valuable for refuting claims which could otherwise result in long and costly legal procedures.

Site supervision, close-out reports and materials testing provide confidence that the site works are being carried out and documented in an appropriate manner. Improvements in data recording, management and storage have increased the importance of these subjects, particularly when modifications are carried out to existing structures or where sites are redeveloped. For example, there has been more interest recently in the re-use of foundation elements, especially deep foundations in congested urban environments. Knowledge captured as part of the original construction (as covered in all of the chapters in this section) will form a valuable source of data for future construction teams which may be faced with a new set of challenges.

The testing of deep foundations allows efficiencies to be made in the design of foundations and can confirm that the piles have been constructed in accordance with the design intent. Deep foundation testing has seen recent changes in techniques (e.g. automated static testing, rapid load testing) as well as new applications through the re-use of piles. This means that the engineer needs to decide:

- when testing is necessary;
- how many piles to test;
- the most appropriate techniques to use;
- how to assess and interpret the findings of testing.

The publishing and dissemination of pile test data is also invaluable for improving the collective knowledge for the design and construction of piles in similar materials in the future.

All of the chapters covered in this section contribute significantly towards the centre of the geotechnical triangle (Burland, 1987 and Chapter 4 *The geotechnical triangle*). The geotechnical triangle is used as a framework for geotechnical engineering; its centre focuses on:

- *establishing precedent* E.g. the use of instrumentation, monitoring and the observational methods for new construction techniques, or where ground conditions are unfamiliar or have uncertain behaviour, or are difficult;
- *the use of empiricism* E.g. pile capacity testing to refine design assumptions and increase efficiency;
- *forming well-winnowed experience* E.g. technical site supervision, selection of appropriate materials and dealing with non-conformances.

The range of subject matter in this section has been chosen to assist an engineer in making appropriate decisions to manage and verify construction processes. It is important, however, to realise that such techniques are only one component in the overall strategy to successfully complete a set of foundations or other geotechnical works. Construction verification must also consider the influence of the ground profile, soil behaviour and the need for an appropriate model to compare experience with. In this respect, the following quote from Terzaghi (1935) summarises the importance of having knowledge from both theory and experience: 'Experience alone leads to a mass of incoherent facts. But theory alone is equally worthless in the field of foundation engineering, because there are too many factors whose relative importance can be learned only from experience.'

As a profession, we can collectively make improvements to existing knowledge by utilising the best practice, some of which is contained in this section. This will enable improvements and efficiencies to be made on current and future projects.

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Chapter 93

Quality assurance

David Corke DCProjectSolutions, Northwich, UK
Tony P. Suckling Balfour Beatty Ground Engineering, Basingstoke, UK

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Quality assurance is an essential part of the design and construction process and provides confidence to the other parties to the contract that the end product will be built to at least a minimum standard. The specification defines this minimum standard – in geotechnical engineering there are many different model specifications available to suit different construction processes. An additional complementary project-specific specification is needed considering the unique nature of the project and its ground conditions. Workmanship is verified either by a resident engineer, or similar, or by the contractor undertaking self-certification. A key component of quality assurance is the identification and reparation, if necessary, of non-conformances. If any workmanship is thought to be faulty, or if there is a failure, then a forensic investigation will need to be carried out. The records produced by the quality assurance procedure provide factual data for use as part of this investigation.

93.1 Introduction

Geotechnical works are often supplied by different parties undertaking the choice of the type, design, procurement and construction of the works. However, all of these are inter-dependent and a failure in one will likely lead to a failure of the whole works.

Quality assurance (QA) is the management system that will ensure that all key parts of the geotechnical design and construction process are undertaken consistently in order to provide at least a minimum standard. Certification of the QA system by an accredited independent body is the evidence that clients, consultants or contractors can rely upon so that each party does not need to audit the other organisations.

A designer will use quality control (QC) systems to ensure that the design and checking process is undertaken to meet the QA requirements. The contractor will use QC systems to ensure that the construction is in accordance with the specification and design. Dependent upon the contractual arrangements on any particular project, the contractor will either have his workmanship verified by an external party, such as a resident engineer (RE), or will be required to self-certify his own work.

Identifying and rectifying non-conformant work is a key part of the QA process. For geotechnical work, it is paramount that a thorough forensic investigation is carried out at the earliest opportunity so that these issues are closed out early on in the construction process.

93.2 Quality management systems

For background information on quality management systems refer to ISO 9001, ISO 14001 and OHSAS 18001.

Both the design and construction of geotechnical work need to produce an end product that meets three key criteria:

(i) to have an adequate safety margin against failure;

(ii) to have acceptable displacements over the likely range of applied loads;

(iii) to be durable for the stated design life.

For UK design work, *British Standards* and *Codes of Practice* define methods of achieving these criteria. Execution codes, as listed in Chapter 78 *Procurement and specification*, also provide recommendations to help the designer achieve the criteria.

For geotechnical work it is essential that the method of construction is compatible with the design, and that the workmanship meets at least the minimum requirements of the design. The contractor will use QC systems to ensure that these are achieved. QC systems are important in all aspects of construction, but are especially so in geotechnical work as there may be variation in the ground conditions not shown by a site investigation – which only provides information from a limited and discrete number of locations. Many geotechnical processes, such as piling, have to be carried out without necessarily having the knowledge of specific ground information at the particular location. Consequently, continuous monitoring of these processes is a key aspect of the QA management system.

93.3 Geotechnical specifications

The specification defines the minimum standard of work necessary to meet the requirements of the design and of the employer. Because there are many geotechnical processes and solutions, there are many model specifications available; some of these are listed in Chapter 78 *Procurement and specification*.

Model specifications should be used because these have been written by experts in the field and will have been subjected to critical scrutiny by the geotechnical community. However, geotechnical processes can be undertaken with variations and so systems and methods may vary from those stated in a model specification. Such variations in the method of construction or equipment must be clearly identified in the

specialist sub-contractor's method statement and be compatible with the design.

Geotechnical engineering is a very innovative part of the construction industry and both design methods and construction processes are improving all the time. This means that model specifications, if not recently updated, can become outdated and may even be contradictory to current practice. It is essential that experienced geotechnical engineers are used to assess any proposals that would vary the requirements of these specifications.

In addition to model specifications, a project-specific specification is very important for geotechnical work because ground conditions are unique for any site. The particular aspects which may affect the geotechnical work need to be stated so that all concerned can be informed. Project-specific specifications should include all available factual information on the ground conditions, including the groundwater conditions. Clause B1.2 of the *ICE Specification for Piling and Embedded Retaining Walls* (2007) is a good example of what needs to be included in a project-specific specification.

93.4 Role of the resident engineer

The detailed responsibilities of the RE on any site will probably not be defined in the contract documents. Normally the role of the RE is to supervise the works on behalf of the employer and to make sure that the employer's project is delivered as intended, see Chapter 96 *Technical supervision of site works*. If the engineer is also the designer of the geotechnical work, then the RE will also be supervising the works to ensure that the standard of construction is in accordance with the design and specification requirements.

To save costs, it has become common for there to be no RE at all, or for the RE to be on site on a part-time basis with no formal power under the contract. This is not recommended except for the most straightforward of geotechnical work. Geotechnical engineering requires experience of a wide range of ground conditions and of complex construction processes, and it can only be of benefit to the project, and ultimately the employer, to have an RE present. It is preferable that the RE supervising geotechnical work should be a geotechnical engineer experienced in the prevailing ground conditions and construction process being used. There are many specialist geotechnical consultancies that can provide this service if the employer's professional team does not have the appropriate experience.

93.5 Self-certification

Most geotechnical work is carried out on a design and construct basis by specialist organisations that are often contracted to self-certify their work and provide a warranty for it. Self-certification by the trade contractor or specialist sub-contractor requires them to apply the QA processes on site in relation to their own works. If a project is self-certified there may not necessarily be an RE on site; if there is, they will have little or no contractual responsibility for the self-certified works. The process is often associated with the use of the new engineering contract (NEC), especially on large civil engineering projects, although the NEC itself does not impose self-certification, nor does it actually impose QA. Consequently, any requirement for self-certification will either be defined in specific contract amendment clauses ('Z' clauses in NEC), the *project specification* or in *works information*.

Self-certification by the contractor reduces the burden on the employer and the professional team, particularly where new technologies are involved. If the process is carried out efficiently, the specialist contractor is best placed to coordinate the construction and the checking of the construction process. Any faults or non-conformances should be found and dealt with at the earliest opportunity, minimising the cost of rectification and any consequences.

93.5.1 The self-certification process

The fundamental requirements of the self-certification process are that every part of the construction process should be monitored for compliance and performance. The basic questions are:

(i) Does the work satisfy the specification?

(ii) Is the work being implemented as planned?

(iii) Are the construction and design requirements being met?

The contractor's staff on site should be competent to self-check their own work and supervisory staff made responsible for review and inspection.

The employer will need to approve the contractor's quality management system and will monitor that it is being applied correctly, and carry out spot checks on finished work. Under NEC option 3 (target cost), non-conformances identified by the contractor are paid for by the employer. However, if they are not identified by the contractor but found subsequently, the contractor pays – which provides an incentive to the contractor to ensure that their quality control is applied consistently. If the process is to be carried out with confidence, the contractor must set out in his quality management system a detailed description of how the construction standards will be monitored, checked and maintained, at all times.

Clause 40.3 in the NEC describes how self-certification works best:

> The contractor and the supervisor each notifies the other of each of his tests and inspections before it starts and afterwards notifies the other of its results. The contractor notifies the supervisor in time for a test or inspection to be arranged and done before doing work which would obstruct the test or inspection. The supervisor may watch any test done by the contractor.

The Construction (Design and Management) Regulations 2007 (CDM) require the contractor to:

> Plan, manage and monitor construction work carried out by him or under his control in a way which ensures that, so far as is reasonably practicable, it is carried out without risks to health and safety.

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These CDM requirements must be satisfied in addition to any self-certification requirements, including monitoring and testing.

The records that need to be kept and submitted as part of the self-certification process need to be very detailed, and are likely to be incorporated into the contractor's method statements. The contractor's quality management system needs to define who will be responsible for the implementation of each element of the works, who will be responsible for independent inspection and, if appropriate, who will be responsible for auditing the whole process.

An important part of the self-certification process is the ability to react quickly to any revealed non-conformances. The processes and responsibilities for dealing with these situations should also be defined by the contractor's quality management system.

93.6 Finding non-conformances

Undetected problems with work carried out in the ground can have severe consequences later on, including major financial implications for all involved. For example, a defective pile can normally be easily replaced during the piling works, but once the sub-structure and superstructure are in place it can be very difficult to do so. (Also refer to Chapter 82 *Piling problems*.)

The methods for finding non-conformances vary depending on the construction process, but would normally involve a combination of some or all of the following: design review, observation, measurement, testing and investigation. The most common non-conformances in geotechnical work are described below.

93.6.1 Concrete in the ground

Concrete placed in the ground is particularly at risk from expensive investigation and remedial measures if the concrete test results subsequently prove to be unacceptable. The first point to consider is the source of the concrete. Ready mixed concrete is often delivered to the site, but on larger projects the concrete may be batched and mixed on site. The quality scheme for ready mixed concrete (QSRMC) certification for the batching plant should be checked, along with the date of certification and any changes that may have been made at the batching plant since then.

The strength of blended mixed concretes can be particularly sensitive to weighing accuracy. These are mixes which use either pulverised fuel ash (PFA) or ground granulated blast furnace slag (GGBFS) as partial replacement for the cement content. The higher the cement replacement ratio, i.e. the less cement the mix contains, the more sensitive the ultimate mix strength becomes to the batch weight of the cement.

Different types of concrete have different characteristic rates of gain of strength that are well documented; they can be used to assess the likely 28 or 56 day strength from earlier testing. Careful tabulation, analysis and plotting of early age (3 or 7 day) testing maximises the chance of early identification of any problematic concrete. The presence of low strength concrete may affect its durability, the protection of reinforcement and the structural capacity of the element.

It is always advisable to review the early cube strength test results in order to check for any indications that the specified strength may not be achieved. If low strength concrete results occur, with a knowledge of the typical rate of gain of strength characteristics of the particular type of mix, it may be possible to predict whether or not the required strength will be achieved – even if at a later date than specified. Alternatively, cores can be taken and tested to establish the actual *in situ* strength of the concrete.

93.6.2 Pile load testing

One risk associated with load testing is that in certain ground conditions, the ultimate load capacity may increase with time (see Chapter 98 *Pile capacity testing*). Consequently, testing foundation elements such as piles too early may result in underestimating their load capacity, even if the concrete strength has developed sufficiently to carry the load. If early testing of a pile, typically 7 to 14 days after installation, does not verify the required load capacity, re-testing at a later date may show some improvement. This behaviour can affect the performance of bored and driven displacement piles in fine-grained soils and rocks.

If load testing cannot verify the required pile load capacity, the most common solution is to down-rate capacity and install additional piles which may require modifications to the sub-structure.

93.6.3 Integrity testing of piles

There are two basic types of integrity testing for piles, see Chapter 97 *Pile integrity testing*. The sonic echo method is usually restricted to small to medium diameter piles due to the limited energy that can be imparted with a small hammer. The potential difficulty with this testing is that although the measurement of the response to the applied impulse is common to all practitioners, the signal processing that is applied to produce the test response can vary. This may result in testing by one organisation showing an anomaly in a pile, whereas testing by another may not show the anomaly. Integrity problems are least likely to occur in relatively uniform ground conditions and are most likely to occur in highly variable ground conditions.

Reported anomalies can also result from reflections at the bottom of a reinforcement cage or from a bulge in the concrete in soft ground, as well as from defects such as cracks and necking of the concrete section.

If integrity testing does indicate an anomaly and retesting is carried out, the reported results should include the recorded data, i.e. before processing, so that it is available for an independent re-analysis. Ultimately it is often necessary to excavate out around the pile to investigate the true nature of a reported anomaly (see **Figure 93.1**). Small diameter piles are particularly



Figure 93.1 Temporary casing withdrawn from pile after initial setting of the concrete

susceptible to lateral impact or ground-induced loading, e.g. from a truck, piling rig or crane passing by too close. These piles need to be protected from such events either by increasing reinforcement provision (using high visibility marking on any protruding reinforcement), or by fencing off completed areas. Trimming piles can also induce pile damage which will not be identified by integrity testing, if the testing takes place before the pile is trimmed down to its cut-off level.

For cross-hole sonic logging (see Chapter 97 *Pile integrity testing*), apparent anomalies can occur if the tubes are locally debonded from the adjacent concrete, or if there is a small volume of no-fines concrete close to the tube.

93.6.4 Retaining wall deflections

Retaining wall deflections are often specified as a means of controlling a wide range of performances, as they are relatively simple to measure. However, care should be taken to specify deflections that are appropriate to the required performance and that should realistically be achieved, to ensure that economy in design is maintained. Any behaviour which is not 'as predicted' must be observed and promptly reported to experienced geotechnical engineers.

93.6.5 Prop loads

In major excavations it is becoming more common to monitor the loads in props, the largest of which may carry loads of several hundred tonnes. Monitoring of the loads, which may include an initial pre-stress load, should be carried out from installation, and the measured loads compared with the predicted ones for each relevant stage in the construction. If the monitoring shows that there are early signs that the prop load is increasing faster than predicted, this may indicate not only that the final load in the prop may exceed the allowable value, but also that the loads on the supported wall are higher than anticipated – which may result in overstressing of the wall itself.

The first thing to establish is whether the prop overload is indicating that the total load from the wall may have been underestimated, or the distribution of load from the wall to the prop(s) is not as predicted. Increased loads are often due to temperature effects or to construction imperfections, or they may indicate a departure from the design construction sequence or a change in the construction itself.

In order to be able to redesign the construction process, the design must be reassessed and parameters and loadings reviewed and adjusted until the design model predicts the measured prop load(s) and the observed wall deflection. Then, using the revised parameters and loading, additional props and/or berms can be considered in a revised design and construction sequence.

93.6.6 Lack of cover to reinforcement

The three main reasons for a pile, barrette or diaphragm wall panel losing cover to the reinforcement cage are:

Concrete If the concrete mix is too stiff to flow and fill the annulus outside the reinforcement, then the thickness of the concrete cover is likely to be variable, and typically be thinnest in front of the reinforcing bars.

Reinforcement spacing The specification should state the minimum spacing for both vertical and horizontal reinforcement so that there is adequate space for the concrete to flow in between the reinforcement and completely fill the annulus beyond it. The execution codes BS EN 1536 and BS EN 1538 give the minimum reinforcement spacing for various types of piles and for diaphragm walls.

Tremie concreting Control of the flow of fresh concrete into a pile, barrette or diaphragm wall panel is essential in order to ensure that no segregation of the concrete takes place, and that old hardening and stiffening concrete is not continually pushed vertically upwards where it could impede the flow of the fresh concrete around and to the outside of the reinforcement. The specification combined with the contractor's method statement should describe the acceptable procedures for placing concrete by tremie pipes.

Remedial measures can include breaking of exposed concrete back to the reinforcement and then re-concreting the surface to make good the cover. The difficulty with this method is that if the cover has been compromised on the exposed face of an element, it is also likely to be compromised on unseen

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and relatively inaccessible faces cast against the ground. In extreme cases, measures such as jet grouting adjacent to the ground face may be required.

93.6.7 Water ingress

The specification should set out the minimum standard for water ingress into any excavation below the water table (see **Figure 93.2**) and how this is to be measured, although this may be modified to suit particular usage requirements. If water ingress does not satisfy the specified requirements, then it is most common to resort either to grouting of the ground (by drilling from an exposed face) or, in some situations, to drilling from the ground surface.

There are two types of risk associated with this particular problem. Firstly, there is an inevitable delay while remedial grouting takes place. Secondly, and potentially more serious, water flow may have an impact on the integrity of the wall itself or induce ground settlements that may cause damage to adjacent structures.

93.7 Forensic investigations

The definition of a failure is 'an unacceptable difference between expected and observed performance' (Leonards, 1982). When geotechnical failures occur, they are often a surprise event and can happen very quickly. Having said that, it is also common to find that there were earlier warning signs that were either unseen, or seen but unheeded. It is common to find that when first called upon to investigate a failure, there is a lack of relevant information.



Figure 93.2 Example of a damp water patch

Forensic engineering is becoming increasingly important since foundation and other geotechnical failures can lead to litigation and possibly criminal action. The causes of a failure may be complex, but it is important to be able to explain the failure in simple terms to those charged with resolving any contractual or legal matters arising from the failure.

Failures can occur, for example, due a fundamental flaw in the design assumptions, or due to a small defective detail. There is no unique recipe for failure and no definitive exhaustive checklist of possible causes.

Attention to any possible warning signs may give some advance notice of an impending failure, may save money and ultimately may save lives. Simple signs, such cracking of concrete sections, cracks around bolted or welded connections in steel structures, or unexpected deflections, should be noted and reported to experienced engineers, and reacted to promptly. To ignore any such signs may lead to expensive or even fatal consequences.

93.7.1 The process

Following a failure, information gathering is often a cyclical process. Initial documents provided are reviewed, collated and assessed. This invariably leads to requests for other documents that were mentioned but not provided, or there may be existing documents that have not yet been referred to. Much of the documentation presented may ultimately prove to be irrelevant, but everything should be considered. Small pieces of information contained in large documents can turn out to be vital.

It is necessary to be completely methodical with all the information provided, which should be catalogued and arranged chronologically. The precise order and timing of events leading up to a failure is often important.

Cross-checking information is always an important part of the forensic process. For example, do the 'as constructed' records agree with the approved design and construction sequence? Is the ground investigation data (including field and laboratory test results) as would be expected, taking into account the local geology? Are any materials tests consistent within themselves, e.g. are variations in concrete strength reflected in variations of the measured concrete density?

When reviewing the information, note all the important points as they are found. It is frustrating to remember seeing something important, but not being able to remember which document/page/line it came from.

93.7.2 Design

It is often the case that the fundamental cause of a failure is that the geotechnical or structural behaviour assumed for design purposes is not appropriate and does not reflect reality.

The following quotations from eminent geotechnical practitioners relate specifically to the topic of geotechnical design, and have much to commend them:

'Simple calculations based on a sound conception are far more meaningful than elaborate calculations which ignore controlling factors.' *Ralph B Peck*, 1973

'Recent examples of inappropriate analysis … suggest modern engineers could benefit from a wider appreciation of elementary principles.' *Sir Alan Muir Wood*, 2004

'I express much concern over the extent to which engineers, presented with a technical problem, tend to fly directly to numerical solutions without pausing to understand the physical basis of the problem. Too often, the numerical model does not address the actual conditions, or provides output in a form that confuses rather than informs. Almost universally, the apparent precision of a solution bears no relationship to the uncertainty of the data. The ability to make a first estimate on the basis of a sketch and the formulation of equations for simple analysis continues to provide an essential tool to avoid expensive errors and misunderstandings. ' *Sir Alan Muir Wood*, 2004

When considering the design of the elements relevant to a failure, some basic questions need to be asked:

(i) Does the analytical model represent the likely actual mechanical behaviour of the construction in the ground?

(ii) How were the design parameters derived, and are they representative of the physical behaviour of the structure and the ground?

(iii) Have design parameters been used that are appropriate to the particular situation? Short- and long-term parameters, small and high strain parameters: were they appropriate to the structural mode in compression, tension, shear and bending?

Finally, does the end result of the analysis appear reasonable? The use of simplistic hand calculations to check the outcome is invaluable – it is too easy for the effect of a slipped decimal place to become lost in a complex numerical model analysis.

At the same time as considering the design, construction and workmanship must also be investigated. The construction records must be checked to confirm that the correct sequence was followed. Webcams that continuously monitor and record site activities have been found to be invaluable when checking against contractor's records.

Test results and monitoring records need to be interrogated for clues regarding the possible causes of failure.

Observations can take many forms, from anecdotal evidence to measurements of deflections, monitored stresses and strains. Some of this evidence may prove to be correct and true, some may prove to be untrue, and for some it may not be possible to verify either way. As with ground investigation information, the task of forensic investigation is to attempt to identify what is reliable, and why this should be so. Conversely, it must also attempt to identify what is not reliable, and for what reasons.

Having considered all the available evidence, one or more hypotheses must be formulated and tested for consistency with the observed behaviour. It is important to consider that there may be more than one explanation for what has occurred. All possibilities should be considered, and rejected only when they have been thoroughly examined. It is never sufficient to consider just one explanation and look no further, even if that one explanation happens to be consistent with the observed behaviour.

93.7.3 Reporting

Forensic investigation usually has two functions: (i) to find a solution to a problem; (ii) to document the investigation, the findings and the conclusions. If there has been a significant problem resulting in a dispute, then the matter may be resolved directly under the project contract, or indirectly by such means as set out in the contract. Resolution may involve adjudication, mediation, arbitration or ultimately, litigation. In case the matter eventually has to be resolved by litigation, great care should be taken when reporting on a forensic investigation to ensure the accuracy and validity of all statements made.

If it is possible that a report may be used in litigation, then Civil Procedure Rules Part 35 (CPR35) *Experts and Assessors*, and the accompanying *Practice Direction*, include the requirements for expert evidence to be given in a written report. It is important to distinguish between the evidence considered (the input data), the assessment of the evidence and opinions expressed, and the conclusions reached, and present a report with clearly-defined sections dealing with all aspects required by CPR35. Thus the forensic report should be clearly divided into the following sections:

- evidence;
- assessment of the evidence;
- opinions and conclusions.

Most reports on a forensic investigation will be read by people who will be less qualified than the author. It is therefore helpful to minimise the use of technical terminology and to add explanations where necessary. Judgements may be made based on the report, so it has to be easily understood by the reader. A style that works well is to minimise the content of the main body of the report so that it is easy to read and absorb, and to use appendices for reference information. Conclusions should be as succinct as possible.

A one page summary is a useful inclusion in a report on a forensic investigation, both for the author and the reader. For the author it ensures that the essence of the problem and the conclusions are presented logically, and for the reader it provides an easily absorbable overview.

93.8 Conclusions

QA is the management system that will ensure that all key parts of the geotechnical design and construction process are

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undertaken consistently in order to provide at least a minimum standard. A designer will use QC systems to ensure that the design and checking process is undertaken to meet the QA requirements. The contractor will use QC systems to ensure that the construction is in accordance with the specification and design.

Dependent upon the contractual arrangements on any particular project, the contractor will either have his workmanship verified by an external party such as an RE, or will be required to self-certify his own work.

Identifying and rectifying non-conformant work is a key part of the QA process. For geotechnical work it is paramount that a thorough forensic investigation is carried out at the earliest opportunity so that these issues are closed out early on in the construction process.

Disclaimer

The example non-conformances included in this chapter are for illustrative purposes only and are not associated with, or reflective of, the authors or their employers.

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93.9.1 Useful websites

Quality Scheme for Ready Mixed Concrete (QSRMC); www.qsrmc.co.uk

It is recommended this chapter is read in conjunction with

- Section 8 *Construction processes*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 94

Principles of geotechnical monitoring

John Dunnicliff Geotechnical Instrumentation Consultant, Devon, UK
W. Allen Marr Geocomp Corporation, Acton, MA, USA
Jamie Standing Imperial College London, UK

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Almost every construction project involving soil or rock runs some risk of encountering surprises, in particular 'unforeseen ground conditions'. Compared with other branches of civil engineering where there is greater control over the materials used, monitoring is vital to the practice of geotechnical design and construction. For this reason geotechnical engineers, unlike their colleagues in many other fields, must have more than a casual knowledge of geotechnical monitoring and instrumentation: it is an essential working tool.

The benefits of geotechnical monitoring are outlined in this chapter. These are followed by a systematic approach to planning monitoring programmes using geotechnical instrumentation, an example of systematic planning and general guidelines on the execution of monitoring programmes.

94.1 Introduction

Every geotechnical design has to cater for some uncertainty and every construction project involving soil or rock runs some risk of encountering surprises. These circumstances are the inevitable result of working with materials that are a product of nature, created by processes that seldom result in uniform conditions. The limitation of ground investigation procedures to detect all significant properties and conditions of natural materials in advance, requires the designer to make assumptions that may not reflect the overall ground conditions. The construction constructor may choose equipment and construction procedures without full knowledge of what might be encountered. Despite the limitations of not being able to explore every point within a site, geotechnical monitoring, including quantitative measurements obtained by field instrumentation, provides the means by which the geotechnical engineer can design a project to be safe and efficient, and the constructor can execute the work with safety and economy. Thus, geotechnical monitoring is vital to the practice of geotechnical design and construction. For this reason, geotechnical engineers frequently need a comprehensive knowledge of geotechnical monitoring and instrumentation.

Geotechnical monitoring is not merely the selection of instruments but a comprehensive step-by-step engineering process, beginning with a definition of the objective(s) and ending with implementation of contingency measures if the data warrant. Each step is critical to the success (or failure) of the monitoring programme, and the monitoring process involves combining the capabilities of instruments and people.

Peck (1984) states 'The legitimate uses of instrumentation are so many, and the questions that instruments and observation can answer so vital, that we should not risk discrediting their value by using them improperly or unnecessarily.'

Clauses 4.5, 11.7, 12.7 and Annex J in Eurocode 7, Part 1 (EN 1997–1:2004) provide guidelines on geotechnical monitoring.

The benefits of geotechnical monitoring are outlined in the following sections. These are followed by a systematic approach to planning monitoring programmes using geotechnical instrumentation, an example of systematic planning, and general guidelines on the execution of monitoring programmes.

Instruments that are used for geotechnical monitoring are described in Chapter 95 *Types of geotechnical instrumentation and their usage.*

Primary sources of text for this chapter have been Dunnicliff (1988, 1993), Dunnicliff (1998) and, with permission from the American Society of Civil Engineers (ASCE), Marr (2007).

94.2 Benefits of geotechnical monitoring

The following are the principal technical reasons for recommending a geotechnical monitoring programme for a project:

1. Minimising damage to adjacent structures.
2. Implementing the observational method.
3. Revealing unknowns.
4. Assessing a contractor's construction methods.
5. Devising remedial measures to address problems.
6. Improving performance.
7. Documenting performance for assessment of damages.
8. Showing that everything is satisfactory and as expected.
9. Warning of impending failure.
10. Advancing the state-of-knowledge.

Each of these is discussed next in the context of today's practice of geotechnical engineering. In general, a common feature of these technical reasons is that monitoring programmes save money.

94.2.1 Minimising damage to adjacent structures

Geotechnical construction may affect adjacent property with undesirable results, such as expensive repairs, bad relations and protracted litigation. Geotechnical monitoring can be used to warn of unacceptable movements of adjacent property. By doing this, the costs required to repair the damages are likely to be avoided. **Figure 94.1** shows a motorised total station for monitoring any movement of structures whilst tunneling beneath a road.

94.2.2 Implementing the observational method

The complete application of the observational method (see also Chapter 100 *Observational method*) embodies the following steps (Peck, 1969):

1. Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail.
2. Assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions.
3. Establishment of the design based on a working hypothesis of behaviour anticipated under the most probable conditions.
4. Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis.
5. Calculation of values of the same quantities under the most unfavourable conditions compatible with the available data concerning the sub-surface conditions.
6. Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis.
7. Measurement of quantities to be observed and evaluation of actual conditions.
8. Modification of design to suit actual conditions.



Figure 94.1 Motorised total station for monitoring any movement of structures

Photograph courtesy of SolData; all rights reserved

The degree to which all these steps can be followed depends on the nature and complexity of the work, but it is clear that in all cases a geotechnical monitoring programme is essential.

For example, it is possible to build an embankment over a soft soil stratum by constructing it in stages. Placed all at once, the embankment might cause foundation failure. Placing the embankment in stages at suitable time intervals allows the soft soil to strengthen by consolidating between each stage. Instruments to measure settlements and pore water pressures can be used to determine when enough consolidation of the clay has occurred, so that the next stage of fill can safely be added. A balance is usually sought between adding the next stage as quickly as possible to minimise construction time – but not so fast as to cause foundation failure. This example is described in more detail in section 94.4.

Section 2.7 in EN 1997–1 (2004) indicates that it is appropriate to apply the observational method when prediction of geotechnical behaviour is difficult. It includes: requirements to be met before construction is started; requirements to be met during construction; assessment of the results of monitoring; replacement of monitoring equipment.

94.2.3 Revealing unknowns

Using procedures which reveal unknown conditions and engage remedial work as early as possible will lead to the lowest project cost. A good geotechnical monitoring programme is vital to this approach. The alternative of delay, denial and blame almost always costs more.

94.2.4 Assessing a contractor's construction methods

The outcome of some geotechnical projects depends on the construction methods used and the ability of the construction contractor. Geotechnical monitoring is used to determine whether these meet the specified performance requirements.

94.2.5 Devising remedial methods to address problems

When problems occur in geotechnical construction, the cause must be identified as far as possible, and remedied. Finding the best remedy requires an understanding of what went wrong. Data from geotechnical monitoring can help engineers determine what caused the problem. A remedial action can then be devised that focuses on the specific cause rather than addressing several potential causes in the light of uncertainty. **Figure 94.2** shows an unstable slope that is being monitored with sub-surface



Figure 94.2 Instruments in an unstable slope

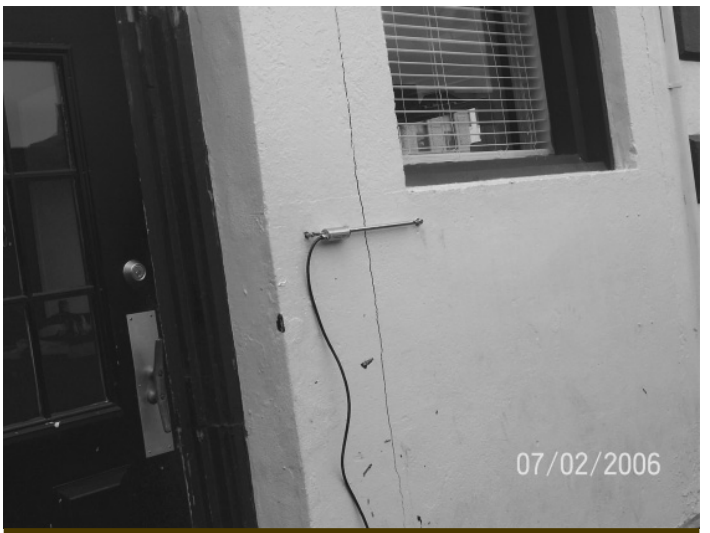


Figure 94.3 Vibrating wire crack gauge on the side of a building

deformation gauges to determine the depth of the failure surface so that an appropriate remedial action can be implemented.

94.2.6 Improving performance

Modern concepts of business management stress continual improvement and the need for measurements to gauge success. A common saying in business practice is 'that which is measured improves, while things not measured eventually fail'. The mere process of measuring performance coupled with normal human behaviour leads to improved performance.

94.2.7 Documenting performance for assessment of damages

Damage claims by third parties represent one of the substantial risks encountered in geotechnical projects. Claims sometimes include charges for damage unrelated to the construction in question. Others may be inflated, such as a claim for structural damage when only minor architectural damage has occurred.

Data from geotechnical monitoring can help establish the validity of such claims, thereby satisfying stakeholders and regulators, and reducing litigation. For example, if instrumentation shows that an adjacent building has not moved during construction, it becomes more difficult for the owner to claim that cracks in the building resulted from the construction activity. **Figure 94.3** shows a vibrating wire crack gauge on the side of a building to determine whether nearby construction is causing changes in crack width.

94.2.8 Showing that everything is satisfactory and as expected

Increasingly we use geotechnical monitoring programmes to demonstrate that the actual performance is within the bounds anticipated by the designers. The presumption is that there will be no surprises or unexpected consequences to the overall cost and schedule, and that unexpected behaviour can be identified early enough to maintain control of the project cost and schedule.

Data from a geotechnical monitoring programme help maintain the various parties' confidence in the performance of the work, thus allowing them to focus on other issues. Increasingly, project owners desire performance monitoring systems that are comprehensive and robust, but with instant reporting as simple as a green light to indicate that everything is progressing acceptably.

Figure 94.4 shows a tied-back retaining wall instrumented with load cells on some of the tiebacks to monitor any changes in load.

94.2.9 Warning of impending failure

Geotechnical structures can fail with catastrophic consequences to life and property. Such failures may be the result of excessive loads, design errors, construction deficiencies, unknown or unexpected conditions, deterioration, operational errors or intentional action. Geotechnical monitoring has been widely used to detect the onset of failure in dams, slopes, embankments and excavations. **Figure 94.5** shows vibrating wire strain gauges mounted on a pipe strut across an internally braced excavation to warn of excessive load in the strut. In practice, monitoring of a single strut in this way is insufficient – a significant number of struts need to be monitored to provide representative data.

94.2.10 Advancing the state-of-knowledge

Many of the advances in understanding ground and structural response in geotechnical engineering have their roots in data from geotechnical monitoring of full-scale projects. The data give us insight into how the ground/structure is responding to construction and about causal relationships. A significant amount of geotechnical instrumentation is currently used and has been used historically as part of a research effort to improve our state-of-knowledge. Much of this has been paid for by governmental agencies with a mission to improve practice. **Figure 94.6** shows a long-stroke vibrating wire strain



Figure 94.4 Load cells on tie-back anchors



Figure 94.5 Vibrating wire strain gauges mounted on a pipe strut across an internally braced excavation

gauge mounted on a geotextile for advancing the state-of-knowledge about geotextile performance.

94.2.11 An overview of the above ten benefits of geotechnical monitoring

Ten principal technical reasons (or benefits) for recommending a geotechnical monitoring programme for a project have been identified and discussed in the context of today's practice of geotechnical engineering. As stated earlier, a common feature of these technical reasons is that monitoring programmes generally save money. It is probable that the first reason, minimising damage to adjacent structures, is the most common of these. However, it is strongly recommended that designers of construction projects should study the other nine potential reasons in the context of the project specifics, and endeavour to persuade their clients to adopt geotechnical monitoring if it can be shown that there is a clear technical or financial benefit.

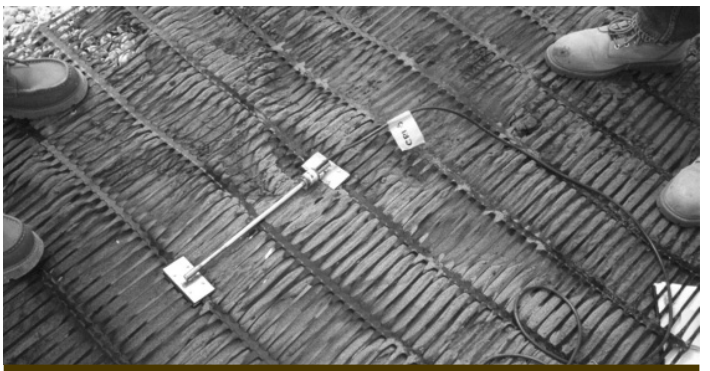


Figure 94.6 Long-stroke vibrating wire strain gauge mounted on a geotextile

94.3 Systematic approach to planning monitoring programmes using geotechnical instrumentation

Planning a monitoring programme using geotechnical instrumentation should begin with defining the objective and end with planning how the measurement data will be implemented. Planning should proceed through the steps listed in **Table 94.1** and outlined below. More detail is given in Dunnicliff (1988, 1993). All of these planning steps should, if possible, be completed before geotechnical monitoring work commences in the field.

94.3.1 Step 1. Define the project conditions

The person responsible for planning a monitoring programme must become very familiar with the project type and layout, sub-surface stratigraphy and engineering properties of sub-surface materials, groundwater conditions, status of nearby structures or other facilities, environmental conditions and the planned construction method.

94.3.2 Step 2. Predict mechanisms that control behaviour

Prior to developing a monitoring programme, one or more working hypotheses must be developed for mechanisms that are likely to control behaviour.

94.3.3 Step 3. Define the geotechnical questions that need to be answered

Every instrument on a project should be selected and placed to assist in answering a specific question: if there is no

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Step	Task
1	Define the project conditions
2	Predict mechanisms that control behaviour
3	Define the geotechnical questions that need to be answered
4	Identify, analyse, allocate and plan for control of risks
5	Select the parameters to be monitored
6	Predict magnitudes of change
7	Devise remedial action
8	Assign tasks for the construction phase
9	Select instruments
10	Select instrument locations
11	Plan documentation of factors that may influence measured data
12	Establish procedures for ensuring data correctness
13	List the specific purpose of each instrument
14	Prepare budget
15	Prepare instrumentation system design report
16	Plan installation
17	Plan regular calibration and maintenance
18	Plan data collection and data management
19	Prepare contract documents
20	Update budget

Table 94.1 Steps in systematic approach to planning monitoring programmes using geotechnical instrumentation

question, there should be no instrumentation. This is the first golden rule.

94.3.4 Step 4. Identify, analyse, allocate and plan for control of risks

All risks associated with construction should be identified, and each 'geotechnical question' should be prioritised based on risk. Responsibility for each risk may be allocated to a single party or to more than one party. Risk responsibility allocation should be included in the construction contract documents.

Risk analysis embodies a wide range of scientific theories and engineering analyses to identify potential sources of risk, determine the probability of occurrence for each source, and estimate the consequences from each source of risk. Total risk is the summation of the probability of each source of risk occurring multiplied by the consequences of that occurrence. Risk can be decreased by actions that reduce the probability of a source of risk or its adverse consequences occurring.

Van Staveren (2006) provides comprehensive guidance on identification, analysis, allocation and planning for control of risks, and Marr (2007) provides an approximate method for quantifying the benefits that have been described above, based on concepts of decision theory and risk analysis. Risks are also discussed in Chapter 7 *Geotechnical risks and their context for the whole project.*

94.3.5 Step 5. Select the parameters to be monitored

Typical geotechnical parameters include:

- pore water pressure;
- deformation;
- tilt;
- total stress;
- load and strain in structural members;
- temperature.

The question: 'which parameters are most relevant?' should be answered.

Clauses 4.4, 11.7, 12.7 and Annex J in EN 1997–1 (2004) includes guidance on selecting the parameters to be monitored.

94.3.6 Step 6. Predict magnitudes of change

Manufacturers of geotechnical instruments will define their range and accuracy. In order that appropriate instruments are selected, there must be a prediction of the maximum change that might occur, hence determining the needed range. For the same reason, the needed accuracy must also be determined.

If measurements are for construction control or safety purposes, a predetermination should be made of numerical values that indicate the need for decisive mitigation measures. These values are often referred to as trigger levels. The concept of green, amber and red trigger levels is useful:

- green indicates that all is well;
- amber indicates the need for cautionary measures including an increase in monitoring frequency;
- red indicates the need for timely remedial actions and being prepared to implement them quickly.

The following are guidelines for selection of trigger levels:

- Early trigger levels can be based on calculated changes, whereas later levels can be based on (unrelated) tolerable changes.
- Trigger levels must recognise the changes that occur from causes other than construction.
- Trigger levels should be several times larger than the accuracy of measured changes (those last four words are very carefully chosen).

94.3.7 Step 7. Devise remedial action

Inherent in the use of instrumentation for construction purposes is the absolute necessity for deciding, in advance, a positive means for dealing with any problem that may be disclosed by the results of the observations. If the observations should demonstrate that remedial action is needed, that action must be based on appropriate, previously anticipated plans. Arrangements should be made to determine how all parties will be forewarned of the planned remedial actions.

94.3.8 Step 8. Assign tasks for the construction phase

The second golden rule (remember that the first golden rule has been given in Step 3 above) is: *Tasks should be assigned to the people who have the greatest motivation to achieve high quality data.* Many geotechnical monitoring programmes have been unsuccessful because planners of the programmes have assigned key tasks to people who have inadequate motivation. Hence this step is of the utmost importance, and is covered in some detail in this section.

The tasks include:

1. buying instruments;
2. installing instruments;
3. collecting data;
4. interpreting data;
5. implementing actions resulting from the data;

and it is crucial to ensure that these tasks are assigned to the people who are most likely to maximise quality.

Clearly data interpretation should be the responsibility of the people who initiated the monitoring programme. Implementation should be by construction personnel.

If principal construction contractors, temporary works contractors, specialist geotechnical sub-contractors or design/build contractors have initiated the monitoring programme, clearly they have the greatest motivation and tasks 1, 2 and 3 should be assigned to them. However, if the programme has been initiated by the designer of the project, there are four reasons for not assigning tasks 1, 2 and 3 to the principal construction contractor:

A. Principal construction contractors may not have enough motivation to ensure quality. Use of the conventional lowest tender procedure, whereby these tasks are included as items in the principal construction contactor's contract, has often led to poor quality data.

B. Monitoring cannot start until after the award of the principal construction contract, hence adequate pre-construction (baseline) data are usually unavailable. Structures move and groundwater regimes often change from season to season, and geotechnical monitoring data cannot be interpreted correctly if baseline data are not established. Whenever practicable, it is highly preferable to establish at least one year of baseline data.

C. It costs the project owner more. Potential monitoring sub-contractors give prices to principal construction contractors prior to the latter tendering. After contract award, there is normally a bargaining process, and the project owner pays £1 for work that costs the principal contractor 85 or 90 pence.

D. For multi-principal contract projects, there would be one monitoring sub-contractor for each principal construction contract.

The following assignments are recommended for tasks 1, 2 and 3:

- *Pre-construction baseline data*: specialist firm under contract with the project owner, using a qualifications-based selection procedure.
- *During construction, outside principal construction contractor's work area*: same as above.
- *During construction, within principal construction contractor's work area*
  - owner's representative, with assistance from principal construction contractor for access as necessary; or
  - specialist firm, as a nominated (assigned) specialist sub-contractor, selected using a qualifications-based procedure, with instruments selected on the basis of proven performance; or
  - principal construction contractor, with partnering and rigorous and *enforced* specifications.

For further information on assignment of tasks and for details of the above recommendations, see Dail and Volterra (2009), Dunnicliff (2009), Dunnicliff and Powderham (2001a, 2001b) and Klingler (2001).

Additional information on writing specifications is given in Chapter 78 *Procurement and specification*.

94.3.9 Step 9. Select instruments

The preceding eight steps should be completed before instruments are selected. When selecting instruments, the overriding desirable feature is *reliability*. Selection of instruments and readout equipment depends directly on the methods used for data collection – see section 94.5.5 for general guidelines. The scale of the geotechnical monitoring programme should match the identified risks and the scale and complexity of the geotechnical questions.

Details of some instruments, together with their applications, are given in Chapter 95 *Types of geotechnical instrumentation and their usage.*

The lowest cost of an instrument should never be allowed to dominate the selection. The least expensive instrument is not likely to result in minimising project cost. In evaluating the economics of alternative instruments, the overall cost of procuring, calibration, installation, maintenance, monitoring and data processing should be compared.

94.3.10 Step 10. Select instrument locations

A practical approach to selecting instrument locations entails three steps:

1. Establish the zones where the risk is highest. Instrument these thoroughly.
2. Zones are identified (normally cross-sections) where predicted behaviour is considered representative of behaviour as a whole. These cross-sections are regarded as *primary instrumented sections*, and instruments are located to

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provide comprehensive information on ground/structural responses and construction performance.

3. Because unknown factors may cause poorer performance at other locations, instrumentation should be installed at a number of *secondary instrumented sections*, to serve as indices of comparative behaviour. Instruments at these secondary sections should be as simple as the requirements allow and be installed at the primary sections so that comparisons can be made. If, in fact, the behaviour at a secondary section appears to be significantly different from the behaviour at the primary sections, additional instrumentation may be installed at the secondary section as construction progresses.

When selecting locations it should be recognised that some instruments will probably cease to function when data are still required, hence some duplication may be needed. Some limited duplication of measurements leads to increased confidence in data, especially in areas of uncertainty or greatest change.

94.3.11 Step 11. Plan documentation of factors that may influence measured data

Measurements by themselves are rarely sufficient to provide useful conclusions. Geotechnical monitoring normally involves relating measurements to causes, and therefore records and diaries must be maintained of all factors that might cause changes in the measured parameters, including construction details and progress.

94.3.12 Step 12. Establish procedures for ensuring data correctness

Personnel responsible for geotechnical monitoring must be able to answer the question: *Are the data realistic?* The ability to answer depends on availability of good evidence, for which planning is required. In critical situations, duplicate instruments can be used. A backup system is often useful and will frequently provide an answer even when its accuracy is significantly less than that of the primary system.

94.3.13 Step 13. List the specific purpose of each instrument

At this point in the planning, it is wise to question whether all planned instruments are justified. Each planned instrument should be numbered and its purpose listed. If no viable specific purpose can be found for a planned instrument, it should be deleted.

94.3.14 Step 14. Prepare budget

Even though the planning steps are not complete, a budget should be prepared at this stage for all tasks that are included in section 94.5 to ensure that sufficient funds are available. A frequent error in budget preparation is to underestimate the duration of the project and the total costs for data collection and interpretation. If insufficient funds are available, the monitoring programme may have to be curtailed or more funds obtained.

94.3.15 Step 15. Prepare instrumentation system design report

This report should summarise the results of planning steps 1–14. It should then be reviewed by others to ensure that everything is consistent, that the plan is a good one and that it covers the needs of the project.

94.3.16 Step 16. Plan installation

Installation procedures should be planned well in advance of scheduled installation dates. Written step-by-step procedures should be prepared, making use of the manufacturer's instruction manual and the designer's knowledge of specific site geotechnical conditions. The written procedures should include a detailed listing of required materials and tools, and installation record sheets should be prepared for documenting factors that may influence measured data. Installation plans should be coordinated with the construction contractor and arrangements made for access and protection from damage of installed instruments. An installation schedule should be prepared, consistent with the construction schedule.

94.3.17 Step 17. Plan regular calibration and maintenance

Calibration and maintenance should be planned well before field work starts; calibrations are described in section 94.5.2. Maintenance planning should include readout units, field terminals and embedded components.

94.3.18 Step 18. Plan data collection and data management

Written procedures for data collection and data management should be prepared well before field work starts. These are significant tasks, and the effort required for them should not be underestimated. Guidelines are given by Dunnicliff (1988, 1993) and in section 94.5.5.

At this stage in the planning, a verification should be made to ensure that field staff are appropriately trained, remedial actions have been planned, personnel responsible for interpretation of monitoring data have the contractual authority to signal remedial action, communication channels between design and construction personnel are open, and that arrangements have been made to forewarn all parties of the planned remedial actions.

94.3.19 Step 19. Prepare contract documents

Any necessary contract documents should be prepared, to match the decisions made during step 8.

94.3.20 Step 20. Update budget

With planning complete, the budget for all tasks listed in planning step 8 should be updated in light of all planning steps.

94.3.21 An overview of the most common failures in implementing a systematic approach to planning monitoring programmes

All the above 20 steps are important. In the experience of the authors, the following steps are the most crucial:

- Step 3. Define the geotechnical questions that need to be answered. As indicated above, every instrument on a project should be selected and placed to assist in answering a specific question: if there is no question, there should be no instrumentation. This is the first golden rule, and is commonly ignored.
- Step 4. Identify, analyse, allocate and plan for control of risks. When a risk analysis is made, it is often not comprehensive enough, and risk responsibility allocation is commonly not adequately covered in the construction contract documents.
- Step 7. Devise remedial action. All too often instrumentation programmes are planned without ensuring that viable remedial actions are available in the event that trigger levels are reached.
- Step 8. Assign tasks for the construction phase. As indicated above, the second golden rule is that tasks should be assigned to the people who have the greatest motivation to achieve high quality data. Many geotechnical monitoring programmes have been unsuccessful because planners of the programmes have assigned key tasks to people who have inadequate motivation.

Singling out these four steps in this overview in no way implies that the other 16 steps are unimportant.

94.4 Example of a systematic approach to planning a monitoring programme: using geotechnical instrumentation for an embankment on soft ground

In this example, the geotechnical questions are associated only with the soft ground below the embankment, and not with the embankment itself. Only selected planning steps are included in the example (i.e. not all the 20 steps that have been detailed in the previous section). Ladd (1991) presents valuable guidelines for geotechnical monitoring of embankments on soft ground.

Examples of steps 3, 5 and 9 for ten additional project types are given in Chapter 95 *Types of geotechnical instrumentation and their usage*. These additional project types are:

- internally braced excavations;
- externally braced excavations;
- clay embankments;
- cut slopes in soil;
- landslides in soil;
- cut slopes in rock;
- landslides in rock;
- tunnels;
- driven piles;
- bored piles.

94.4.1 Step 1. Define the project conditions

- **Figure 94.7** illustrates the project conditions for this example of planning.
- The embankment is wide compared with the thickness of the very soft clay layer.
- Staged construction and vertical drains are planned so that the roadway on top of the embankment can be completed as soon as possible.
- **Figure 94.8** shows the installation of vertical drains through very soft clay.
- There are some silt layers in very soft clay.
- Initial groundwater conditions are hydrostatic.
- There are no nearby structures.
- The predicted total vertical compression of the clay is 15% strain.

94.4.2 Step 2. Predict mechanisms that control behaviour

- Settlement (consolidation of very soft clay).

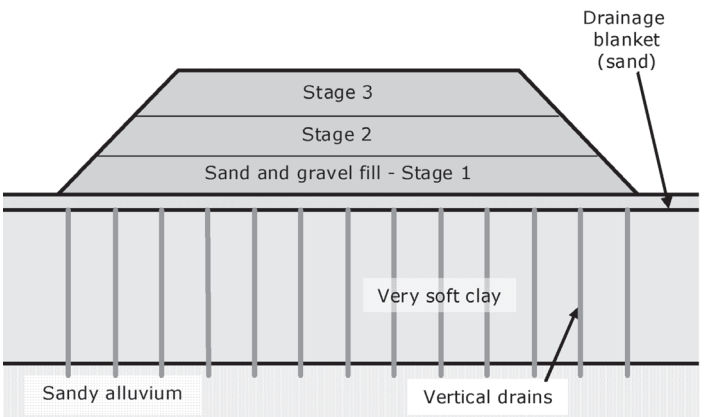


Figure 94.7 Project conditions for an embankment on soft ground



Figure 94.8 Installation of vertical drains through soft clay

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- Lateral bulging displacements of the clay, causing heave either side of the embankment.
- A rotational slide, causing failure.

94.4.3 Step 3. Define the geotechnical questions that need to be answered

(See Chapter 95 *Types of geotechnical instrumentation and their usage*.)

- What are the initial site conditions in the soft ground?
- Is the embankment stable in the short term?
- So that decisions can be made as to when to place the next stage of fill, what is the progress of consolidation and associated gain of strength of the soft ground?

94.4.4 Step 4. Identify, analyse, allocate and plan for control of risks

- Risks are that (i) an embankment failure may occur and (ii) the completion of the roadway is delayed.
- The project owner's construction manager will be on site to make decisions about placing subsequent stages of fill, and the project owner accepts responsibility for risks (i) and (ii) above.
- Strength gain will occur more slowly than predicted, thereby slowing construction, delaying completion and creating a possibility for the construction contractor to submit a claim for extra money.

94.4.5 Step 5. Select the parameters to be monitored

(See Table 95.3.)

- Pore water pressure in the clay.
- Vertical deformation.
- Horizontal deformation.

94.4.6 Step 6. Predict magnitudes of change

- Pore water pressure in the clay:
  - range – initial pressure plus additional pressure caused by the weight of the embankment;
  - accuracy – consider the time plot that shows dissipation of pore water pressure under a single stage of fill, and the need to extrapolate from that to decide when the next stage can be placed.
- Vertical deformation:
  - range – as the predicted vertical compression is given as 15%, surface settlement can be calculated;
  - accuracy – same logic as for pore water pressure.
- Horizontal deformation:
  - range – base this on an estimate of the maximum deformation that could occur;
  - accuracy – no general recommendation can be given, as required accuracy depends on project details.
- Trigger levels need to be determined.

94.4.7 Step 7. Devise remedial action

- Remove fill.
- Place a berm at the toe of the embankment.

94.4.8 Step 8. Assign tasks for the construction phase

- The monitoring programme has been initiated by the project designers.
- Responsibility for pre-construction baseline data is assigned to a specialist firm under contract with the project owner, selected using a qualifications-based procedure.
- During construction, the tasks of buying and installing instruments and for collecting data are assigned to a nominated specialist sub-contractor, selected using a qualifications-based procedure. Instruments are selected on the basis of proven performance.
- The project designers are responsible for interpreting monitoring data.
- The construction contractor is responsible for implementing actions resulting from the data.

94.4.9 Step 9. Select instruments

(See Table 95.3.)

- Pore water pressure in the clay:
  - vibrating wire piezometers.
- Vertical deformation:
  - surface monuments with surveying;
  - probe extensometers – probably magnet/reed switch type.
- Horizontal deformation:
  - surface monuments with surveying;
  - inclinometers.

94.4.10 Step 10. Select instrument locations

- This would depend on the length of the new roadway.
- Perhaps a minimum of three primary instrumented sections and three times as many secondary instrumented sections.
- Within each of these sections decisions would have to be made regarding the position of instruments (e.g. piezometers midway between vertical drains.

94.4.11 Step 11. Plan recording of factors that may influence measured data

- Fill level.
- Weather, e.g. amount of precipitation, sun, wind.
- Temperature.
- Barometric pressure (if vibrating wire piezometers have been selected).
- Visual observations of unexpected or unusual behaviour.

94.4.12 Step 12. Establish procedures for ensuring data correctness

- Visual observations.
- Consistency among instruments for monitoring the same parameter.
- Consistency between pore pressure dissipation data and vertical compression data.
- Consistency of reaction of instruments to a new lift of fill.
- Study of repeatability, over both short and longer term.

- Measured pore water pressures, under no load change, should extrapolate to initial pressures.
- Regular calibrations of readout units and survey equipment.

94.5 General guidelines on execution of monitoring programmes

94.5.1 Introduction

This section includes general guidelines for the following tasks:

- calibration;
- installation;
- maintenance;
- data collection;
- processing and presentation of monitoring data;
- interpretation of monitoring data;
- reporting of conclusions;
- implementation.

Additional guidelines are given by Dunnicliff (1988, 1993).

94.5.2 Calibration

An instrument reading is only useful if the correct calibration is applied. Changes in calibration factors occur due to wear and tear, misuse, creep, moisture ingress and corrosion. If these changes are not accounted for, the entire monitoring programme can become worthless or lead to wrong conclusions and costly recommendations. To maximise their effectiveness, all instruments must be calibrated and checked properly.

Instrument calibrations and/or function checks are generally required at the following stages:

- Calibrations prior to shipment. These are the responsibility of the manufacturer, and are referred to as *factory calibrations*. Calibration certificates should be provided to the user. **Figure 94.9** shows factory calibrations of transducers for vibrating wire piezometers.
- Function checks by the user before installation. These are referred to as *pre-installation acceptance tests*, their purpose being to verify that no damage has occurred during transit. Manufacturers will provide guidelines on what checks should be made.
- Function checks after installation. These are referred to as *post-installation acceptance tests*. Try to apply/identify some known change to test whether the instrument continues to function as expected.
- Calibrations during service life. Calibrations or function checks of instruments and readout units are required during service life. These calibrations are usually performed by personnel responsible for data collection.

94.5.3 Installation

Each instrument must be installed before its planned finish date. The schedule will be based on the need to establish pre-construction (baseline) conditions, construction activity, availability of installation personnel, instrument delivery dates and other relevant factors.



Figure 94.9 Factory calibration of transducers for vibrating wire piezometers
Courtesy of Geokon Inc., Lebanon, New Hampshire, USA

Figures 94.10–94.12 show installations of a borehole extensometer, inclinometer casing, and electrical resistance strain gauges, respectively.

Surface exposures of instruments should usually be protected with robust cover plates, welded, bolted or otherwise attached to the surface. Exposed tubes and cables are extremely vulnerable to damage and should usually be protected by conduit or armouring. **Figure 94.13** shows precast concrete pipe segments used as surface protection for borehole instruments.

The presence of a measuring instrument should cause minimum alteration to the value of the parameter being measured. If the instrument alters the value significantly, it is said to have *poor conformance*.

As indicated in step 17, installation record sheets should be prepared during the planning phase and completed during the installation. These records serve two purposes. Firstly, when installation personnel are required to enter data/information in the blank spaces on a field form, they are more likely to follow the installation procedure with care. Secondly, 'as-built' data are required both for record purposes and for use during evaluation of data. These records form an important part of any quality assurance programme.

On completion of installation, an installation report should be produced to provide a complete compilation of information needed by personnel responsible for data collection, processing, presentation and interpretation. The installation report should contain as-built information, descriptions of instruments and readout units, results for calibrations and checks, details of installation procedures, initial readings and a copy of each installation record sheet.

94.5.4 Maintenance

Regular maintenance required during service life is usually performed by personnel responsible for data collection.

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Figure 94.10 Installation of borehole extensometer above future tunnel



Figure 94.11 Installation of inclinometer casing

They should always be on the lookout for damage, potential for damage, and deterioration or malfunction.

94.5.5 Data collection

There are three primary means of collecting data:

- Manual reading involves people going to the instrument readout location and using some device to read the instrument. The reading is manually recorded on paper or perhaps in an electronic device. This approach may be the most cost effective when readings are required less than once per week and the distance to the instrument is not too far, and the readout location can be safely reached. Instruments for manual readings do not have to be electronic and may be of low cost and easy to maintain. **Figure 94.14** shows the manual reading of piezometric level in an open standpipe piezometer.
- Data loggers with periodic manual data download may be used when the reading interval is less than one week or it is difficult to get to the instrument location and there is no immediate need to know the readings. Readings are stored in the memory of the data logger, where they are periodically downloaded manually to a portable electronic device for transport back to the office. Someone travels



Figure 94.12 Installation of electrical resistance strain gauges on segment of cast iron tunnel lining

to the data logger once per week or month or quarter to collect data recorded since the last visit. This approach requires the use of electronic instruments that are read by the data logger. Hardware for this approach may cost more than for the manual reading approach but the cost of collecting data may be less, and more frequent readings can be obtained for essentially no added cost.

- Data loggers with remote data access are used when there is an immediate need for the instrument readings and when reading frequency is more than once per day. In this approach, the data logger is connected to a remote computer system by landline or mobile phone modem, satellite modem, radio or the internet. These systems allow users to see data within seconds of when it is recorded. They also allow alarm systems to be set up that can inform project staff whenever an instrument indicates a value that exceeds a pre-established trigger level. An example of such a data logger, with solar panels for battery charging, is shown in **Figure 94.15**.



Figure 94.13 Borehole instruments protected from damage by precast concrete pipe segments



Figure 94.14 Manual reading of piezometric level in an open standpipe piezometer

Each of these approaches has its advantages and limitations which directly influence how data are collected, reduced and communicated – hence the importance of selecting the data collection approach during the planning phases of the monitoring programme.

The frequency of data collection should be related to construction activity, to the rate at which the readings are changing, and to the requirements of data interpretation. Clause 4.5 in EN 1997–1 (2004) includes guidance on the duration of monitoring.

When construction commences and approaches the instrument location, readings should be taken frequently; for example, once a week, once a day, once a shift or even more frequently in relation to construction activity (such as before and after each blast, during pile driving, or during the placement or removal of a preload). It is often wise to increase the frequency



Figure 94.15 Data logger with remote data access for monitoring lateral movement of a lock wall

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of readings during heavy precipitation. As construction activity moves away from the instrument location or ceases altogether, and when readings have stabilised, the frequency may then be decreased. It is generally advisable to continue the monitoring effort until near completion of all construction activities because some effects of construction take time to fully develop and there may be activities by others that affect your project.

94.5.6 Processing and presentation of monitoring data

The first aim of data processing and presentation is to provide a rapid assessment of data in order to detect changes that require immediate action. The second aim is to summarise and present the data in order to show trends and to compare observed with predicted performance so that any necessary action can be taken.

After calculations have been made, graphs of data should always be prepared. Various types of graphs are described by Dunnicliff (1988, 1993):

- graphs to assist with data screening;
- graphs of data versus time. An example of this type of graph is shown in **Figure 94.16**;
- graphs for comparing observed and predicted behaviour;
- graphs for comparison of measurements with construction events. **Figure 94.16** includes construction events on the time axis;
- graphs to examine 'cause and effect' relationships. **Figure 94.17** is an example of this type of graph, showing load as 'cause' and displacement as 'effect' during a pile load test;
- summary graphs.

Increasingly project management staff want to know the status of the monitoring programme without being involved with the details, unless unacceptable values are imminent. A one-page summary report of important information is useful for this purpose.

94.5.7 Interpretation of monitoring data

Many monitoring programmes have failed because the collected data were never used. If there is a clear sense of purpose for a monitoring programme, the method of data interpretation will be guided by that sense of purpose.

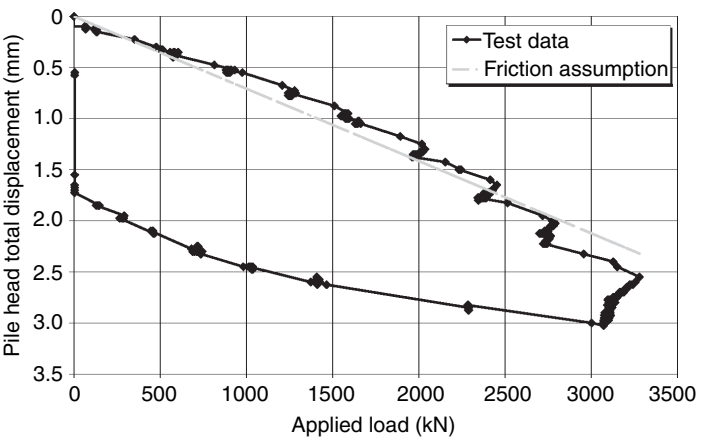


Figure 94.17 Graph of pile load test data

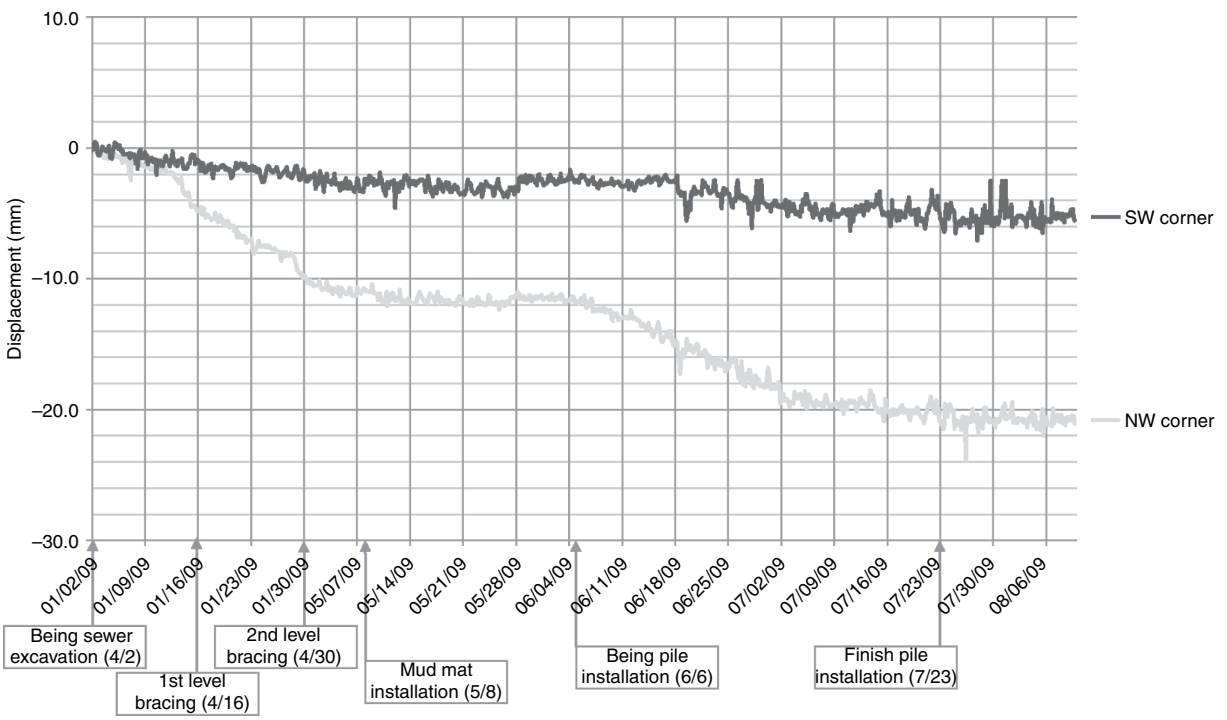


Figure 94.16 Graph of building settlement adjacent to excavation versus time

The first data interpretation step should be to provide a rapid assessment of data in order to detect changes that require immediate action. The essence of subsequent data interpretation steps is to correlate the instrument readings with other factors (cause and effect relationships) and to evaluate the deviation of the readings from the predicted behaviour.

Trigger levels (see section 94.3.6) should be watched very closely when interpreting monitoring data. Don't ignore changes during the green trigger level period by simply waiting for the green flag to change to amber. Trends during the green period can give useful forewarning.

Interpretation is an ongoing process. Interpretations should be made as frequently as change is expected; those from early data will usually be revised or refined as more data become available, and as a clearer understanding of real behaviour is developed.

An open communication channel should be maintained between design and construction personnel, so that discussions can be held between design engineers (who raised the questions that caused the instrumentation to be used), field engineers (who provide the data) and construction or operations personnel (who use the data). It is important to establish at the start of the project the channel of communications for transmitting reports and alarm messages.

Clause 4.5 in EN 1997–1 (2004) includes guidance on evaluation of monitoring data.

94.5.8 Reporting of conclusions

After each set of data has been interpreted, conclusions should be reported in the form of an interim monitoring report and submitted to personnel responsible for implementation of contingency measures. These interim reports may be daily, weekly or monthly, depending on the specific needs of the project.

94.5.9 Implementation

Implementation includes carrying out all the steps described above. For the geotechnical monitoring to be effective, there must be actions that implement the data in order to manage risk. These actions include:

- execute remedial actions where required;
- revise design as necessary;
- change contractor's construction methods;
- evaluate measured effects on remaining work;
- synthesise lessons learned and modify standard procedures where required.

94.6 Summary

Ten principal technical reasons for recommending a geotechnical monitoring programme for a project have been outlined. Each of these has been discussed in the context of today's practice of geotechnical engineering. In general, a common feature of these technical reasons is that monitoring programmes save money.

There is a tendency among some users of instrumentation to proceed in an illogical manner, often first selecting an instrument, making measurements and then wondering what to do with the resulting data. Planning a monitoring programme using geotechnical instrumentation should begin with defining the objective and end with planning how the measurement data will be implemented. Twenty steps in a systematic planning process have been outlined, and an example has been given.

Finally, guidance is given for maximising the quality of field tasks associated with monitoring programmes.

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94.7.1 Useful websites

Geotechnical Instrumentation News (GIN) articles; www.geotechnicalnews.com/instrumentation_news.php

Articles published since 2001 can be downloaded from this site. At the end of each year, that year's articles will be posted on the site.

Symposia on Field Measurements in GeoMechanics; www.fmgm.no

The *Publications* page contains a search function for authors and paper titles from the proceedings of ten international symposia on geotechnical monitoring.

www.asce.org as follows: Publications. Research Databases. ASCE Online Research Library. Papers presented at the American Society of Civil Engineers Symposium on Field Measurements in GeoMechanics (FMGM), Boston, 2007 can be accessed on this site.

It is recommended this chapter is read in conjunction with

- Chapter 78 *Procurement and specification*
- Chapter 79 *Sequencing of geotechnical works*
- Chapter 100 *Observational method*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 95

Types of geotechnical instrumentation and their usage

John Dunnicliff Geotechnical Instrumentation Consultant, Devon, UK

Instruments for the monitoring of groundwater pressure, deformation, load and strain in structural members and total stress are described, together with a brief explanation of how they work, schematic diagrams of selected instruments and other important issues. Applications for the instruments are included. The general role of instrumentation for various types of projects is followed by tabular listings of possible geotechnical questions that need to be answered and that may lead to the use of instrumentation, with summaries to indicate some of the types of instruments that can be considered for helping to provide answers to these geotechnical questions.

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95.1 Introduction

In the first part of this chapter, instruments for the monitoring of groundwater pressure, deformation, load and strain in structural members and total stress are described, together with a brief explanation of how they work, schematic diagrams of selected instruments and other important issues. Applications for the instruments are included. During the past ten years there has been an increasing trend towards the development and acceptance of instruments that allow for wireless data transmission, thereby reducing costs of data transmission and collection, and reducing the potential for damage to electrical cables.

In the second part of this chapter, the general role of instrumentation is given for various types of projects. This is followed by tabular listings of possible 'geotechnical questions that need to be answered' (see Chapter 94 *Principles of geotechnical monitoring*) that may lead to the use of instrumentation, with summaries to indicate some of the types of instruments that can be considered for helping to provide answers to these geotechnical questions.

When references are cited, priority is given to those that are available online. A significant part of the contents of this chapter is based on Dunnicliff (1988, 1993) and Dunnicliff (1998).

Associated Chapter 94 *Principles of geotechnical monitoring* includes the benefits of geotechnical monitoring, a systematic approach to planning monitoring programmes using geotechnical instrumentation, and general guidelines on the execution of monitoring programmes.

95.2 Instruments for monitoring groundwater pressure

95.2.1 Introduction

In this chapter the term 'piezometer' is used to indicate a device that is sealed within the ground so that it responds only to groundwater pressure around itself and not to groundwater pressures at other elevations. Piezometers are used to monitor positive (and sometimes negative) pore water pressure in soil and joint water pressure in rock. An observation well is a different type of instrument: it has no subsurface seals, and creates a vertical connection between strata.

Applications for piezometers fall into three general categories:

- First, for monitoring the pattern of water flow; second, to provide an index of soil or rock mass strength and third, for measuring groundwater pressure at an installed response zone elevation. Examples in the first category include monitoring subsurface water flow during large-scale pumping tests to determine permeability *in situ*, and monitoring the long-term seepage pattern in slopes.
- Monitoring pore water pressure (positive or negative) or joint water pressure allows an estimate of effective stress to be made and thus an assessment of strength. Examples include assessing the strength along a potential failure plane behind a cut slope in soil or rock, and monitoring of pore water pressure to control staged construction over soft clay foundations.
- When readings are taken from a number of different instruments with response zones at different elevations, a piezometric profile can be inferred. This is of significance for geotechnical design.

95.2.2 Observation wells

As shown in **Figure 95.1**, an observation well consists of a perforated section of pipe attached to a riser pipe, installed in a sand- or gravel-filled borehole. Typical diameters are: pipe 25–50 mm; borehole 75–250 mm. The surface seal, with cement mortar or other material, is needed to prevent surface run-off from entering the borehole, and a vent is required in the pipe cap so that water is free to flow through the wellpoint. The elevation of the water surface in the observation well is determined by sounding with an electrical probe attached to a graduated cable (sometimes an integral survey tape and cable), commonly referred to as a dipmeter. When the probe reaches the water surface in the riser pipe, an electrical circuit is completed and this is indicated by a light or buzzer in the read-out unit.

Appropriate applications for observation wells are very limited. In current practice they are frequently installed in boreholes during the site investigation phase of a project, ostensibly to define initial groundwater pressures and seasonal fluctuations. However, because observation wells create a vertical connection between strata, their *only* valid application is in continuously permeable ground in which groundwater pressure increases uniformly with depth. This condition can rarely be assumed. Here is a specific caution with respect to specifying observation wells with screens bridging between 'contaminated' and inert strata separated by relatively impermeable strata – this provides a preferential route for contaminant flow, hence is improper practice.

95.2.3 Open standpipe piezometers

The components are identical in principle to components of an observation well, with the addition of a subsurface seal, so that the instrument responds only to groundwater pressure around the filter element and not to groundwater pressures at other elevations. **Figure 95.2** shows an installation in a borehole. Typical diameters of standpipe and borehole are the same as for observation wells. The piezometer is sometimes referred to as a Casagrande piezometer (Casagrande, 1949). Traditionally, following Casagrande's installation procedure, a seal of bentonite chips or pellets was installed between the sand and the special grout. However, this is no longer necessary (Dunnicliff, 2008). The special grout (Mikkelsen, 2002; Contreras *et al.*, 2008) can be tremied directly onto the sand filter. To avoid

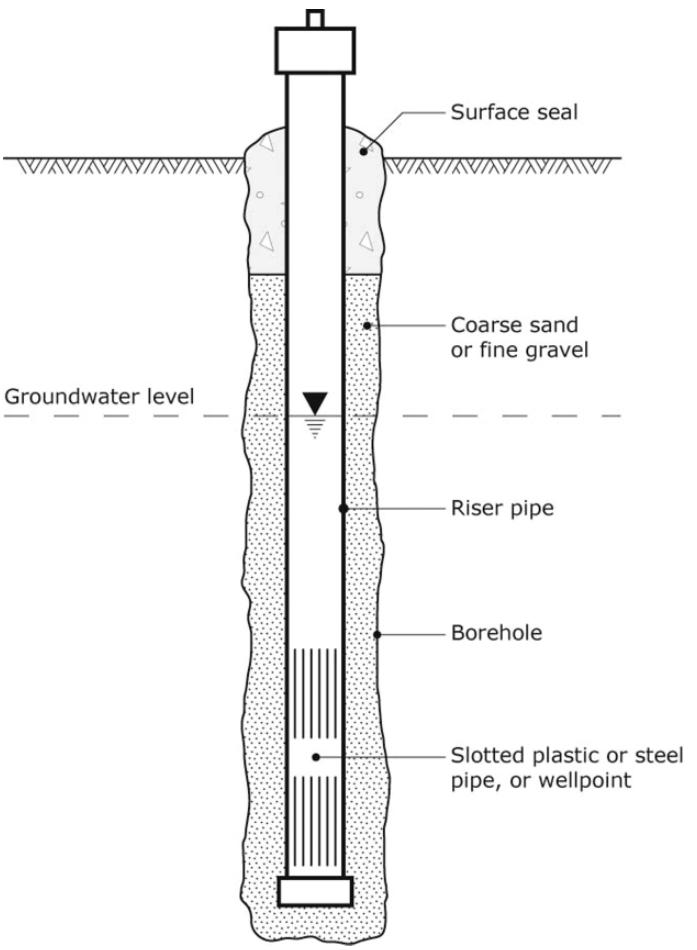


Figure 95.1 Schematic of observation well

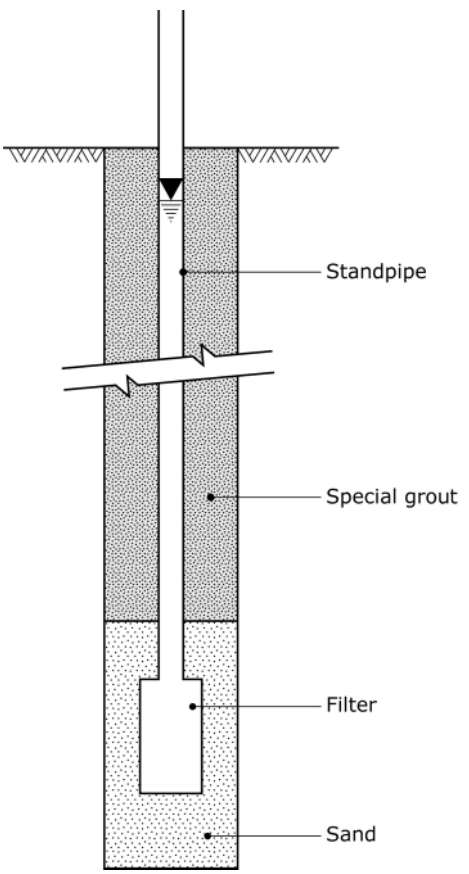


Figure 95.2 Schematic of open standpipe piezometer installed in a borehole

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jetting of the sand, the bottom of the grout pipe should be plugged and the grout pipe should have side-discharge holes with either grouting under gravity or pumping very slowly.

As for observation wells, the elevation of the water surface in the standpipe is determined by sounding with an electrical probe.

Open standpipe piezometers have a long successful performance record, and can be converted to remote-reading piezometers by insertion of pressure transducers. However, they have a long time lag (response time), are subject to damage by construction equipment and by vertical compression of soil around the standpipe, and have freezing problems if the piezometric level rises above frost line.

95.2.4 Vibrating wire piezometers

As shown in **Figure 95.3**, the piezometer has a metallic diaphragm separating the pore water from the measuring system. A tensioned wire is attached to the mid-point of the diaphragm such that deflection of the diaphragm causes changes in wire tension. As with a piano string, the frequency of vibration varies with the tension in the wire. The wire is plucked magnetically by an electrical coil attached near the wire at its mid-point, and either this same coil or a second coil is used to measure the frequency of vibration by using a frequency counter. The manufacturer supplies a calibration between frequency and pressure. In the figure P is the current pore water pressure, K is a constant that is provided by the manufacturer, f_o is the pre-installation frequency of vibration and f is the current frequency of vibration. Vibrating wire piezometers respond to changes of atmospheric pressure so that, if these changes need to be removed from monitored data, atmospheric pressure must be monitored separately. Attempts to circumvent this issue by using 'vented' piezometers have generally been unsuccessful, as the small diameter vent tube usually does not allow for pressure equalisation.

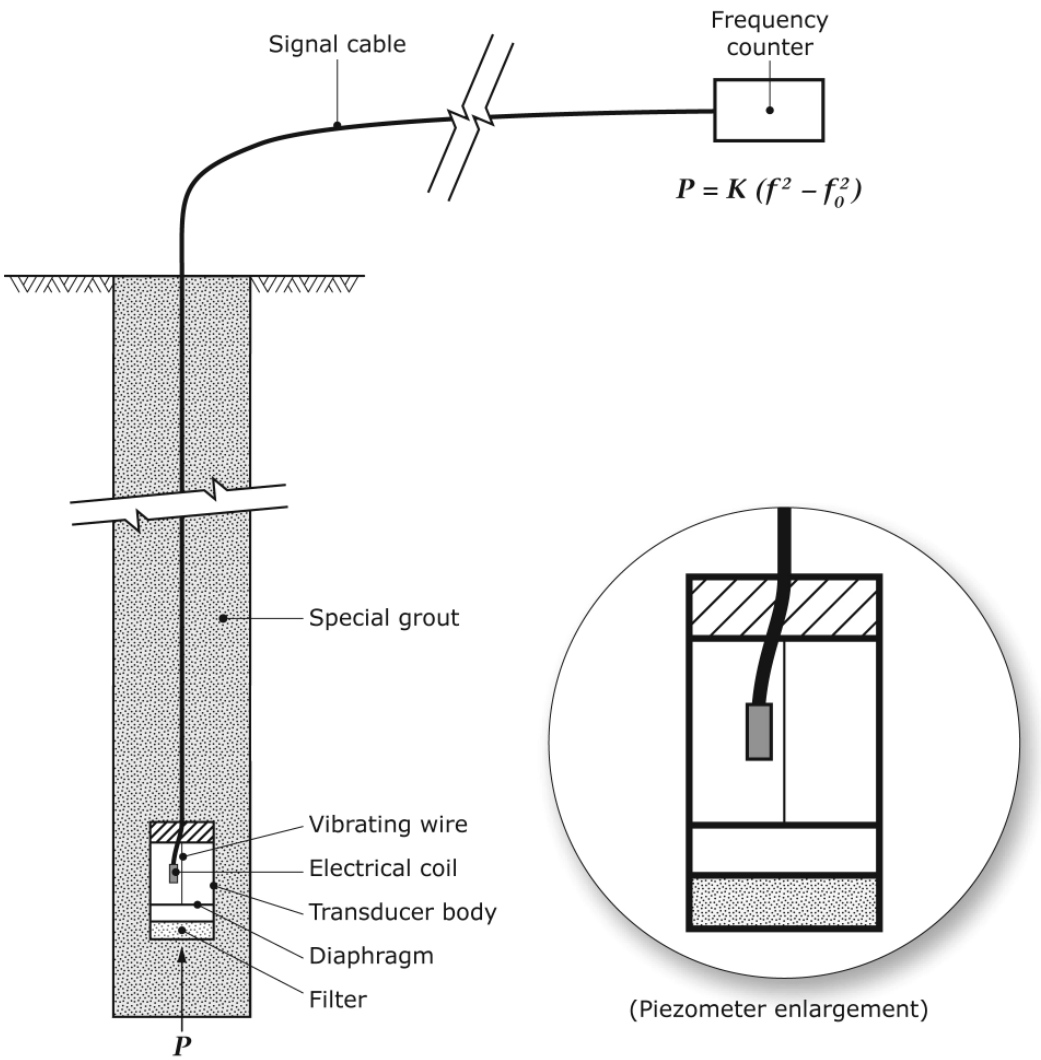


Figure 95.3 Schematic of vibrating wire piezometer installed in a borehole by the fully grouted method

Figure 95.3 shows the piezometer installed by the 'fully grouted method' (Contreras *et al*., 2008; Weber, 2009). As mentioned above, following Casagrande's installation procedure for open standpipe piezometers (Casagrande, 1949), a seal of bentonite chips or pellets used to be installed between the sand and the special grout. Several decades ago Vaughan (1969) showed that it is acceptable to use the fully grouted method, but few practitioners adopted the procedure. It is not necessary to use sand and bentonite chips or pellets (Contreras *et al*., 2008). Dunnicliff (2008) summarises some world-wide experiences with the fully grouted method, providing contact information for eight practitioners who have adopted the method as their standard procedure. Typical borehole diameter is 100–150 mm.

Wireless data transmission is available for vibrating wire piezometers. They are also manufactured for installation in soft soils by the push-in method.

Vibrating wire piezometers are very often the instruments of choice for monitoring groundwater pressure. They are easy to read and to datalog, have a short time lag, cause minimum interference to construction, and lead wire effects are minimal. They have long-term stability and can be used with long signal cables. However, lightning protection is required in locations where electrical storms occur.

95.2.5 Pneumatic piezometers

Figure 95.4 shows the basic arrangement of a pneumatic piezometer. An increasing gas pressure is applied to the inlet tube and, while the gas pressure is less than the pore water pressure, it merely builds up in the inlet tube. When the gas pressure exceeds the pore water pressure, the diaphragm deflects, allowing gas to circulate behind the diaphragm into the outlet tube, and flow is recognised either by noting a peak on the

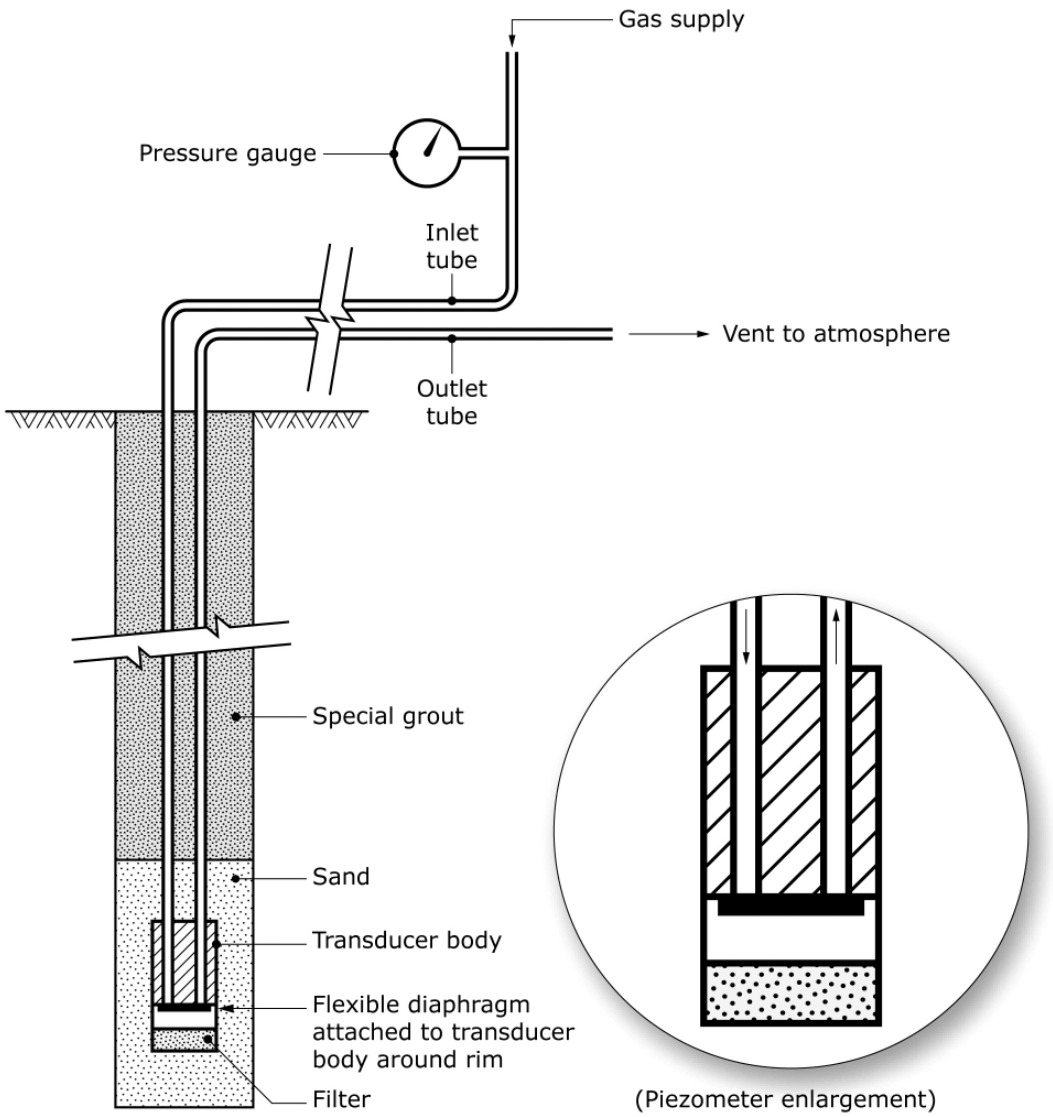


Figure 95.4 Schematic of pneumatic piezometer installed in a borehole

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pressure gauge, or by hearing the gas flow emerging from the outlet tube. The gas supply is then shut off at the inlet valve, and any excess pressure in the tubes bleeds away, such that the diaphragm returns to its original position when the pressure in the inlet tube equals the pore water pressure. This pressure is read on a Bourdon tube or electrical pressure gauge. As for vibrating wire piezometers, pneumatic piezometers respond to changes of atmospheric pressure so that, if these changes need to be removed from monitored data, atmospheric pressure must be monitored separately. As for open standpipe piezometers, there is no need for a bentonite chip or pellet seal between the sand and special grout. The fully grouted method is not recommended for installation of pneumatic piezometers. Typical borehole diameter is 100–150 mm.

Pneumatic piezometers are also manufactured for installation in soft soils by the push-in method.

As for vibrating wire piezometers, they have a short time lag and cause minimum interference to construction. There is no need for lightning protection, but they cannot readily be datalogged and readings are somewhat operator-dependent.

95.2.6 Twin-tube hydraulic piezometers

Twin-tube hydraulic piezometers were developed for installation in the foundations and fill during construction of embankment dams. Each consists of a porous filter element connected to two plastic tubes, with a Bourdon tube pressure gauge or pressure transducer on the end of each tube. The tubes are filled with de-aired water and piezometric elevation is determined from the pressure readings. When used in the unsaturated cores of embankment dams attention must be paid to many details to ensure that pore water pressure (as opposed to pore gas pressure) is measured, and that the tubes remain continuously filled with water (Dunnicliff, 1988, 1993).

95.2.7 Flushable piezometers for measuring negative pore water pressures

Flushable piezometers (Ridley, 2003; Ridley *et al.*, 2003) have been developed primarily for measuring negative pore water pressures. Appropriate applications are slopes and excavations in clays. Negative pore water pressures will transmit tension to the water in a piezometer. This is likely to lead to the formation of air in a piezometer and eventually it can become dry. If air is present in a piezometer it may not record the correct pore water pressures. Therefore the air must be removed and replaced with fresh de-aired water.

Twin-tube hydraulic piezometers are inherently flushable, but suffer from the principal disadvantage that the hydraulic tubes that are used for flushing water through them have to be laid horizontally to maximise the range of negative pore water pressures that can be recorded and to avoid air becoming trapped within the tubes. The flushable piezometer shown in **Figure 95.5** overcomes this problem by incorporating a hydraulically operated shuttle valve that enables the piezometers to be installed in boreholes, typically 70 mm diameter. The valve is used to isolate the sensor from the flushing tubes, thereby enabling the piezometers to measure positive pore water pressures and negative pore water pressures to approximately –95 kPa, irrespective of the depth of installation. Using

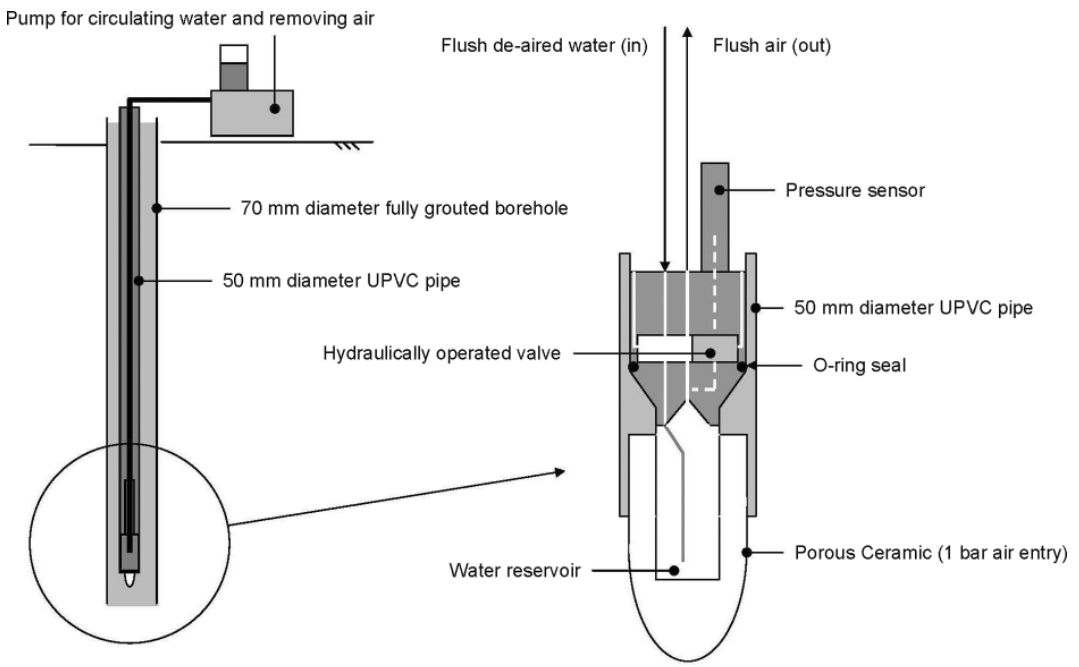


Figure 95.5 Schematic of flushable piezometer
Courtesy of Geotechnical Observations Ltd, Weybridge, Surrey

a high air entry porous filter also inhibits the formation of air in the piezometer, and adopting the fully grouted method of installation allows hydraulic continuity to be maintained between the pore water and the water in the piezometer.

95.2.8 Fibre-optic piezometers

Fibre-optic piezometers are recent developments (Glišić and Inaudi, 2007; Inaudi and Glišić, 2007a). They are categorised as 'Fabry-Peŕot interferometric point sensors', and consist of a capillary glass tube containing two partially mirrored optical fibres facing each other, but leaving an air cavity of a few microns between them. When light is coupled into one of the fibres, a back-reflected interference signal is obtained, due to the reflection of the incoming light on the two mirrors. When packaged as a pressure transducer this signal is processed to indicate pressure.

95.3 Instruments for monitoring deformation

95.3.1 Surveying methods

Surveying methods are used to monitor the magnitude and rate of horizontal and vertical deformations of structures, the ground surface, and accessible parts of subsurface instruments in a wide variety of construction situations. In general, whenever geotechnical instruments are used to monitor deformation, surveying methods are also used to relate measurements to a reference datum: a benchmark for vertical deformation monitoring or a horizontal control station for horizontal deformation monitoring.

Surveyors who work on construction sites often have little experience with the accuracies required for deformation monitoring, and a well-trained survey crew is essential when maximum accuracy is required. Measurement accuracy is controlled by the choice and quality of surveying technique and by characteristics of reference data and measuring points (Cheney, 1973). Survey instrument technology is well established, and most reputable manufacturers include a statement of accuracy in their instrument specifications, which can be relied on if the instrument is calibrated and operated in accordance with instructions.

A detailed discussion of surveying methods is beyond the scope of this chapter, and readers should make use of information provided by manufacturers of surveying equipment. However, there is a current trend to make wide use of both manual and motorised (robotic) total stations (Cook, 2006; Beth 2007; Kontogianni, 2007; Hope and Chaqui, 2008; Marr, 2008; Volterra, 2008). Total stations are survey instruments combining a theodolite and electronic distance measurement. Targets are mounted at the locations of interest and their deformation is monitored in 3D space. When a motor is added to the theodolite and automatic target recognition included, monitoring can be achieved automatically and remotely.

95.3.2 Probe extensometers

Probe extensometers are defined in this chapter as devices for monitoring the changing distance between two or more points along a common axis, by passing a probe through a pipe. Measuring points along the pipe are identified by the probe, and the distance between points is determined by measurements of probe position. Typical applications of probe extensometers are monitoring vertical strain within embankments or embankment foundations, and settlement alongside excavations and above tunnels.

Various types of transducers are used in probe extensometers, including induction coils and magnet/reed switch transducers. A magnet/reed switch transducer is an on/off position detector, arranged to indicate when the reed switch is in a certain position with respect to a magnet. The switch contacts are normally open and one of the reeds must be magnetically susceptible. When the switch enters a sufficiently strong magnetic field, the reed contacts snap closed and remain closed as long as they stay in the magnetic field. The closed contacts actuate a buzzer or indicator light in a portable read-out unit (Burland *et al*., 1972). A schematic of a borehole installation is shown in **Figure 95.6**.

The spider magnets shown in **Figure 95.6** are suitable for installation in soft ground. The three springs above the magnets and the three below are held back while being lowered into the borehole, and tripped once at the required depth. To ensure that a spider magnet moves the same as the ground around it, the stiffness of the six springs should be such that a force of at least 20 N is required to move a 200-mm-long

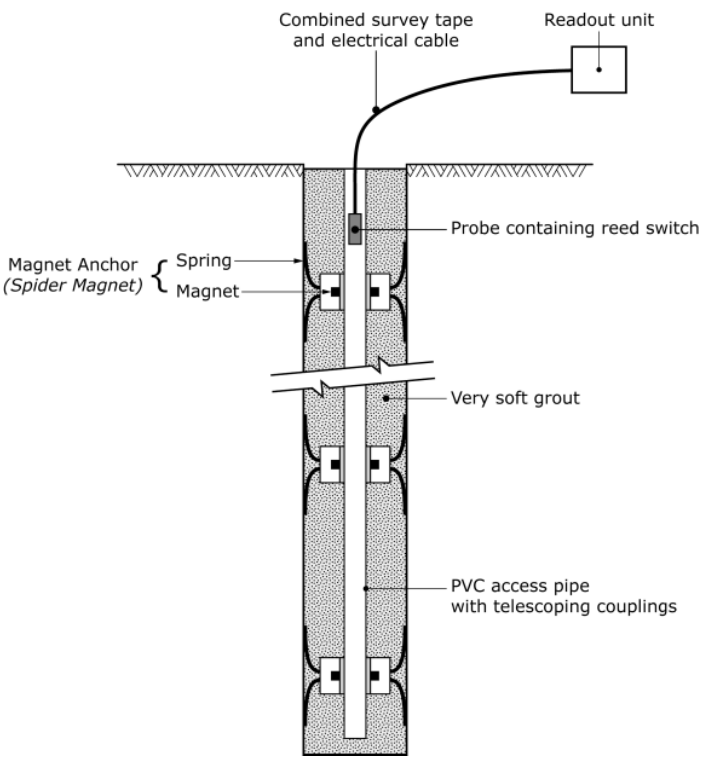


Figure 95.6 Schematic of probe extensometer with magnet/reed switch transducer

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spring from its extended position to the longitudinal axis of the magnet housing. The tip-to-tip diameter of the springs, when in their extended positions should be at least 220 mm. For installation in stiff ground it is preferable to have only the three upper springs and to push them (no holding back) to the required depth with an insertion pipe that surrounds the access pipe, followed by an upward pull to ensure that they 'bite' into the ground. A bayonet disconnect between the bottom of the insertion pipe and the magnet housing allows removal of the insertion pipe.

The diameter of the access pipe is typically 33 mm, and the borehole diameter typically 65–225 mm. A telescoping access pipe is needed when the predicted vertical strain is greater than about 1%.

Some users have installed this instrument surrounding inclinometer casing, hoping to be able to monitor deformation in 3D. However, because the criteria for grout properties are different – high compressibility for vertical deformation data and significantly less compressibility for horizontal deformation data – data have often been unsatisfactory.

95.3.3 Fixed borehole extensometers

Fixed borehole extensometers are defined in this chapter as devices installed in boreholes in soil or rock for monitoring the changing distance between two or more points along the axis of a borehole, without use of a movable probe. When the location of a measurement point is determined with respect to a fixed reference datum, the devices also provide absolute deformation data. The operating principle is shown in **Figure 95.7**. Typical applications are monitoring deformations around underground excavation in rock and behind the faces of excavated slopes.

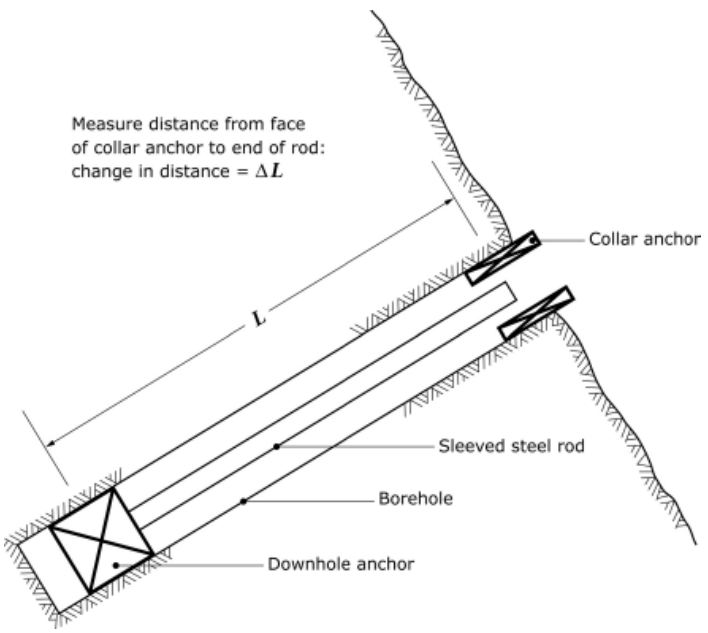


Figure 95.7 Operating principle of fixed borehole extensometer

The distance from the face of the collar anchor to the end of the rod is measured using either a mechanical or an electrical transducer. The device shown is a single-point borehole extensometer (SPBX), but several down-hole anchors can be located in a single borehole, each with an attached rod from the down-hole anchor to the collar anchor, to create a multi-point borehole extensometer (MPBX). The normal maximum number of anchors and rods for an MPBX is eight, and typical borehole diameter is 150 mm.

Many types of fixed borehole extensometers are available, the primary variables being choices of anchor type, SPBX or MPBX, transducer type, and extensometer head. Wireless data transmission is available for fixed borehole extensometers.

95.3.4 Tiltmeters

Tiltmeters are used to monitor the change in inclination (rotation) of points on or in the ground or a structure. A tiltmeter consists of a gravity-sensing transducer within an appropriate housing, either fixed-in-place or arranged as a portable device by mating with reference points permanently attached to the surface of the ground or structure. Various types of transducers are used in tiltmeters, including force balance servo-accelerometers, electrolytic levels (electrolevels), vibrating wire transducers and micro-electrical mechanical systems (MEMS). Wireless data transmission is available for tiltmeters.

Force balance servo-accelerometers are most familiar to users of instrumentation as the transducers in inclinometers. The transducer is based on a flexure-suspended proof mass which is deflected as the component of gravity changes with tilt angle.

An electrolytic level consists of a sealed glass vial similar to the vial on a conventional spirit level, partly filled with a conductive liquid. Electrical conductors inserted through the vial enable tilt to be monitored remotely. Although an inexpensive and often-used transducer, problems have often been experienced with temperature sensitivity.

A vibrating wire transducer consists of a pendulous mass which is supported by a vibrating wire strain gauge and an elastic hinge.

MEMS (Sellers and Taylor, 2008; Sheahan *et al.*, 2008) have recently become available as tilt transducers for the geotechnical instrumentation community. They are the integration of mechanical elements, sensors, actuators and electronics on a common silicon substrate through microfabrication technology. There are numerous types of MEMS, a large and rapidly growing high technology area. Similar to the force balance servo-accelerometer, a MEMS tilt sensor is based on a flexure-suspended proof mass which is deflected as the component of gravity changes with tilt angle. A position sensor senses this movement by differential capacitance sensors of high sensitivity. The proof mass, flexible mounting, position sensor and supporting electronics are all constructed from a single wafer of non-metallic material, usually silicon.

95.3.5 Inclinometers

Inclinometers are devices for monitoring horizontal deformation below the ground surface. Typical applications include determining the zone of landslide movement, monitoring the extent and rate of horizontal movement of embankments on soft ground and alongside open cut excavations, and monitoring the deflection of bulkheads, piles or retaining walls.

Inclinometers have four major components. First, a permanently installed guide casing, usually made of plastic or fibreglass, installed in a near-vertical alignment. The guide casing has tracking grooves for controlling orientation of the probe. Second, a portable probe containing two tilt transducers mounted 90 degrees apart for monitoring horizontal deformations in two directions. Third, a portable read-out unit for power supply and indication of probe inclination. Fourth, a graduated electrical cable linking the probe to the read-out unit.

Figure 95.8 shows the normal principle of inclinometer operation. After installation of the casing, the probe is lowered to the bottom and an inclination reading is made. Additional readings are made as the probe is raised incrementally to the top of the casing, providing data for determination of initial casing alignment. The differences between these initial readings and a subsequent set define any change in alignment. Provided that the lower end of the casing is fixed from translation by installing it in material such as bedrock that will not move laterally, these differences allow calculation of absolute horizontal deformation at any point along the casing.

Standard inclinometer casing diameters are 70 mm and 85 mm.

Most commercially available inclinometers make use of a force balance servo-accelerometer transducer but recently MEMS transducers have become available. Most have portable read-out units that store data on on-board memory.

Inclinometer casings can also be installed horizontally such that vertical deformation data are monitored, for example below embankments on soft ground.

Valuable guidelines on inclinometer data analysis are given by Mikkelsen (2003).

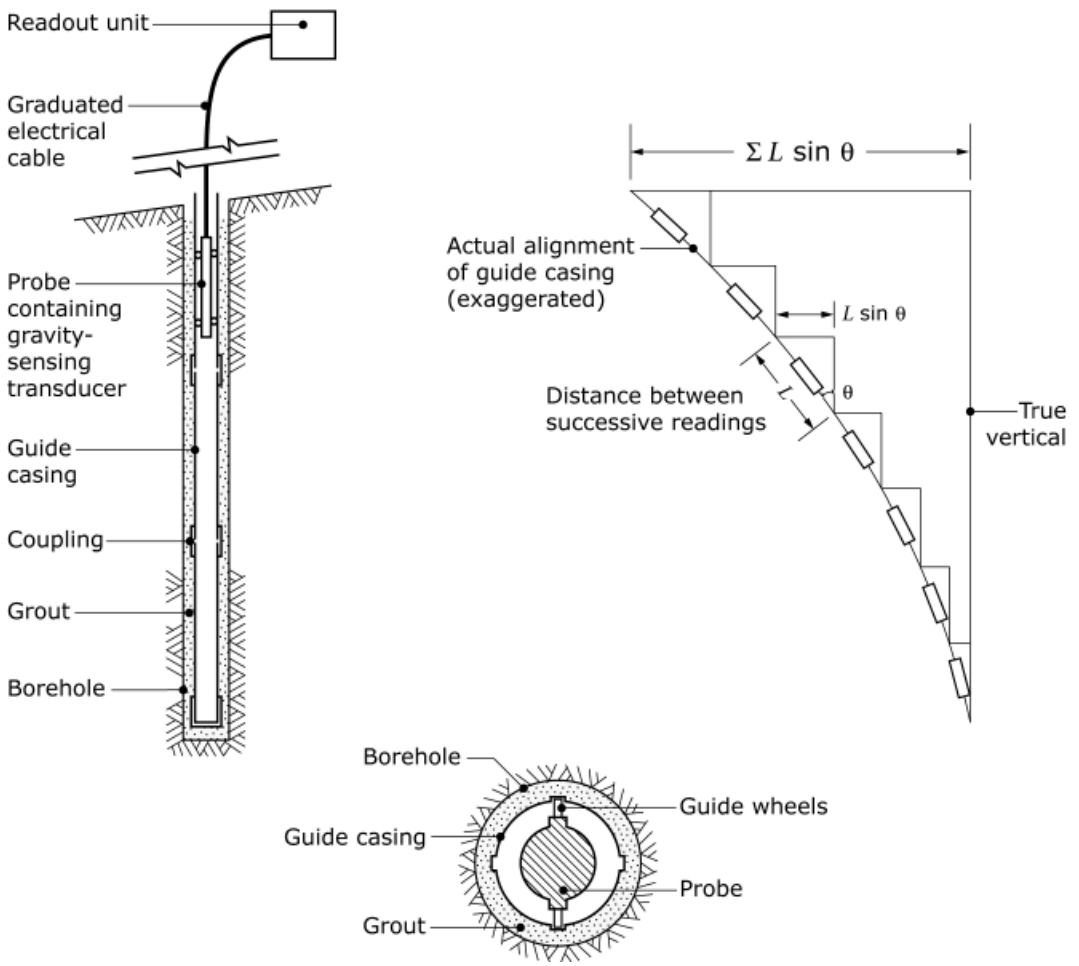


Figure 95.8 Principle of inclinometer operation

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95.3.6 In-place inclinometers

In-place inclinometers are typically used for monitoring subsurface deformations around excavations or within slopes, when rapid or automatic monitoring is required.

An in-place inclinometer is generally designed to operate in a near-vertical borehole and provides essentially the same data as a conventional inclinometer. The device, shown schematically in **Figure 95.9**, consists of a series of tilt transducers joined by articulated rods. The transducers are the same as used in tiltmeters – force balance servo-accelerometers, electrolytic levels (electrolevels), vibrating wire transducers and MEMS. Uniaxial or biaxial transducers can be used. The transducers are positioned at intervals along the borehole axis and can be concentrated in zones of expected movement. Movement data are calculated using the same methods as for conventional inclinometers. Wireless data transmission is available for in-place inclinometers.

In-place inclinometers can be used effectively in combination with a conventional inclinometer. An in-place version can first be installed to define the location of any horizontal deformation, with minimal labour costs for reading. If deformation occurs, the in-place system can be removed and the moving zone monitored with a conventional inclinometer. Alternatively, a conventional inclinometer can be used first to indicate any deformation and an in-place version later installed across a critical deforming zone to minimise subsequent effort and perhaps to provide an alarm trigger.

A recent development, the ShapeAccelArray, or SAA (Abdoun and Bennett, 2008; Barendse, 2008; Mikkelsen and Dunnicliff, 2008; www.measurand.com), consists of a series of 300-mm-long rigid segments connected by composite joints that prevent torsion but allow flexibility of segments in two degrees of freedom. MEMS transducers are contained in each segment, such that when installed in a vertical borehole, horizontal deformation data are obtained in real time. The instrument is typically installed within a 25 mm pipe and, in contrast to inclinometers and conventional in-place inclinometers, tracking grooves are not required for controlling orientation. The SAA can also be installed horizontally to monitor vertical deformation.

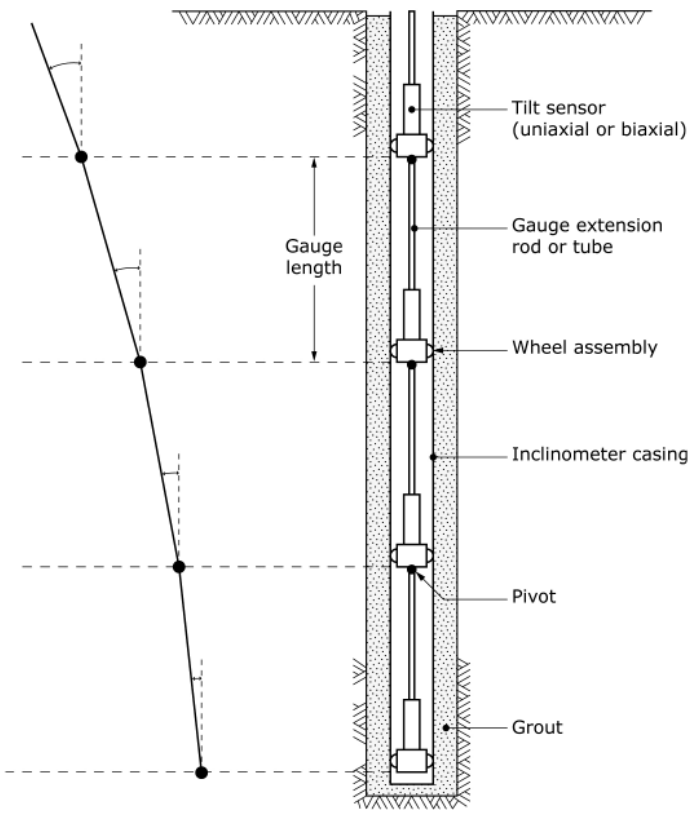


Figure 95.9 Schematic of in-place inclinometer

95.3.7 Liquid level gauges

Liquid level gauges are defined in this chapter as instruments that incorporate a liquid-filled tube or pipe for determination of relative vertical deformation. Relative elevation is determined either from the equivalence of liquid level in a manometer or from the pressure transmitted by the liquid.

The primary application for liquid level gauges is monitoring settlements within embankments or embankment foundations. They allow installation to be made without frequent interruption to normal fill placement and compaction, and minimising the potential for instrument damage. Most liquid level gauges also allow measurements to be taken at a central reading location. The gauges only provide a means of measuring *relative* elevations between two or more points. If *absolute* settlement or heave is required, as is usually the case, data must be referenced to a benchmark. If one end of the gauge cannot be mounted directly on a benchmark, a surveying method will normally be used, and accuracy may be dependent on accuracy of the surveying method.

The most common type of liquid level gauge is shown in **Figure 95.10**. The pressure transducer is usually a vibrating wire transducer. The upper surface of the liquid column is at a known elevation at the read-out location, therefore relative elevation of the transducer and reservoir can be determined from the pressure measurement and liquid density. **Figure 95.10** shows only one liquid-filled tube, but in practice it is preferable to have two tubes so that flushing is possible and independent measurements can be made on each tube as a check. The greatest potential source of error is discontinuity of liquid caused by the presence of gas, and this is a frequent problem despite use of de-aired liquid. A much better arrangement is shown in **Figure 95.11** (McRae, 2000; www.geokon.com). Sufficient backpressure is applied to force any gas into solution, and any increase in backpressure will not change the transducer reading.

The same arrangement can be packaged as a probe that is pulled along a buried pipe, thereby providing vertical deformation data at any point along the pipe. This is called a full-profile liquid level gauge.

Various liquid level gauges (sometimes called liquid levelling systems) are available with interconnected sensors, for

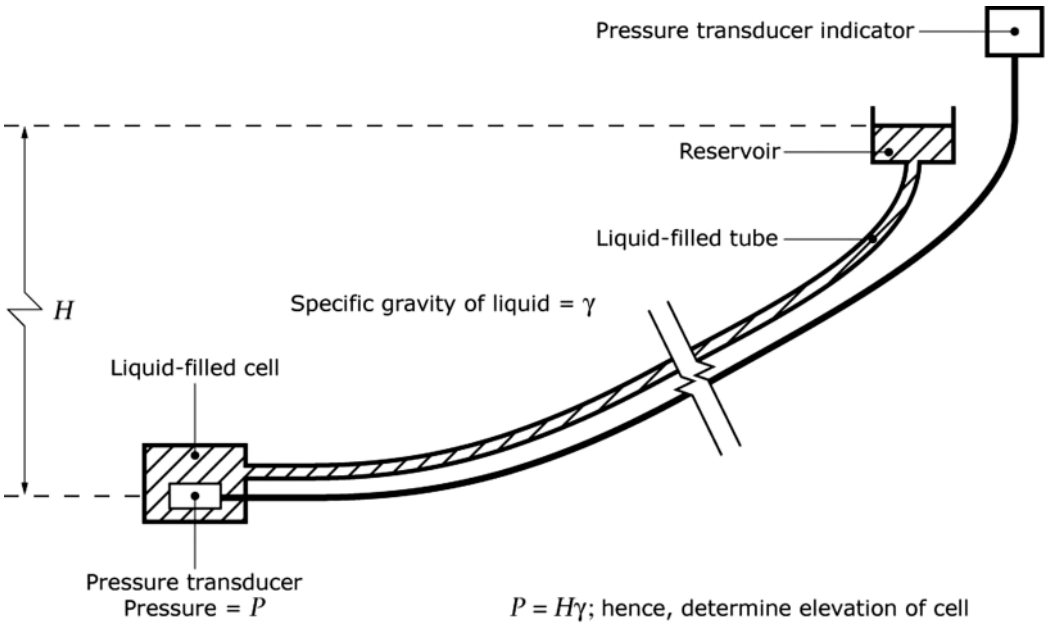


Figure 95.10 Schematic of liquid level gauge

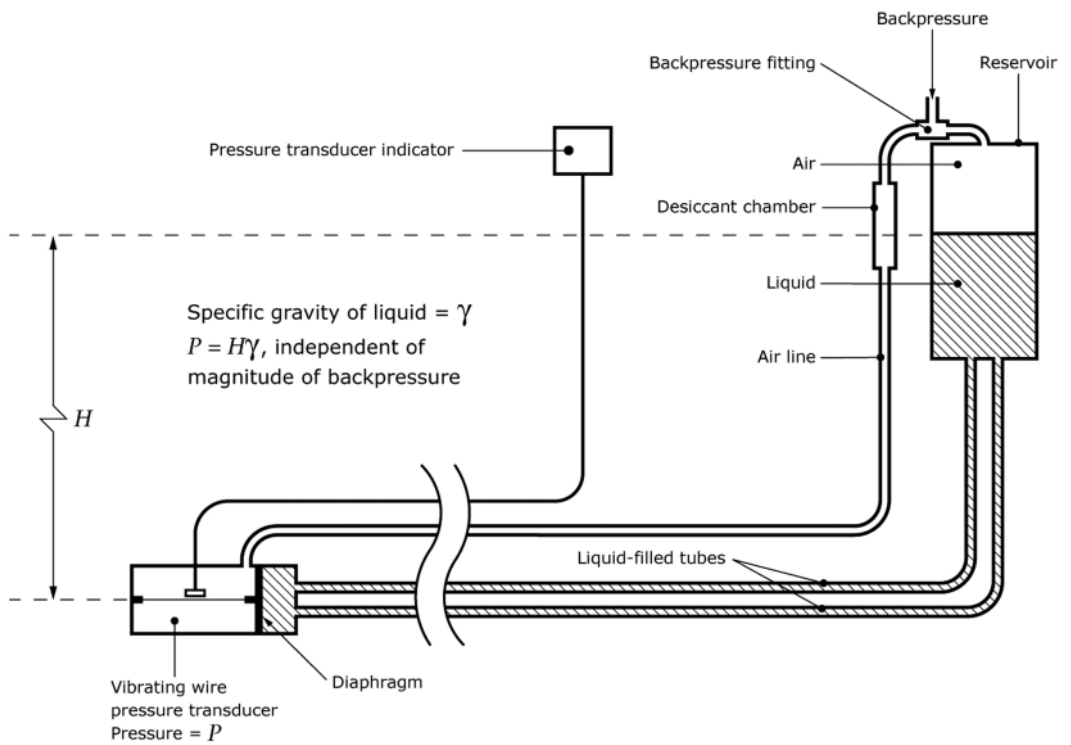


Figure 95.11 Schematic of closed loop liquid level gauge
Courtesy of Geokon, Inc., Lebanon, NH, USA

monitoring settlement at a series of locations. The sensors are connected together with both a liquid-filled tube and a tube for equalising air pressure. However, the small diameter of the liquid-level tube normally results in discontinuities in the liquid and a consequent major maintenance effort.

95.3.8 Crack gauges

Crack gauges (sometimes called jointmeters) are typically used for monitoring tension cracks behind slopes and for monitoring cracks in concrete or other structures, or joints or faults in rock. There are numerous versions, including portable or

fixed-in-place mechanical gauges, and portable or fixed-in-place gauges that incorporate a locally or remotely read displacement transducer. Wireless data transmission is available for crack gauges with electrical transducers.

95.3.9 Convergence gauges

Convergence gauges have typically been used for monitoring convergence across braced excavations and tunnels. While they are still used for these applications today, in most cases the measurements are now made by using total stations.

A typical convergence gauge, often called a tape extensometer, has a spring to control tension in the tape, and each end of the tape is attached to an anchor installed at an end of the span to be monitored. After attachment of the extensometer to the anchors and standardising the tension, readings of distance are made by adding the dial indicator reading to the tape reading.

95.3.10 Settlement platforms

Settlement platforms are typically used for monitoring settlement below embankments on soft ground. A settlement platform consists of a square plate of steel, wood or concrete placed on the original ground surface, to which a riser pipe is attached. Optical levelling measurements to the top of the riser pipe provide a record of plate elevations. The riser pipe is extended as embankment filling proceeds.

95.3.11 Fibre-optic instruments

Fibre-optic instruments for monitoring deformation are recent developments. The basic principle is given in the earlier section on fibre-optic piezometers. The signal is processed to indicate deformation. Glišić and Inaudi (2007) and Inaudi and Glišić (2007a) describe four types of fibre-optic sensing techniques: Fabry-Peŕot interferometric point sensors for monitoring distance across a discontinuity such as a crack, fibre Bragg Grating sensors that allow deformation measurements at multiple points along a single fibre line, SOFO interferometric sensors that are used for integrating deformation measurement over a long measurement base, and distributed fibre-optic sensors that are able to sense deformation at any point along a single fibre line. Inaudi and Glišić (2007b) describe the fourth technique in more detail. This last technique has very high potential for monitoring deformation over large areas, such as on slopes.

95.3.12 Time domain reflectometry

Time domain reflectometry (TDR) (O'Connor and Dowding, 1999) is a remote sensing electrical technique, similar in concept to radar along a cable. A voltage pulse, produced by a TDR pulser, travels along a two-conductor coaxial metallic cable until it is partially reflected by deformation of the cable. The distance to the deformation can be calculated knowing the propagation velocity of the signal in the cable and the time of travel of the voltage pulse from the disruption to the cable tester. TDR is used for monitoring unstable slopes, and for other applications for which an inclinometer might be considered – this application is described by Dowding *et al*. (2003). O'Connor (2008) describes geotechnical alarm systems based on TDR technology, and Lin (2009) presents some details of various types of measurements. Wireless data transmission is available for TDR.

95.3.13 Global positioning system

The global positioning system (GPS) consists of three parts: satellites, a ground control network, and user equipment. Radio signals are used in an interferometric mode. Two or more GPS receivers simultaneously receive signals from the same set of satellites, and the resulting observations are subsequently processed to obtain the inter-station difference in position. If one of the receivers is placed at a known position, the three-dimensional position of the second receiver may be determined, and the number of stations determined simultaneously is limited only by the number of receivers available. GPS is proving to be very useful for long-term performance monitoring of dams and other large structures (Rutledge and Meyerholtz, 2005).

95.3.14 Acoustic emission monitoring

Acoustic emission (AE) is a non-destructive technique for providing an early warning of failures. The primary application is for monitoring slopes in soil and rock. AE is a natural phenomenon that occurs when a solid is subjected to stress. This stress, from an external source, causes a sudden release of sound waves resulting in microseismic activity, which can be detected by transducers. Within a slope the stress induced by destabilising forces causes a re-arrangement of particles along developing shear surfaces. This inter-particle friction results in the release of AE, and is an indication of straining within a soil body.

Although historically used as a qualitative monitoring technique, Spriggs and Dixon (2005) describe a quantitative approach to using AE data as an early warning system.

95.4 Instruments for monitoring load and strain in structural members

95.4.1 Introduction

Instruments for measuring load and strain in structures fall into two groups: load cells and strain gauges. In each case, the transducers are used to measure small extensions and compressions. Load cells are interposed in the structure in such a way that structural forces pass through the cells, and strain gauges are attached directly to the surface of the structure or are embedded within the structure to sense the extensions and compressions in the structure itself.

Typical load cell applications include load testing of driven and bored piles, tiebacks and rockbolts, and long-term performance monitoring of tiebacks and end-anchored rockbolts. Calibrated hydraulic jacks are designed for application of load, and not for measurement of load. When used for load measurement in the field, errors caused by friction between the cylinder and the piston can cause underestimates of load of up to 20%.

Strain gauges are used where load cells cannot be interposed in the structure for reasons of geometry, capacity or economy and where load and stress can be calculated with adequate accuracy from knowledge of the relationship between strain and stress. Strain gauges are used, for example, to determine temporary or permanent stresses or loads in struts across braced excavations, in rockbolts, retaining walls, diaphragm walls and deep foundations.

95.4.2 Load cells

Most load cells have either electrical resistance strain gauge or vibrating wire transducers. As shown in **Figure 95.12** a cell with electrical resistance strain gauge transducers consists of a cylinder of steel or aluminum alloy, with electrical resistance strain gauges bonded to the outer periphery of the cylinder at its mid-section. Several strain gauges are used at regular intervals around the periphery, thereby reducing errors that result from load misalignment and off-centre loading. However, in all cases adequately thick bearing plates should be used at both ends of the cell, ground flat, smooth and parallel. Typical thicknesses are 38, 63 and 76 mm for cell capacities of 0.7, 1.8 and 2.2 MN respectively.

In most vibrating wire load cells, deformation of the load-bearing member is measured by using three or more vibrating wire transducers, and outputs from each transducer must be measured separately and averaged. The arrangement is similar to the electrical resistance load cell shown in **Figure 95.12**, but with vibrating wire transducers instead of bonded resistance strain gauges. However, the vibrating wire strain gauges are installed at a diameter mid-way between the inside and outside diameters of the cylindrical load-bearing member, thereby minimising errors caused by size mismatch between the cell and adjacent components. Wireless data transmission is available for load cells.

Load cells with outside diameters ranging from 80 to 320 mm are available, the size depending on the load range.

Sellers (1994) reports on tests made to examine the error caused by a size mismatch between electrical resistance load cells and adjacent hydraulic jacks. If the hydraulic jack is larger than the load cell there is a tendency for it to try to wrap the intervening bearing plate around the load cell and to bend the cylinder walls inward. If the hydraulic jack is smaller than the load cell it will try to push the intervening bearing plate through the hole in the load cell and to bend the cylinder

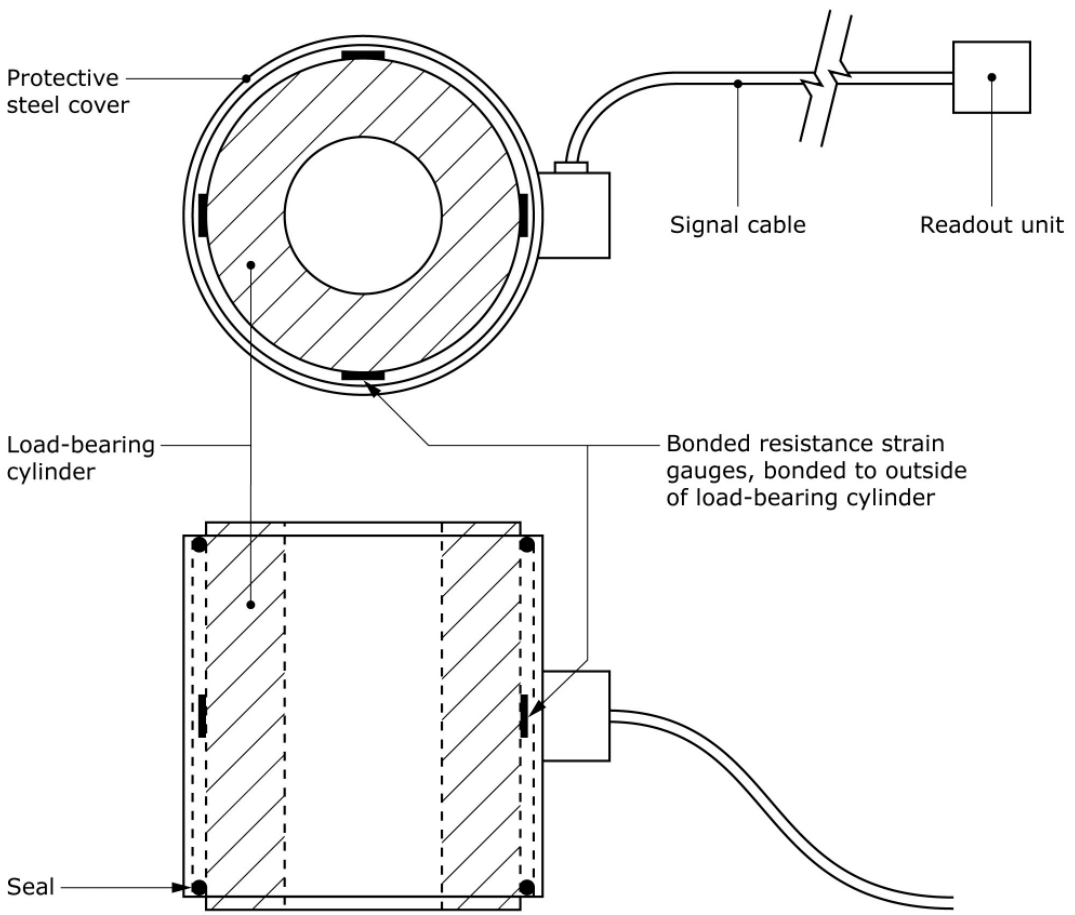


Figure 95.12 Schematic of load cell with electrical resistance transducers

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walls outward. Sellers reports on errors of up to 8%. Dunnicliff (1995) reports on tests made by Sellers, subsequent to the tests referred to above, to examine the errors caused by a size mismatch between vibrating wire load cells and adjacent hydraulic jacks, where maximum errors were only 1%. In summary, it appears that cells with vibrating wire transducers are preferable to cells with electrical resistance strain gauge transducers.

Calibrated hydraulic jacks are used for the application of load to rockbolts, tieback anchors, piles, struts across braced excavations, and other structural elements. However, reliance on the measured pressure in the jack fluid as the sole method for load determination can often lead to significant inaccuracies. Off-centre and/or misaligned loading causes friction between the piston and cylinder, so use of the hydraulic pressure as an indication of load will indicate that the load is more than it truly is, i.e. a measurement on the unsafe side. Good practice when load testing piles and tiebacks includes use of a load cell in series with the hydraulic jack.

95.4.3 Surface-mounted strain gauges

Historically, three types of strain gauges have been used for measurements on structural surfaces: mechanical, electrical resistance and vibrating wire, but most applications today make use of vibrating wire gauges. The recent development of fibre-optic instruments provides a fourth method.

A schematic of a surface-mounted vibrating wire gauge is shown in **Figure 95.13**. The operating principle is described above in the section on vibrating wire piezometers. The gauge is available in the two basic configurations: typically 100–200 mm long for installation by arc welding and typically 50 mm long for installation by spot welding. When compared with the arc-welded gauge, advantages of the spot-welded gauge are small size, an installation procedure not requiring arc welding, and minimum errors that result from bending of the structure because of the close proximity between the vibrating wire and the structure surface. However, if adequate space is available, the larger arc-welded version is generally preferred. Wireless data transmission is available for these gauges.

Fibre-optic gauges are described above for monitoring deformation, and references are given. When used for monitoring strain all the same four techniques are applicable. Bennett (2008) describes some applications for the fourth technique, distributed fibre-optic sensors, for monitoring strain in piles and tunnels.

Strain data are rarely of interest; the data are merely a step in the determination of stress. When making measurements on steel, for which the modulus is of course known, strict attention must be made to the influence of temperature (Boone and Crawford, 2000; Druss, 2000; Boone and Bidhendi, 2001; Hashash and Marulanda, 2003; Daigle, 2005; Osborne and Tan, 2009).

Stress determination in concrete is by no means straightforward, and accurate results should not be expected. Strain in concrete can be caused by several factors other than stress change, and a strain gauge responds to all causes of strain. Strain other than that caused by stress may be due to creep (strain under constant stress), shrinkage and swelling (moisture content change), temperature change, and the progress of autogenous volume change (dimensional change that is self-generated and not due to temperature, moisture change or stress); other strain may also occur as concrete cures. Strain due to creep is generally more extensive in highly stressed pre-stressed concrete than in conventional reinforced concrete; therefore, for structural members such as pre-stressed concrete piles, creep strains are particularly significant. A special effort must therefore be made to determine the relationship between strain and stress, as described by Dunnicliff (1988, 1993) and Acerbis *et al*. (2011).

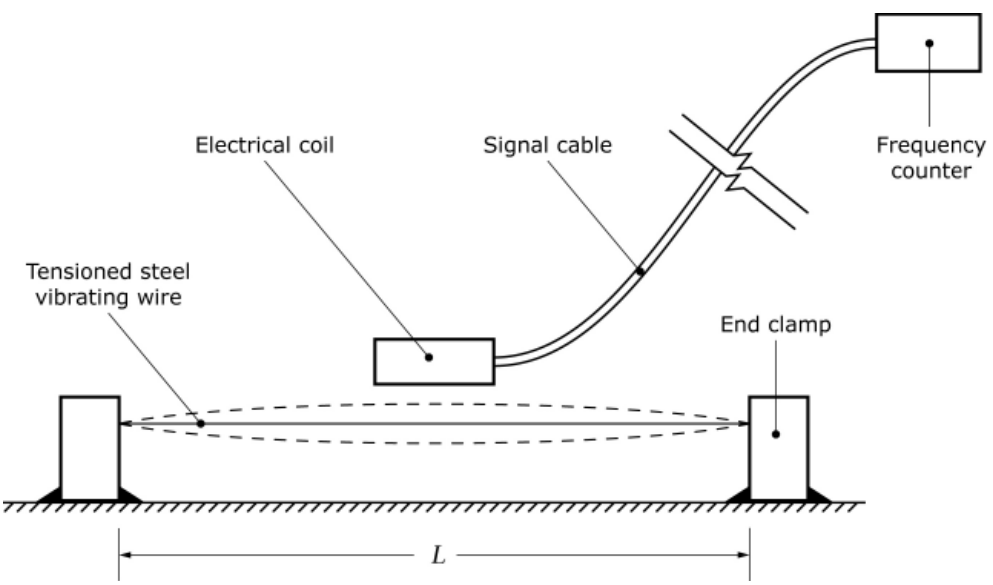


Figure 95.13 Schematic of surface-mounted vibrating wire strain gauge

95.4.4 Embedment strain gauges

The primary application for embedment strain gauges is measurement of strain in concrete. Typical applications are driven concrete piles, bored piles, diaphragm walls, concrete tunnel linings and concrete dams. As for surface-mounted versions, vibrating wire gauges are used in most applications today, although the recent development of fibre-optic gauges holds promise.

There are two basic types of vibrating wire gauge. The first is similar to the arrangement shown in **Figure 95.13**, with circular end flanges instead of the end blocks, typically 100–200 mm long. Most gauges are supplied with a preset tension, which can be specified by the user. The second type consists of a vibrating wire transducer mounted in the central portion of a length of steel bar, such as reinforcing steel. The use of a length of steel bar, embedded in concrete to measure strain, is sometimes referred to as a sister bar, or a rebar strain meter. **Figure 95.14** shows a sister bar.

The first type cannot be installed without protection prior to concreting, because of the likely damage when the concrete is vibrated. It must therefore be encapsulated in a briquette of fine-aggregate concrete, and the briquette must not be more than 24 hours old at the time that the primary concrete is placed – if it is older than this there will be an inclusion effect that will distort the gauge reading. The second type can be installed directly in concrete without protection, and is therefore preferred if sufficient space is available.

As for gauges surface-mounted on concrete, a special effort must be made to determine the relationship between strain and stress, as described by Dunnicliff (1988, 1993) and Acerbis *et al*. (2011).

95.5 Instruments for monitoring total stress

95.5.1 Introduction

Total stress measurements in soil fall into two basic categories: measurements within a soil mass and measurements at the face of a structural element. Instruments are referred to as earth pressure cells, soil stress cells and soil pressure cells, and in this chapter the terms ‘embedment earth pressure cells’ and ‘contact earth pressure cells’ will be used for the two basic categories.

Embedment earth pressure cells are installed within fill, for example, to determine the distribution, magnitude and direction of total stress within an embankment. Applications for contact earth pressure cells include measurement of total stress against retaining walls, culverts, piles and diaphragm walls.

The primary reasons for use of earth pressure cells are to confirm design assumptions and to provide information for the improvement of future designs; they are less commonly used for construction control or other reasons. When concerned with stresses acting on a structure during construction or after construction is complete, it is usually preferable to isolate a portion of the structure and to determine stresses by use of load cells and strain gauges within the structure. For example, this approach has been used successfully in the determination of earth pressures in braced excavation from measurements of support loads.

95.5.2 Embedment earth pressure cells

Attempts to measure total stress within a soil mass are plagued by errors because both the presence of the cell and the installation method generally create significant changes in the free-field stress. Therefore it is usually impossible to measure total stress with great accuracy, and realistic applications are rare.

Most available embedment earth pressure cells are of the type shown in **Figure 95.15**. A cell consists of two circular or rectangular steel plates, welded together around their periphery, with liquid filling the intervening cavity and a length of high-pressure steel tubing connecting the cavity to a nearby pressure transducer. Total stress acting on the outside of the cell is balanced by an equal pressure induced in the internal liquid. It is essential that the cell is filled with de-aired liquid and that no gas bubbles are trapped within the cavity during filling. Typical cell diameter is 200 mm, but larger diameter cells are available.

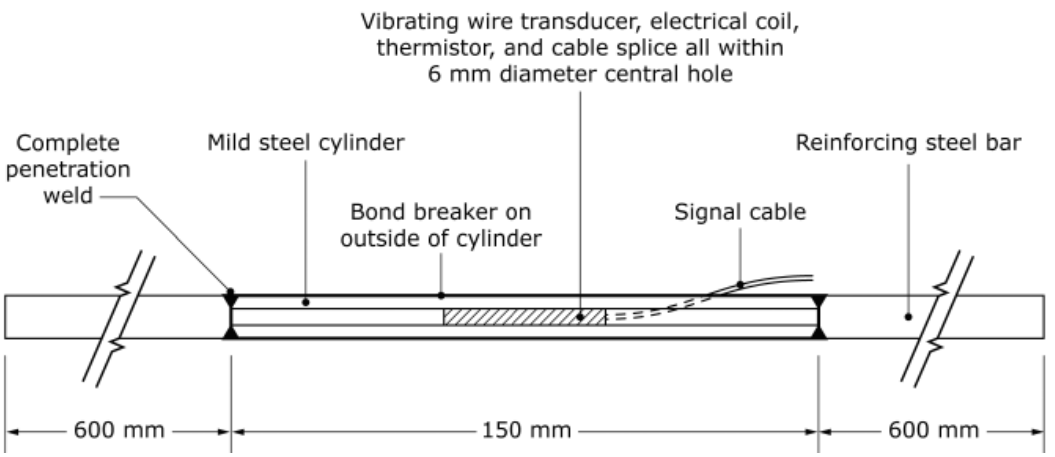


Figure 95.14 Schematic of sister bar with vibrating wire transducer

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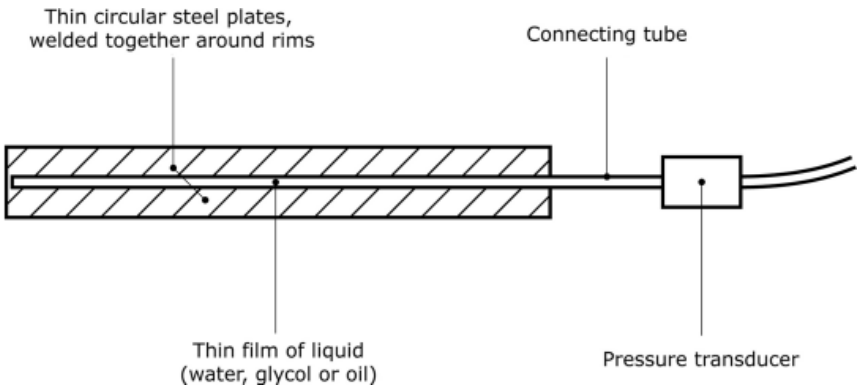


Figure 95.15 Schematic of embedment earth pressure cell

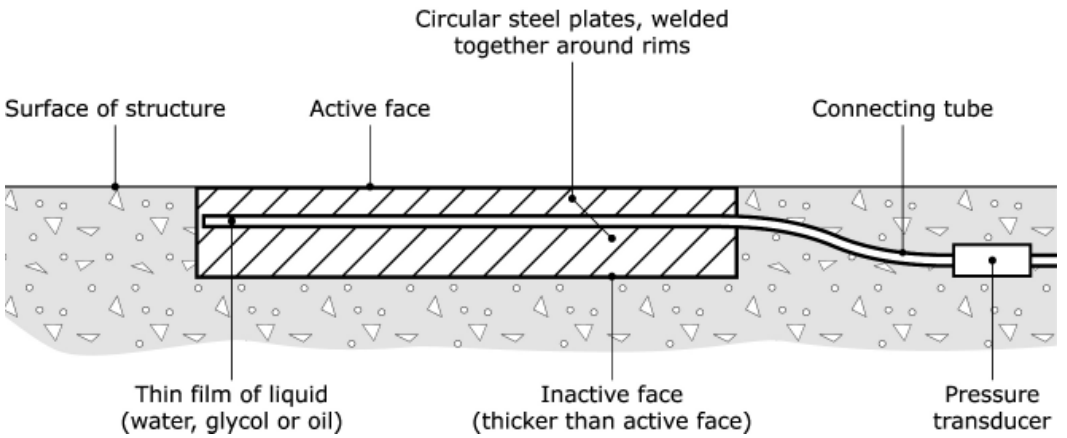


Figure 95.16 Schematic of contact earth pressure cell

95.5.3 Contact earth pressure cells

Measurements of total stress against a structure are not plagued by so many of the errors associated with measurements within a soil mass, and it is possible to measure total stress at the face of a structural element with greater accuracy than within a soil mass.

Figure 95.16 shows a typical contact earth pressure cell. This is similar to the embedment cell shown in **Figure 95.15**, with one of the active faces replaced by a thick inactive face. Cell diameters are the same as for embedment cells.

95.6 General role of instrumentation, and summaries of instruments to be considered for helping to provide answers to various geotechnical questions

95.6.1 Introduction

Instrumentation that may be considered for various types of construction projects is described in the following sections. For each type of construction project the general role of geotechnical instrumentation is given. This is followed by a tabular listing of the possible geotechnical questions that may lead to the use of instrumentation, indicating some of the types of instruments can be considered for helping to provide answers to those questions. It is recognised that there may be additional geotechnical questions and also additional instruments that are not described in this chapter.

The sequence of geotechnical questions is intended to match the time sequence in which the question may be addressed during the design, construction and performance process, and does not indicate any rating of importance.

The suggestions for types of instruments are not intended to be dogmatic, because the selection always depends on issues specific to each project, and is influenced by the personal experience of the person making the selection. In the tables some of the most likely instruments to consider are listed, with other possible types in brackets.

95.6.2 Internally braced excavations

95.6.2.1 General role of instrumentation

The design of internally braced (strutted) excavations is based for the most part on empirical procedures and past

experience. The consequences of poor performance can be severe and may on occasion be catastrophic. A monitoring programme may not be required if the design is very conservative, if there is previous experience with design and construction of similar facilities under similar conditions, or if the consequences of poor performance will not be severe. However, under other circumstances a monitoring programme will normally be required to demonstrate that the excavation is stable and that nearby structures are not affected adversely. Depending on the specific needs of each case, the monitoring programme may apply to the wall and struts, to the ground beneath or surrounding the excavation and/or to adjacent structures or utilities.

95.6.2.2 Summary of instruments to be considered for helping to provide answers to various geotechnical questions

Table 95.1 lists the possible geotechnical questions that may lead to the use of instrumentation for internally braced excavations, together with possible instruments that can be considered for helping to provide answers to those questions.

Some geotechnical questions	Measurement	Some instruments to consider
What are the initial site conditions?	Groundwater pressure	Open standpipe piezometers
		Vibrating wire piezometers installed by the fully grouted method
		(Pneumatic piezometers)
	Vertical deformation	Surveying methods
	Widths of cracks in structures	Crack gauges
Are the struts being installed correctly?	Load in struts	Calibrated hydraulic jacks
Is the excavation stable, and are nearby structures being affected adversely by ground movements?	Settlement of ground surface, structures and top of supporting wall	Surveying methods
	Horizontal deformation of ground surface, structures, and exposed part of supporting wall	Surveying methods
		(Convergence gauges)
	Change in width of cracks in structures and utilities	Crack gauges
	Subsurface horizontal deformation of ground	Inclinometers
		In-place inclinometers
		(Fixed borehole extensometers)
		(Fibre-optic instruments)
	Subsurface settlement of ground and utilities	Probe extensometers
		(Fixed borehole extensometers)
	Load in struts	Surface-mounted strain gauges
	Groundwater pressure	Open standpipe piezometers
		Vibrating wire piezometers installed by the fully grouted method
		(Pneumatic piezometers)
	Base heave	Probe extensometers
Is an individual strut being overloaded?	Load in strut	Surface-mounted strain gauges
Is the groundwater table being lowered?	Groundwater pressure	Open standpipe piezometers
		Vibrating wire piezometers installed by the fully grouted method
		(Pneumatic piezometers)
Is excessive base heave occurring?	Base heave	Probe extensometers
	Subsurface horizontal deformation	Inclinometers
		In-place inclinometers

Table 95.1 Some instruments to consider for monitoring internally braced excavations

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95.6.3 Externally braced excavations

95.6.3.1 General role of instrumentation

The general role of instrumentation for externally braced excavations is the same as for internally braced excavations. However, it is possible to make regular visual inspections of internal bracing, but external bracing cannot be *seen*. Although confidence in the performance of an externally braced excavation is increased by conducting a proof test on every tieback anchor, if an anchor subsequently fails, the failure may be progressive and catastrophic. In general, therefore, instrumentation plays a role in three phases of external bracing that are not applicable to internal bracing: testing of *test anchors* during the design phase or at the start of construction, as input to design of the project anchors; *performance* and *proof testing* of anchors during construction; and subsequent *monitoring* of selected representative anchors. The third phase may be omitted if a conservative design has been used.

95.6.3.2 Summary of instruments to be considered for helping to provide answers to various geotechnical questions

Table 95.2 lists the possible geotechnical questions that may lead to the use of instrumentation for externally braced excavations, together with possible instruments that can be considered for helping to provide answers to those questions.

95.6.4 Embankments on soft ground

95.6.4.1 General role of instrumentation

This section relates to the use of geotechnical instrumentation where all the geotechnical questions are associated with the soft ground itself, and not with the embankment.

In many cases, selection of soil parameters for the foundation soil is reliably conservative. The embankment is therefore designed with confidence that performance will be satisfactory, and 'comfortable' factors of safety are used. In such cases, many projects will proceed without the use of instrumentation. However, some uncertainties always exist. Where design uncertainties are great, factors of safety small, or the consequences of poor performance severe, a prudent designer will include a performance monitoring programme in the design.

In spite of a long record of embankment construction throughout the history of civil engineering, embankments that are designed with a factor of safety greater than unity fail embarrassingly often. On the other hand, some test embankments that are designed to fail intentionally, never do. Thus, it is not surprising that instrumentation plays a significant role in design and construction of embankments on soft ground.

The most frequent uses of instrumentation for embankments on soft ground are to monitor the progress of consolidation and to determine whether the embankment is stable. If the calculated factor of safety is likely to approach unity, instrumentation will generally be installed to provide a warning of any instability, thereby allowing remedial measures to be implemented before critical situations arise.

Chapter 70 *Design of new earthworks* provides additional information on embankments on soft ground.

95.6.4.2 Summary of instruments to be considered for helping to provide answers to various geotechnical questions

Table 95.3 lists the possible geotechnical questions that may lead to the use of instrumentation for embankments on soft

Some geotechnical questions	Measurement	Some instruments to consider
What are the initial site conditions?	As in **Table 95.1**	As in **Table 95.1**
What is a suitable design for tieback anchors (by constructing and testing *test anchors*)?	Load in tieback	Load cells
	Deformation at head	Dial indicators
	Load transfer in grouted zone	Surface-mounted strain gauges
Are the tiebacks being installed correctly (by *performance* and *proof testing*)?	Load in tieback	Calibrated hydraulic jacks (Load cell)
	Deformation at head	Dial indicators
Is the excavation stable, and are nearby structures being affected adversely by ground movement?	As in **Table 95.1**, except for load in struts	As in **Table 95.1**, except for load in struts
	Load in tieback	Load cells (Calibrated hydraulic jacks and load cells: *lift-off tests*) Surface-mounted strain gauges
Is the groundwater table being lowered?	As in **Table 95.1**	As in **Table 95.1**
Is excessive bottom heave occurring?	As in **Table 95.1**	As in **Table 95.1**

Table 95.2 Some instruments to consider for monitoring externally braced excavations

Some geotechnical questions	Measurement	Some instruments to consider
What are the initial site conditions in the soft ground?	Pore water pressure	Vibrating wire piezometers installed by the fully grouted method
		(Open standpipe piezometers)
		(Pneumatic piezometers)
	Vertical deformation	Surveying methods
Is the embankment stable?	Horizontal deformation	Surveying methods
		Inclinometers
		(In-place inclinometers)
What is the progress of consolidation of the soft ground?	Vertical deformation of embankment surface and ground surface at and beyond toe of embankment	Surveying methods
	Vertical deformation of original ground surface below embankment	Probe extensometers
		(Single-point and full-profile liquid level gauges)
		(Settlement platforms)
		(Horizontal inclinometers)
	Vertical deformation and compression of subsurface	Probe extensometers
	Pore water pressure	Vibrating wire piezometers installed by the push-in method

Table 95.3 Some instruments to consider for monitoring embankments on soft ground

ground, together with possible instruments that can be considered for helping to provide answers to those questions.

95.6.5 Clay embankments

95.6.5.1 General role of instrumentation

This section relates to the use of geotechnical instrumentation where the primary geotechnical questions are associated with the embankment, and not necessarily with the ground below the embankment.

All clay embankments begin life with negative pore water pressures, which should provide enhanced stability. Increases of the pore water pressure in the fill could lead to the development of instability. In most cases such failures will occur shortly after construction. Longer-term failures can occur in embankments that have an otherwise stable geometry, caused by progressive deformations induced by shrinkage and swelling that result from the presence of vegetation. It is these that give most cause for concern. Seasonal variations of pore water pressure giving rise to shrinkage and swelling also cause problems with the serviceability of clay embankments, particularly where they carry railway infrastructure. Identifying the presence of seasonal pore water pressure changes or shrinkage and swelling is therefore an important factor in determining the need for further assessment.

Once identified, the magnitude of the stress changes can be used in an analysis of the embankment, to determine if a failure surface is likely to develop. However, it is important to recognise that the analysis will be imprecise in its ability to predict the timing of a failure because the seasonal changes will not be consistent from year to year, and the history of stress changes will be unknown. Therefore long-term monitoring of the pore water pressures is likely to be required and suitable instrumentation should be selected. Inclinometers can be used to detect acceleration in the movement along discontinuities near the toe of the embankment, but these measurements must also be long-term.

Ultimate stability will be influenced by the underlying foundation. Low permeability soils could prolong the increase of pore water pressures within the embankment during extreme environmental conditions. Higher permeability soils will provide underdrainage and help to maintain lower pore water pressures within the embankment. Deep open standpipe piezometers are therefore useful for identifying the underlying groundwater regime.

95.6.5.2 Summary of instruments to be considered for helping to provide answers to various geotechnical questions

Table 95.4 lists the possible geotechnical questions that may lead to the use of instrumentation for clay embankments,

Some geotechnical questions	Measurement	Some instruments to consider
Is the embankment stable?	Negative pore water pressure	Flushable piezometers
	Horizontal deformation	Surveying methods
		Inclinometers
		(In-place inclinometers)
What is the underlying drainage condition?	Pore water pressure	Open standpipe piezometers
Is there any shrink swell?	Pore water pressure	Flushable piezometers
	Horizontal deformation	Inclinometers
		(In-place inclinometers)
	Vertical deformation	Probe extensometers
		Surveying methods

Table 95.4 Some instruments to consider for monitoring clay embankments

together with possible instruments that can be considered for helping to provide answers to those questions.

95.6.6 Embankment dams

95.6.6.1 General role of instrumentation

The main purpose of instrumentation installed within an embankment dam is to study whether or not the dam is behaving according to design predictions. This general statement can be subdivided into two categories: first, the study of special problems at individual sites that are related to special foundation conditions or uncommon design features and second, the study of behaviour when there are no such special problems.

When instrumentation is used for studying special problems that are related to special foundation conditions or uncommon design features, the design of the monitoring programme is tuned directly for the special conditions and features and, because the designers of the dam know the weaknesses of the particular site and the sensitive features of the design, they must play a leading role in the choice of type and location of instruments.

The practice of placing instruments routinely in embankment dams where there is no special problem to be studied, for example, a conventional dam of moderate height on a good foundation, has seen several ups and downs in the last fifty years. In the 1960s, many important dams were constructed with essentially no internal instruments. Each failure or major problem tends to be widely publicised, and dam engineers feel pressure to install instruments routinely for their own protection, even if they do not believe it absolutely necessary.

Peck (1985) comments:

> Instrumentation, vital for obtaining quantitative answers to significant questions, is too often misused, especially in earth and rockfill dams. In some countries regulations concerning the safety of dams demand the incorporation of inclinometers, settlement indicators, and piezometers in the cores of virtually all new dams, but for what purpose? Not for research, because the patterns of deformation and pore pressure development for ordinary geometries and materials are now well known and can be predicted by calculation. Only under unusual circumstances can it be said that design assumptions in these regards require verification. Yet, installation of instruments, even under the best of circumstances, introduces inhomogeneities into the cores, and occasionally is the direct cause of such local defects as sinkholes. The potential weakness introduced by an installation should be balanced against the potential benefit from the observations. In contrast to those located in cores, piezometers in foundation materials near the downstream toes detect upward seepage pressures that cannot be predicted reliably, and can thus give timely warning if measures are needed to ensure safety. There is a danger that instrumentation may be discredited because of indiscriminate use. Of all the internal instruments in embankment dams, piezometers are by far the most common, and an installation consisting only of piezometers can serve two purposes. First, piezometers in a clay core indicate rates of dissipation of pore water pressure and the approach of equilibrium conditions after impounding. Second, piezometers may be appropriate where it is desired to avoid criticism when designing a dam of conventional design and moderate height on a good foundation.

Although written in 1985, these comments are still applicable today.

Peck (2006) puts instrumentation into perspective: 'Monitoring of every dam is mandatory, because dams change with age and may develop defects. There is no substitute for systematic and intelligent surveillance. But monitoring and surveillance are not synonymous with instrumentation.'

95.6.6.2 Instruments to be considered for helping to provide answers to various geotechnical questions

Among the possible geotechnical questions are:

- What are the initial site conditions?
- Is performance satisfactory during construction?
- Is performance satisfactory during first filling?
- Is performance satisfactory during drawdown?
- Is long-term performance satisfactory?

The types of instruments to be considered for helping to provide answers to these questions are too numerous to be tabulated here, and readers are referred to Dunnicliff (1988, 1993). The most important methods for addressing the last question are visual observations of the entire structure by appropriately trained people and monitoring of leakage emerging downstream by using leakage weirs. Monitoring methods that have been developed or improved since the above reference was written, and which may be applicable for addressing the last question, include global positioning systems, distributed fibre-optic sensing and robotic total stations for monitoring deformation. Distributed fibre-optic sensors also have large potential for monitoring the temperature of seepage water (Inaudi and Glišić, 2007b), thereby indicating areas where water flow is concentrated.

95.6.7 Cut slopes in soil

95.6.7.1 General role of instrumentation

It is imperative that, prior to planning an instrumentation programme for a cut slope in soil, an engineer first develop one or more working hypotheses for a potential behaviour mechanism. The hypotheses must be based on a comprehensive knowledge of the locations and properties of stratigraphic discontinuities.

Instrumentation can be used to define the groundwater regime prior to excavating a slope. Results of measurements during excavation can be used as a basis for modification of the designed slope angle. Measurements of ground movement and positive or negative groundwater pressure can assist in documenting whether or not performance during and after excavation is in accordance with predicted behaviour. Measurements can also be used to document whether short- and long-term surface and/or subsurface drainage measures are performing effectively. If evidence of instability appears during or after construction, instrumentation plays a role in defining the characteristics of the instability, thus permitting selection of an appropriate remedy.

A very important subset is the case of a cut slope in clay. Here negative pore water pressures generated during excavation can give rise to temporary stability, the lifetime of which will be related to the height of the slope and the slope angle. Therefore monitoring the negative pore water pressures is an effective way of assessing the stability of a cut slope in clay. In some instances the stability may be maintained for long enough to undertake temporary works within the excavation and thereby save on expensive stabilisation measures.

Chapter 72 *Slope stabilisation methods* provides additional information on cut slopes in soil.

95.6.7.2 Summary of instruments to be considered for helping to provide answers to various geotechnical questions

Table 95.5 lists the possible geotechnical questions that may lead to the use of instrumentation for cut slopes in soil, together with possible instruments that can be considered for helping to provide answers to those questions.

95.6.8 Landslides in soil

95.6.8.1 General role of instrumentation

If there is evidence of slope instability, its characteristics must be defined so that any necessary remedial measures may be taken. The question *how much ground is moving?* can be answered by use of instrumentation. The question *why is the ground moving?* will not be answered by instrumentation alone: the answer of course also requires a complete geotechnical investigation and analysis. Instrumentation also plays a role in monitoring the long-term stability of the slope after remedial measures have been taken.

Chapter 72 *Slope stabilisation methods* provides additional information on landslides in soil.

95.6.8.2 Summary of instruments to be considered for helping to provide answers to various geotechnical questions

Table 95.6 lists the possible geotechnical questions that may lead to the use of instrumentation for landslides in soil, together with possible instruments that can be considered for helping to provide answers to those questions.

95.6.9 Cut slopes in rock

95.6.9.1 General role of instrumentation

The general role of instrumentation is identical to the role for cut slopes in soil, as discussed above. However, when planning to monitor the stability of rock slopes, it is important to recognise that if the slope is subject to a brittle failure mode, movement will be sudden. In such cases, geotechnical instrumentation may not be appropriate to forewarn of instability. It may be more appropriate to develop an area-wide correlation between rainfall intensity and slope instability, and to use rainfall measurements to warn of potential problems.

Chapters 72 *Slope stabilisation methods* and 87 *Rock stabilisation* provide additional information on cut slopes in rock.

95.6.9.2 Summary of instruments to be considered for helping to provide answers to various geotechnical questions

Table 95.7 lists the possible geotechnical questions that may lead to the use of instrumentation for cut slopes in rock, together with possible instruments that can be considered for helping to provide answers to those questions.

95.6.10 Landslides in rock

95.6.10.1 General role of instrumentation

The general role of instrumentation is identical to the role for landslides in soil, as discussed above.

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Some geotechnical questions	Measurement	Some instruments to consider
What are the initial site conditions?	Pore water pressure	Open standpipe piezometers
		Vibrating wire piezometers installed by the fully grouted method
		Flushable piezometers
		(Pneumatic piezometers)
	Surface deformation	Surveying methods
		(Tiltmeters)
		(Fibre-optic instruments)
		(Global positioning system)
	Subsurface deformation	Inclinometers
		In-place inclinometers
		(Time domain reflectometry)
		(Fibre-optic instruments)
Is the slope stable during excavation?	Surface deformation	Surveying methods
		(Tiltmeters)
		(Time domain reflectometry)
		(Fibre-optic instruments)
		(Global positioning system)
	Subsurface deformation	Inclinometers
		In-place inclinometers
		(Time domain reflectometry)
		(Fibre-optic instruments)
	Pore water pressure	Vibrating wire piezometers installed by the fully grouted method
		Flushable piezometers
Is the slope stable in the long term?	As for 'Is the slope stable during excavation?'	As for 'Is the slope stable during excavation?'
	Rainfall, for possible correlation with any deformation	Rain gauges
	Load in tiebacks	Load cells

Table 95.5 Some instruments to consider for monitoring cut slopes in soil

95.6.10.2 Summary of instruments to be considered for helping to provide answers to various geotechnical questions

Table 95.8 lists the possible geotechnical questions that may lead to the use of instrumentation for landslides in rock, together with possible instruments that can be considered for helping to provide answers to those questions.

95.6.11 Tunnels

95.6.11.1 General role of instrumentation

The consequence of poor performance of a tunnel can be severe and may on occasion be catastrophic. A monitoring programme may not be required if the design is very conservative, if there is previous experience with design and construction of similar facilities under similar conditions, or if the consequences of poor performance will not be severe. However, under other circumstances a monitoring programme will normally be required to demonstrate that the tunnel is stable and that nearby structures are not affected adversely.

95.6.11.2 Summary of instruments to be considered for helping to provide answers to various geotechnical questions

Table 95.9 lists the possible geotechnical questions that may lead to the use of instrumentation for tunnels, together with possible instruments that can be considered for helping to provide answers to those questions.

Some geotechnical questions	Measurement	Some instruments to consider
What are the post-landslide conditions?	Pore water pressure	Open standpipe piezometers
		Vibrating wire piezometers installed by the fully grouted method
		Flushable piezometers
		(Pneumatic piezometers)
	Surface deformation	Surveying methods
		(Tiltmeters)
		(Fibre-optic instruments)
		(Global positioning system)
	Subsurface deformation	Inclinometers
		In-place inclinometers
		Time domain reflectometry
		(Fibre-optic instruments)
Is the slope stable in the long term?	As for 'What are the post-landslide conditions?'	As for 'What are the post-landslide conditions?'
	Rainfall, for possible correlation with any deformation	Rain gauges
	Load in tiebacks	Load cells

Table 95.6 Some instruments to consider for monitoring landslides in soil

95.6.12 Driven piles

95.6.12.1 General role of instrumentation

The subsurface length of a driven pile cannot usually be inspected after driving; thus, its physical condition and alignment are unknown. Subsurface geotechnical conditions are rarely known with certainty, and therefore the design of driven piles involves assumptions and uncertainties that are often addressed by conducting instrumented full-scale tests. Tests may examine the behaviour of the pile under load applied to the pile head or under load caused by settlement of soil with respect to the pile.

Defects in piles can be created during driving, and inspection procedures are available for examining the condition and alignment after driving. Certain types of driven pile cause large displacements and changes of pore water pressure in the surrounding soil, and these may in turn have a detrimental effect on neighbouring piles or on the stability of the site as a whole. Instrumentation can be used to quantify the consequences of pile driving and thus to assist in planning any necessary action.

The following chapters provide additional information on driven piles: Chapter 54 *Single piles*, 55 *Pile-group design*, 81 *Types of bearing piles* and 82 *Piling problems.*

95.6.12.2 Summary of instruments to be considered for helping to provide answers to various geotechnical questions

Table 95.10 lists the possible geotechnical questions that may lead to the use of instrumentation for driven piles, together with possible instruments that can be considered for helping to provide answers to those questions.

95.6.13 Bored piles

95.6.13.1 General role of instrumentation

Many uncertainties exist during design of bored piles, and instrumentation plays a role in determining the load–movement relationship by conducting load tests. Concrete integrity is often uncertain during construction, particularly when bored piles are constructed in granular soils below the water table or in softer, squeezing clays, when concrete slump is inadequate, or when concrete placement practices are inferior. Instrumentation can be used to examine the integrity of the concrete. For piles cast under support fluid, concrete integrity at the pile toe is particularly important.

95.6.13.2 Summary of instruments to be considered for helping to provide answers to various geotechnical questions

Table 95.11 lists the possible geotechnical questions that may lead to the use of instrumentation for bored piles, together with possible instruments that can be considered for helping to provide answers to those questions.

95.7 Acknowledgement

The author is greatly indebted to Andrew Ridley BSc (Hons) MSc DIC PhD, Managing Director, Geotechnical Observations Limited, Weybridge, Surrey for the text on the description and applications of flushable piezometers for measuring negative pore water pressure.

Some geotechnical questions	Measurement	Some instruments to consider
What are the initial site conditions?	Joint water pressure	Open standpipe piezometers
		Vibrating wire piezometers installed by the fully grouted method
		(Pneumatic piezometers)
	Surface deformation	Surveying methods
		Crack gauges
		(Tiltmeters)
		(Fibre-optic instruments)
		(Global positioning system)
	Subsurface deformation	Fixed borehole extensometers
		In-place inclinometers
		(Acoustic emission monitoring)
		(Time domain reflectometry)
		(Fibre-optic instruments)
Is the slope stable during excavation?	Surface deformation	Surveying methods
		Crack gauges
		(Tiltmeters)
		(Time domain reflectometry)
		(Fibre-optic instruments)
		(Global positioning system)
	Subsurface deformation	Fixed borehole extensometers
		In-place inclinometers
		(Acoustic emission monitoring)
		(Time domain reflectometry)
		(Fibre-optic instruments)
	Joint water pressure	Vibrating wire piezometers installed by the fully grouted method
Is the slope stable in the long term?	As for 'Is the slope stable during excavation?'	As for 'Is the slope stable during excavation?'
	Rainfall, for possible correlation with any deformation	Rain gauges
	Load in tiebacks	Load cells

Table 95.7 Some instruments to consider for monitoring cut slopes in rock

Some geotechnical questions	Measurement	Some instruments to consider
What are the post-landslide conditions?	As in **Table 95.7** for 'What are the initial site conditions?'	As in **Table 95.7** for 'What are the initial site conditions?'
Is the slope stable in the long term?	As in **Table 95.7** for 'Is the slope stable in the long term?'	As in **Table 95.7** for 'Is the slope stable in the long term?'
	Rainfall, for possible correlation with any deformation	Rain gauges
	Load in tiebacks	Load cells

Table 95.8 Some instruments to consider for monitoring landslides in rock

Some geotechnical questions	Measurement	Some instruments to consider
What are the initial site conditions?	As in **Table 95.1**	As in **Table 95.1**
Is the tunnel stable, and are overlying structures being affected adversely by ground movements?	Settlement of ground surface and structures	Surveying methods
	Horizontal deformation of ground surface and structures	Surveying methods
	Change in width of cracks in structures and utilities	Crack gauges
	Subsurface horizontal deformation of ground	Inclinometers
		In-place inclinometers
		(Fibre-optic instruments)
	Subsurface settlement of ground and utilities	Probe extensometers
		Fixed borehole extensometers
	Deformation within tunnel	Surveying methods
		(Fibre-optic instruments)

Table 95.9 Some instruments to consider for monitoring tunnels

Some geotechnical questions	Measurement	Some instruments to consider
What is the load–movement relationship of the pile? (See Chapter 98 *Pile capacity testing*)	Deformation at head	Dial indicators with reference beams
		Wire/mirror/scale
		Surveying methods
	Load at head	Load cell
	Deformation at toe	Telltales
	Stress along pile	Embedment or surface-mounted strain gauges
		(Fibre-optic instruments)
Has the capacity of the pile been reduced by defects caused during driving?	Curvature of pile	Inclinometer
	Condition of pile	Integrity testing (See Chapter 97 *Pile integrity testing*)

Table 95.10 Some instruments to consider for monitoring driven piles

Some geotechnical questions	Measurement	Some instruments to consider
What is the load–movement relationship of the pile? (See Chapter 98 *Pile capacity testing*)	As in **Table 95.10**	As in **Table 95.10**
	Load at toe or in pile	Osterberg load cell
What is the integrity of the concrete?	Condition of pile	Integrity testing (See Chapter 97 *Pile integrity testing*)

Table 95.11 Some instruments to consider for monitoring bored piles

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* Reference can be downloaded from www.geotechnicalnews.com/instrumentation_news.php

95.8.1 Useful websites

Field Measurements in GeoMechanics; www.fmgm.no. The 'Publications' page contains a search function for authors and paper titles from the proceedings of ten international symposia on geotechnical monitoring.
Geotechnical Instrumentation News (GIN) articles. Articles published since 2001 can be downloaded from www.geotechnicalnews.com/instrumentation_news.php. At the end of each year, that year's published articles will be posted on the site.
Papers presented at the American Society of Civil Engineers Symposium on Field Measurements in Geomechanics (FMGM), Boston, 2007; http://scitation.aip.org/dbt/dbt.jsp?KEY=ASCECP&Volume=307&Issue=40940
2011 Symposium on Field Measurements in Geomechanics (FMGM 2011); www.fmgm2011.org

It is recommended this chapter is read in conjunction with

- Chapter 100 *Observational method*
- Section 8 *Construction processes*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

Chapter 96

Technical supervision of site works

Sarah Glover Arup London, UK
Jonathan Chew Arup London, UK

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This chapter explains the need for geotechnical site supervision and the general tasks expected of an engineer responsible for supervising site work. It explains how an engineer's site role can vary depending on the contract form and procurement route and gives guidance to help an engineer prepare for and carry out a technical site role. Particular attention is paid to the documentation process and typical engineer's responsibilities – in general, to health and safety and for the main types of geotechnical site supervision.

Box 96.1 Note on terminology

The titles in this section are used in their general sense, not as contractual terms. Particularly, the term 'resident engineer' (RE) is used to describe the general role of an engineer responsible for independent overviewing of site works.

96.1 Introduction

Fundamentally, all branches of civil engineering culminate in construction. For this reason it is likely that every civil engineer will spend at least some part of his or her career involved in site work. Perhaps more than any other engineering discipline, the role of site supervision in geotechnical engineering is essential to understanding the predesign ground conditions and later verifying construction quality prior to the works being covered.

In the early stages of a project, investigation of the ground and groundwater conditions is required to develop the geological model and parameters for use in design. This work will normally be carried out by a specialist contractor; however, the engineer who also attends the site works will gain firsthand knowledge of the ground conditions over and above that which can be gained through study of logs and laboratory test data alone. The engineer will also gain a thorough appreciation of the site constraints and key ground risks which can then be addressed in his or her design.

Site supervision during the construction phase is usually focused on monitoring the quality and progress of the works and ensuring that the contractor's working methods are in line with the design assumptions. Unlike most other forms of construction, for the majority of geotechnical works it is either very difficult or impossible to check the construction works once completed as they are usually buried in the ground. Problems with geotechnical construction are likely to come to light at a much later stage in the project, for example in the form of cracking in a building with poorly constructed foundations where the defects will only become apparent upon construction of most of the building (and therefore application of most of the load). Investigation of problems and the subsequent remedial works are usually expensive and time consuming, resulting in claims, disputes and general dissatisfaction. While the presence of an engineer on site can never guarantee that all defects are eliminated, their role during construction works can provide a valuable check on workmanship. For the client, the costs involved in correcting defects are usually disproportionately expensive compared to the cost of providing site supervision.

For the engineer, site work can be a very enjoyable and rewarding experience. An engineer that has spent time on site will generally have a better understanding of construction practicalities and the difficulties encountered by contractors. This knowledge of buildability will be invaluable to the engineer throughout the design process and should result in more workable designs and better construction details.

However, site supervision can also be a difficult experience if the resident engineer (RE) is not suitably prepared and supported throughout the site works. For all but the largest projects, the RE may be the only person from their organisation on site and in this case regular communication with the office-based team becomes critical. It is important that the design team provide adequate training and a thorough briefing to the RE before commencement of the site role and that they continue to do so throughout the site works. Additionally, it is important to recognise that supervision of site works requires a different skills set to that developed in the design office, with much more emphasis needed on communication, prompt decision-making and pragmatism.

This chapter aims to give guidance to engineers new to site work on:

- how to prepare for a site role (section 96.3);
- what to expect while on site including common difficulties and guidance for dealing with them (section 96.4);
- the engineer's responsibilities with respect to health and safety law (section 96.5);
- various types of geotechnical site work (sections 96.6: Site investigation, 96.7: Piling and 96.8: Earthworks).

It should also act as an *aide-mémoire* to the more experienced engineer who is preparing to send a junior engineer to site.

It is recognised that today's engineering site roles can vary greatly from a site engineer acting as conduit between design team and site, to an engineer working directly for the client or even a contractor. This chapter specifically provides guidance to engineers responsible for independently reviewing contractor's work. However, much of the guidance applies equally to other engineering site roles. The chapter primarily refers to technical supervision and ensuring technical compliance. It is not intended to give a detailed description of site roles that primarily involve contract administration.

96.2 Reasons for supervision of geotechnical works

96.2.1 General

Supervision of site works should be common practice in geotechnical engineering. It is usually required to ensure that the planned works are carried out in accordance with the contract and as an independent check on the quality of workmanship.

Site supervision is particularly important in geotechnics as in most cases the works are covered and are neither visible nor accessible for checking post-construction. For example, where the design of bored piles relies on a dry pile shaft with a base free from debris it is important to observe the works and to visually check the pile bore prior to concreting (**Figure 96.1**). If the pile does not perform as designed it will be very difficult to check these details retrospectively. Supervision therefore becomes a key quality control and risk management tool that can be used to help eliminate the need for remedial works and their associated disproportionately high costs (refer to Chapter 7 *Geotechnical risks and their context for whole project*).

The provision of site supervision is often a contractual requirement of the engineer. Even if it is not, the engineer generally has a duty to advise the client on the appropriate level of supervision for a particular project. This advice should include the level of expertise and skills of the supervisor and the proportion of time that should be spent on site. It should be based on professional judgement and knowledge of the project and should take account of:

- the complexity of the project;
- the degree to which the design is dependent on good workmanship;



Figure 96.1 Inspecting a pile bore before concreting
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- the extent of work that will be covered up post-construction (and subsequently neither visible nor accessible for checking) and the time frame for it;
- the degree of completeness of the design;
- the method of procurement and the responsibility of the contractor;
- the level of experience of the contractor;
- the engineer's responsibilities as defined in their terms of appointment.

These recommendations should always be provided in writing to the client and the final agreed brief clearly documented.

96.2.2 Core aims of supervision

Regardless of the particular project, most supervisory roles will endeavour to meet one or more of the following core aims.

96.2.2.1 Provide an independent check on quality

The contractor will generally have a duty to ensure their workmanship is of the required quality. To demonstrate this, the contractor should put in place their own quality assurance procedures which will normally involve checking by members of their site team. However, it can be reassuring for the client to have an independent check on the quality of the contractor's work, particularly if the work is unusual or if the contractor is relatively inexperienced. The presence of the engineer should never relieve the contractor of their contractual obligations but a second set of eyes can assist with identifying potential quality issues early and agreeing methods for their resolution. Additionally, the engineer is less likely to be influenced by other external factors such as programme deadlines and can therefore ensure that these factors do not adversely influence the quality of the work.

For the particular case of site investigation work, the engineer will normally need to rely on the results of the investigation in later design work. Observation of the work can identify any problems that may affect the soil parameters (for example

a sticking catch on a standard penetration test (SPT) hammer invalidating the use of correlations with SPT 'N' values) and will give confidence that the related design is based on reliable data. Doubts as to the quality of the data may necessitate an overconservative approach, which will have a knock-on effect on the efficiency of the design and therefore project costs. If the engineer is unaware of problems with the data then this could equally result in an unconservative design and, in an extreme case, potential failures.

96.2.2.2 Ensure compliance with technical requirements

Technical requirements for site works will be communicated to the contractor through the contract documents and will normally comprise a specification and drawings. Through preparation of these documents the engineer should have identified any key design assumptions that could be affected by the contractor's working method, for example, the soil properties required from the fill material in an earthworks operation and the associated method and type of compaction plant.

The contractor will be responsible for ensuring that their work meets the specified technical requirements and should do so through agreed method statements, rigorous checking and record keeping. Again, the presence of an independent resident engineer will provide a check that the technical requirements are being met and that the contractor is working in accordance with their method statements. Problems can, in this way, be identified early, remediated and repeat occurrences eliminated rather than being left to manifest themselves later on in the project when their cause will be more difficult to determine and their resolution could have a major impact on the project.

96.2.2.3 Ensure design-related issues are raised and resolved promptly

Where observations give cause for concern, the RE will be responsible for ensuring that problems are raised promptly by the contractor through the predefined formal channels (usually non-conformance reports). The RE can then liaise with the design team to ensure that responses to the non-conformances and suggested resolutions are tackled and reported back to the construction team in a timely manner. This aspect of the role becomes particularly important towards the end of the site works as it is ideally required that any identified problems are resolved and remediated before the contractor leaves site.

96.2.2.4 Provide communication link between design and construction team

A defining factor of many successful civil engineering projects is clear and timely communication between all of the project parties and particularly the design and construction teams. Part of the RE's role will be to assist the contractor in understanding the contract documents and design intentions and providing clarification where there are questions or conflicts within the documents.

Through review of the contractor's method statements, the RE can help to identify issues and conflicts early to allow optimal time for their resolution. The RE can also provide regular updates to the design team, which can help them to plan and prioritise their activities to meet changing site requirements.

96.2.2.5 Contract administration

For some projects the RE will also be responsible for administration of the construction contract, but this will depend on the form of contract and procurement route used for the particular project. For example, civil engineering projects which use ICE conditions of contract will be administered by the engineer whereas building projects using JCT contract forms will normally be administered by an architect or project manager. For many site investigation projects the engineer will be called upon to provide both a technical and contract administration role on site.

The engineer's duties relating to contract administration should be established and agreed early on in the project.

96.2.3 Additional benefits of site supervision

In addition to carrying out a key project role, the engineer's site experience should improve their skills as a designer.

Through observation of the construction process the engineer will be able to understand which elements of their design work well and which cause practical problems for the contractor. This should help to develop a pragmatic approach and result in the production of more effective and buildable designs in the future. For example, by witnessing the reinforcement cages of bored piles being spliced together on site the engineer will appreciate how the design should consider the density and thickness of the bars to facilitate buildability.

Observation of the number of interrelated activities on site should give the engineer an appreciation of the complexity of coordinating site work which should help with the design of future projects in early feasibility and concept stages (**Figure 96.2**). It will also help in discussions with contractors on future projects.

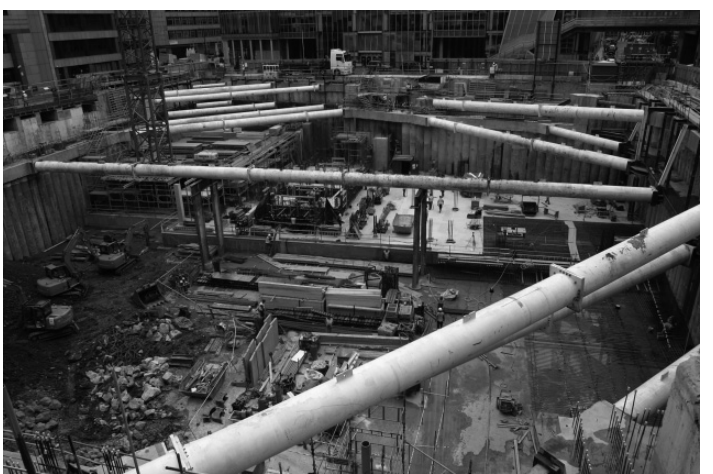


Figure 96.2 Deep basement construction involves a number of interrelated activities

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Through working with the contractor to identify and resolve problems, the engineer will gain valuable experience that should allow those same problems to be eliminated from future designs but also to resolve other problems more quickly and effectively in the future.

Box 96.2 Key points: site supervision

- Site supervision is a key quality control and risk management tool that can be used to help eliminate the need for remedial works.
- The core aims of site supervision are to:
  - provide an independent check on quality;
  - ensure compliance with technical requirements;
  - ensure design-related issues are raised and resolved promptly;
  - provide communication link between design and construction team; and
  - contract administration.
- An engineer's site experience should improve their skills as a designer.

96.3 Preparing for a site role

96.3.1 Importance of preparation

Every site and site role is different and therefore requires a tailored approach based on a clear understanding of the project parties, the procurement route and contract conditions and the particular technical requirements. It is therefore essential that the RE prepares thoroughly for each and every site role so that they can, from the outset, operate effectively with all project parties to fulfil their supervisory obligations.

The first few weeks of site work can often be the most challenging as several project parties, who have not usually worked together before, start to establish working processes and relationships. The RE's time will be filled with carrying out initial site inspections and answering technical queries from the construction team. It is important that the RE has a thorough understanding of the technical requirements of the work so that they can set a clear expectation for the contractor. Once established, this should allow the contractor to settle quickly into a routine and will help to ensure the smooth running of the project.

In addition to setting the technical expectation, it is also essential that the RE establishes clear lines of communication with the various project parties. In order to do this the RE needs to understand the roles and responsibilities of the individuals working on the project and ensure that they in return are clear on those of the RE.

The amount of work required when preparing for a site role should not be underestimated, particularly where the RE is new to the project or to site work in general. It is the responsibility of the design team to ensure that the RE is suitably experienced in the type of fieldwork they are about to supervise or is trained and briefed before starting a site role and always remains well supported throughout. The benefits of this preparation will be felt by all involved in the project.

96.3.2 Defining roles and responsibilities

The roles and responsibilities of the RE should be agreed with the client well in advance of the start of site works. These should be recorded in a clear written statement of brief that is incorporated into the design team's contract.

For some construction contract forms such as the Institution of Civil Engineers 7th edition, the engineer is a named party with specific contractual duties clearly defined in the contract. Other contract forms may refer instead to a supervising officer or contract administrator and may not define the role of the engineer specifically. Under these types of arrangement the role of the contract administrator may be carried out by another project party (for example the architect, project manager or construction manager) and the engineer will usually act as a technical adviser to the contract administrator. In this instance the engineer's role will not be clearly defined in the main contract and it will be necessary for the design team to agree the extent of the technical supervision role (refer to Chapter 78 *Procurement and specification*).

The RE's brief will need to be tailored to suit the procurement route and contract form but should typically describe the following:

- the anticipated duration of the site work and the level of attendance (full time, part time, occasional visits);
- the elements of work that the RE will be responsible for witnessing and those for which the RE will not be responsible;
- the checks that are to be made and the records that must be kept;
- the format, frequency and distribution for progress reports;
- the RE's duties in relation to ensuring that the contractor is carrying out their role satisfactorily;
- the extent of the RE's authority including details of any limitations under the contract;
- the project change management system;
- the procedure for issuing instructions;
- the procedure for responding to requests for information and nonconformance reports;
- the interfaces and lines of communication with other members of the team.

Preparation of this brief will be a useful prompt in defining the boundaries between the RE's responsibilities and those of other project team members. Through the brief it is likely that areas of conflict or split responsibility will be identified and early preparation should enable resolution of areas of conflict prior to start of work on site.

96.3.3 Understanding the construction team

For the RE to operate effectively in the site environment it is important that a good knowledge of the various project parties and their organisation is gained. To develop this understanding it can be useful to draw an organogram of the project team

clearly indicating the lines of communication between the various project parties and summarising their main responsibilities (**Figure 96.3**). This could prove a very simple task, for example where the construction work is procured via a traditional route with the client separately appointing an architect, engineer and main contractor. Or, it could be a more complicated arrangement where there are many more contracts and parties involved in the construction works.

Having established the general project organisation, the RE should be able to identify named individuals with authority to take project decisions, issue instructions and tackle problems. The RE should be able to identify who to communicate with on a day-to-day basis and can then confirm this with the project team.

A major part of the RE's role will normally be to ensure that the contractor fulfils their obligations under the contract. These should be set out clearly in the contract documents for the construction works, which the RE should review and check for compatibility with their own duties.

If, following this assessment of the various site parties' responsibilities, there remain any uncertainties or conflicts, the RE should proactively raise these issues with the project team and ensure that they are resolved prior to start of the site role.

96.3.4 Preparatory work

The RE should be thoroughly briefed on the organisational and technical aspects of the site role by a senior member of the design team (section 96.3.2).

Additionally the design team and RE should gather together a set of base information to refer to while on site. This should include the items in **Checklist 96.1**.

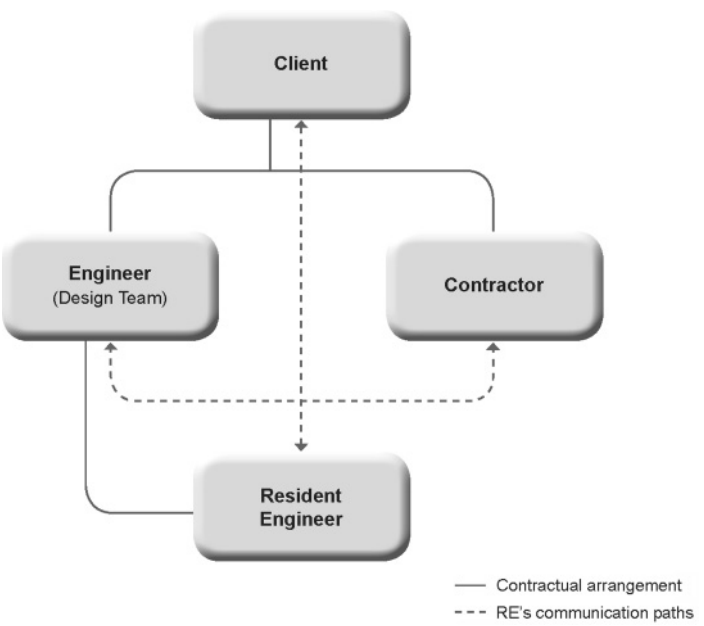


Figure 96.3 Typical lines of communication for a resident engineer under a traditional project arrangement

Generally it is advantageous for the RE to have had some experience of carrying out the design before going to site. However, this is not always possible and therefore the RE should have a good knowledge of the design prior to arriving on site. This will allow the RE to understand the criticality of certain aspects of the design and also reduce their dependence on the design team to resolve technical queries.

It is advisable to involve the RE in the initial review and checking of the contractor's method statements, which will occur prior to start of site work. This will allow the RE to beome familiarised with the site work and to identify the major issues in advance of start of the works. It will also allow the RE to be introduced to the key site parties so that they are already a familiar face before arriving on site.

The RE should insist that the contractor prepares the information required by the contract documents well in advance of the start of site works. These will generally include the items in **Checklist 96.2**.

The RE should ensure that information received is complete, and that their comments are issued promptly and addressed by the contractor.

Checklist 96.1 Preparatory documents

- ✓ The written statement of brief including the schedule of RE duties
- ✓ An overview of the project
- ✓ The design basis and, if appropriate, calculations
- ✓ Identification of critical elements of design that require particular checking procedures
- ✓ Specifications for the elements of the work for which the RE is responsible
- ✓ Construction drawings
- ✓ Proforma check sheets for each element of the work that requires inspection
- ✓ Method statements, inspection and test plans and fabrication drawings prepared by the contractor and agreed with the project team
- ✓ The project programme
- ✓ Health and safety documentation and records of briefings (refer to section 96.5)
- ✓ Copies of relevant codes of practice, handbooks and guidance notes
- ✓ Contact details for key project individuals

Checklist 96.2 Initial information prepared by contractor

- ✓ Programme and sequence of work
- ✓ Site organisation chart
- ✓ Method statements
- ✓ Inspection and testing plan
- ✓ Typical proforma sheets for inspection requests, contractor's submissions, requests for information, non-conformance reports

Box 96.3 Key points: preparatory work

- Thorough preparation is essential to a successful site role.
- A clear brief and schedule of duties will help the RE understand their site role.
- The RE should undertake appropriate training prior to start on site.
- The RE needs to understand the roles and responsibilities of others working on the project and know who to communicate with on a day-to-day basis.
- The RE should ideally be involved in the project for some time prior to the start of a site role and have experience of the type of works they will be responsible for.

96.4 Managing the site works

96.4.1 Setting up on site

The RE should aim to arrive on site slightly in advance of the start of site works to allow time to get organised before the works begin in earnest. This time should be occupied with setting up on site and making contact with key site individuals. Importantly, the RE should check that all health and safety requirements are in place and, if this is not the case, these should be addressed as a priority (section 96.5).

The contractor will normally be required to provide office space for the RE. This may be separate from or (as is becoming more common) contained within the contractor's own site office. The contract documents should contain a list of the facilities the contractor must provide, but these will normally include:

- electric power;
- lighting;
- air conditioner/heater;
- desk with lockable drawers;
- chair;
- filing cabinet and/or bookshelves.

The contractor is also legally obliged to provide adequate welfare provisions. If reasonable welfare facilities are not provided then the contractor should rectify this immediately.

The RE will normally be responsible for arranging provision of other facilities that are needed, for example a laptop, printer and phone, internet access and personal protective equipment (PPE). The RE should also ensure that the relevant project documents and reference material are taken to site so that they are instantly accessible (section 96.3.4 and summarised in **Figure 96.4**).

96.4.2 Documentation and record keeping

The importance of clearly documented, timely and well-organised site records must not be underestimated. Comprehensive and accurate site records can contribute significantly to the smooth running of a project and can be used to demonstrate that the RE is carrying out their role effectively. Good record keeping can also become critically important in the event of claims, particularly when these are raised long after the project has been completed. For example, if a pile integrity test is undertaken many days after a pile has been completed and shows the pile to have inclusions then detailed site records will be essential in understanding the cause of the problem and informing the subsequent investigation and remedial works design.

The volume of records that will be generated for even a short site role can be significant. For this reason, it is imperative that the RE is well organised and establishes a workable system that allows information to be stored neatly and in such a way that it can be found again quickly. Some projects may have clearly defined processes for doing this but if not, the RE should choose a method that suits the particular project and their style of working.

The RE will normally be responsible for both keeping and maintaining their own records and for collating and commenting on those produced by the contractor. **Figure 96.4** provides a schedule of the typical records that should be collated by the RE before, during and at the end of the site works in addition to the parties that would normally prepare the records. Possibly the RE's most important record is their diary. This should be written daily and should include the information listed in **Figure 96.4**.

Project changes can often be the cause of confusion and problems on site, particularly if these need to be justified retrospectively. It is essential the RE clarifies the project change procedure prior to starting the works and follows it precisely should a change be needed. On some projects the change procedure will be clearly defined; however, on other projects it can be useful for the RE to prompt the site and design teams to clarify the change procedure, prior to the site works commencing. For example, if during a site investigation it is necessary to introduce an additional borehole to provide clarity on conflicting information, the RE should follow the project change procedure and clearly state the reason for the addition so that payment can be easily certified later in the work when the contractor submits the application for payment.

96.4.3 Inspection and checking

Probably the most important aspect of the RE's role is the witnessing and selective checking of the contractor's work. The RE should ensure that checks are carried out rigorously and that each check is recorded by the contractor and separately by the RE using the pre-agreed proforma check sheets. It is important not to get behind on this paperwork as it can be difficult or impossible to fill out check sheets even a short time after the check has been carried out.

The contractor should normally be present at the time the RE carries out their checks. If the RE is unhappy with any element of the check they should raise it with the contractor immediately to give the latter an opportunity to rectify the problem before the work is completed. In the unlikely instance that the contractor refuses to rectify the defect then the RE

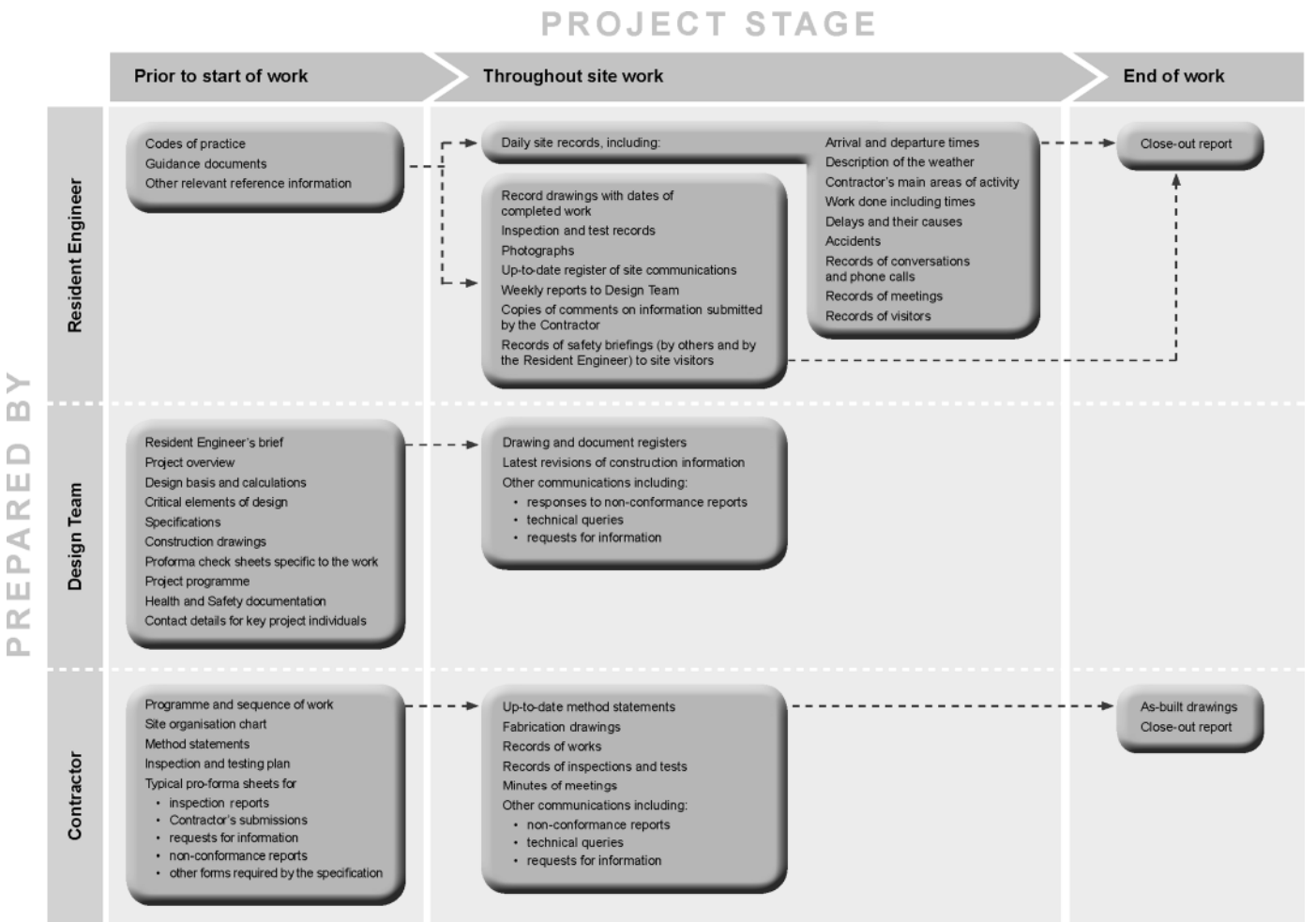


Figure 96.4 Schedule of site work documentation to be collated by the resident engineer

should make the contractor aware of the consequences of not rectifying the defect and then follow up this conversation in writing as soon as possible after the event.

The contractor will normally provide their own records of the works, copies of which should be sent to the RE in addition to the contract administrator. The RE should check the contractor's records for compatibility with their own and raise any concerns or incompatibilities with the contractor and the project team. In advance of start of site works the RE should obtain proforma check sheets from the contractor to check that these cover the elements required by the contract documents.

Precise details of the checks to be made, their frequency and the records to be taken will depend on the nature of the work being inspected and the RE's brief.

96.4.4 Communication

This will depend upon the form of procurement and the construction contract. However, the following sections contain general advice that can be modified to suit the particular project requirements.

96.4.4.1 Communicating with the contractor

The RE's most frequent contact will be with the contractor. Communication with the contractor will range from verbal discussions to formal written site instructions and, at first, it may be difficult to judge the most appropriate means of communication for a given situation.

The RE's first aim should be to establish a good working relationship with the contractor's staff. In doing this the RE must recognise that the contractor's staff will be working to time and cost deadlines; that their jobs may depend on the project's progress and profitability, and that in these circumstances the contractor may seek cost-driven, rather than quality-driven, solutions. However, while the RE should acknowledge that the contractor is in business to make a profit they should also be clear that the contractor is obliged to work within the contract terms.

In all of their site dealings, the RE should aim to adopt a fair, firm and consistent approach. Where the RE observes something going wrong they should raise the issue as early as possible and should not wait until it is completed and they are

formally invited to inspect it. The RE should try not to reverse earlier decisions or to find further faults in work that they have already accepted. In this way the RE will gain increasing respect and authority on site.

It is usual for the RE to have daily discussions with the contractor and these discussions should be summarised in the RE's site diary. If the discussions are informal, such as comments during site inspections or suggestions for improved working, then it should not be necessary to record and circulate them more formally provided the contractor acts on them. The RE should give comments only to pre-identified individuals on the contractor's team and not directly to the contractor's site operatives, or to their sub-contractors.

Where discussions are more formal the RE should send a written record to the contractor's team and distribute it to other affected site parties. These communications should use a pre-agreed form such as a site memo. Examples of when a written record may be required are: when the contractor has not responded to the RE's verbal comments during inspections; when safety matters are ignored; and when requesting written information such as method statements.

Site instructions should be reserved for formal contractual matters such as: issuing sketches and information for construction; instructing the contractor to rectify breaches of the specification; or dealing with safety issues (when the initial correspondence has not received a response); or for requiring the contractor to issue remedial work proposals. Remember that site instructions usually need confirming by the party acting as contract administrator. To aid the understanding of all parties involved, the RE should clearly state the reason for the issue of an instruction as this information can be of use later. If the RE is uncertain whether a site instruction is warranted, they should discuss the matter with their design team.

It is important to recognise that there is no such thing as a 'verbal instruction'. Any instruction given by the RE must be confirmed in writing so that it is on record.

The contractor will send the RE many communications including method statements, requests for inspection, requests for information, site memos and non-conformance reports. The RE should insist that all requests are in writing and that they follow the agreed site procedures. The RE will normally be contractually obliged to respond to each communication within a defined time period and should ensure that this period is known and adhered to. All responses should be in writing. In instances where a large number of requests are received, the RE may find it useful to set up a register to keep a check on which have been answered. The RE will need to liaise well with other members of the design team to seek input where required and ensure a coordinated response.

96.4.4.2 Communicating with the design team

For the benefit of all, the RE will need to keep in close contact with their design team. The RE should speak regularly to a designated member of the design team to give details of site progress, to pass on queries and to receive information. In the early days of a project it is a good idea to make contact at least once a day, even if there is little to report. The RE should make a brief note in their diary to record these communications.

It is a good idea for members of the design team to visit the site, especially at the beginning of the works. This will provide support to the RE and will give the design team an appreciation of the site conditions.

The RE should also provide written summaries of site events. The format and frequency of reporting should be agreed in advance with the design team. For most projects a weekly report summarising the following issues should be sufficient:

- project progress against programmed progress;
- major problems and delays;
- technical issues encountered and their resolution;
- instructions issued;
- communications received and responses outstanding;
- test results received;
- non-conformances identified.

The RE should ensure that the design team is clear on the information that is required on site and that a designated person is responsible for ensuring that the RE is supplied with the latest versions of specifications and drawings in a timely manner. Additionally, the design team will be responsible for keeping the RE apprised of project changes that could affect their work. Increasingly, web-based systems are being used to manage this documentation process and the RE must have the means to access such systems efficiently while on site and be provided notification of new uploads.

The RE should not be afraid to ask the design team for advice, especially in the early days on site. As the RE becomes more familiar with site work, they will become more confident and independent; however, they should recognise that in unfamiliar or unusual situations they should seek clarification from the design team.

96.4.4.3 Communicating with other parties

Depending on their particular brief, the RE may be required to interface with other site parties such as contract administrators, project managers, quantity surveyors, clerks of works, etc. The level and form of communication will vary from project to project and should be clearly defined prior to the start of site work.

While the RE should always be reasonably helpful and informative, it is important to ensure that the level of communication provided is appropriate to the role and authority of the party. Particularly, the RE should only accept instructions from parties that have authority to issue them.

Much of the communication with other site parties will be verbal but should always be recorded in site diaries or by more formal means if deemed necessary.

96.4.5 Site meetings

The contract administrator should arrange regular site meetings with the contractor, to review progress and discuss problems. These should be attended by the RE and representatives of the client and design team should also be invited.

A typical agenda for the meeting should as a minimum include:

- progress against programme and issues delaying progress;
- design-related issues;
- status of requests for information;
- status of non-conformance reports;
- problems encountered and means for resolution;
- health and safety issues;
- proposed works before the next meeting;
- any other business.

Minutes should be prepared for all meetings and these should be distributed to the attendees as soon as possible and certainly before the time of the next meeting. The RE should thoroughly review all minutes to ensure that they are a fair representation of the meeting. Any issues with the minutes should be raised at the start of the next meeting, or earlier if the need dictates. The RE should ensure that action items identified in the minutes are dealt with promptly.

In advance of the meeting the RE should make a note of points they wish to raise such as quality of work, matters of safety, and information required from the contractor. The RE should also take some time before the meeting to brief the design team representative on current topics.

96.4.6 Dealing with problems

In spite of the RE's best efforts, there will be times when problems arise and the RE will need to deal with these as proactively as possible.

To get an unwilling contractor to improve quality or correct mistakes is one of the hardest jobs on site. It is therefore wise to try to avoid head-on confrontation. The RE should initially try to avoid defective work by making the required standard clear the first time that any particular element is constructed or each time a new team arrives on site.

The RE's aim should be to establish a good working relationship with the contractor. If things are wrong or done poorly, the RE should explain the technical reasons behind the requirements. The RE should raise issues as soon as possible to give the contractor opportunity to correct problems quickly.

If the contractor does not act on initial verbal warnings, the RE must write a site memo to the contractor recording their verbal comments and requesting action is taken. This in itself is often enough to resolve issues. It also provides a written record of the request and the date of it, for future reference. The RE should be careful not to tell the contractor how to correct the fault; it is better to ask for their proposals.

Sometimes the contractor will fail to act on the RE's request. If the RE is satisfied that they have given the contractor reasonable opportunity to correct the issue then the RE should not be afraid take further action. In the case of an unresponsive individual within the contractor's organisation, the RE could contact one of their superiors or, in the case of the contractor's organisation generally being unresponsive the RE should contact the contract administrator or project manager. In either case the RE should record their concerns in writing and make reference to previous discussions and written communication. The RE should set out the reasons for their concern and the consequences of not resolving the issue promptly (for example repeating the faulty work).

Obviously such drastic measures should not be taken for something trivial. The RE will need to judge the severity of the fault initially and may need to seek the advice of the design team. If the defective work has since been covered the RE will need to be able to support their claim rigorously particularly if it contradicts the contractor's own records. In such circumstances, clear record photographs can be very convincing evidence of whether the alleged fault ever existed and the degree of its severity.

With one exception, the RE should never issue a stop-work instruction without first consulting the design team and the project manager or contract administrator. It is a very serious step leading to a difficult contractual situation. The one exception would be when there is immediate danger to human life (for example, an unsupported and unstable excavation) but even then it is vital that the design team and project manager are informed immediately.

Refer to Chapter 93 *Quality assurance* for more detailed information on avoiding defective work.

Box 96.4 Key points: managing the site works

- Preparing clearly documented, timely and well-organised site records is an essential part of the RE's duties.
- Numerous records will need to be produced by the RE and other site parties (refer to **Figure 96.4**).
- The RE needs to be clear on the project change procedure and follow it precisely should a change be needed.
- The RE must ensure that inspections and checks are carried out thoroughly and recorded using the pre-agreed forms.
- The RE should communicate in a clear and timely manner with the site teams and should vary the form of communication for each particular situation.
- The RE should not be afraid to ask their team for advice, especially in the early days on site.
- Regular site meetings provide an essential forum for discussing issues relating to the site works.
- When problems arise, the RE will need to deal with them proactively, raising issues as soon as possible to give the contractor the opportunity to correct problems quickly.

96.5 Health and safety responsibilities

96.5.1 The law

Health and safety practice in the UK is primarily governed by the Health and Safety at Work Act 1974 (HSAWA). This Act outlines the general duties imposed upon employers and other professionals to safeguard the health, safety and welfare at work of their employees and others who might be affected by their activities.

Implementation of the Act is through Regulations. These are in turn supplemented by Approved Codes of Practices, which contain detailed provisions and give practical guidance. Numerous regulations exist governing all aspects of construction work with those that are most commonly used listed below:

- The Construction (Design & Management) Regulations 2007 ('The CDM Regulations');
- Managing Health and Safety in Construction. Construction (Design and Management) Regulations 2007. Approved Code of Practice;
- Reporting of Injuries Diseases and Dangerous Occurrences Regulations (RIDDOR) 1995;
- Control of Substances Hazardous to Health Regulations (COSHH) 1999;
- Lifting Operating and Lifting Equipment Regulations (LOLER) 1998;
- Provision and Use of Work Equipment Regulations (PUWER) 1998.

Before embarking on a site role, the RE should ensure that they are familiar with the duties imposed by the Act and also the principal sets of Regulations under the Act.

Refer to Chapter 8 *Health and safety in geotechnical engineering* for more detailed information on health and safety law and its application to geotechnical engineering.

96.5.2 Personal safety

While the contractor is legally responsible for the safe execution of the works and supervision of the workforce, the RE also has a duty to take reasonable care of their own safety. Health and safety should form an essential part of the engineer's preparation for site works and will probably comprise a mix of formal training, briefings and reading of literature, as described in **Checklist 96.3**.

The RE should keep up to date with changes to the site health and safety procedures throughout the site works and should seek to contribute proactively to the management of health and safety on site.

During the site works the RE should ensure that they wear appropriate personal protective equipment at all times and do not take unnecessary risks. If the RE is asked to undertake work that they do not consider safe, for example to enter an unsupported excavation, then the RE should insist the contractor rectifies the problem before carrying out the work.

Checklist 96.3 Preparatory health and safety requirements

✓	Appropriate general training in site safety including a general briefing from their own employer
✓	Health and safety certification required by the contractor (for example a Construction Skills Certification Scheme card)
✓	Briefing on the risks associated with the particular site and construction works by both their own team and the contractor
✓	Access to and understanding of the necessary health and safety documentation including the construction phase plan, designer's risk assessments and contractor's risk assessments

In the event that the RE is injured, they should report the injury to the site team and to their design team immediately. As a minimum, the injury should be reported in the site accident book with further reporting to the HSE if warranted by the nature of the injury.

96.5.3 Safety of others

Under the Health and Safety at Work Act the RE has a duty to 'take reasonable care for the health and safety of themselves and of other persons who may be affected by their acts or omissions at work'. The RE should raise any observations relating to unsafe working practice promptly with the contractor. If these are not addressed in good time, the RE should refer the observations to a higher level within the contractor's and/or their own organisation as appropriate.

During their time on site the RE will receive visitors and they will need to take responsibility for ensuring their health and safety while on site. The RE should carry out a site-specific risk assessment for each visitor and provide a briefing before the site visit. The contractor should always be notified before the RE receives visitors on site and may require visitors to also attend a safety briefing given by the contractor.

Box 96.5 Key points: health and safety responsibility

- Before embarking on a site role, the RE should ensure that they are familiar with the duties imposed by the HSAWA and also the principal sets of Regulations under the Act (for example CDM).
- The RE has a duty to take reasonable care of their own safety. They must ensure that they have:
  - received appropriate training;
  - received site specific safety briefings;
  - are familiar with the key site safety documents including the construction phase plan and risk assessments.
- The RE also has a duty to take reasonable care for the health and safety of others.

96.6 Supervision of site investigation works

Site investigation works aim to provide information on the ground and groundwater conditions. Design of and methods for site investigation are discussed in detail in Section 4 *Site investigation* of this manual.

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Each site investigation will be bespoke, designed to target the issues relating to the particular site and the proposed development. However, the engineer can never really know what the site investigation will reveal and on some sites the results can show ground conditions significantly different to those expected.

The RE for site investigation works therefore needs to be prepared to respond to the findings of the investigation as they are observed, check that they align with the anticipated results and if not, adjust the investigative works to ensure that information suitable to the needs of the project is obtained. In order to do this the RE needs to fully understand the overall aims of the site investigation and the specific aim of each exploratory hole.

The RE needs to collate and assess the results of the exploratory hole information as they are produced and be prepared to instruct changes to respond to unexpected results. In doing so the RE also needs to be aware of the cost implications of the changes they instruct. Where these are large, the RE should get written agreement of the change and the cost implication from the contractor and the contract administrator before the change is implemented.

Sometimes the contractor will suggest alternative ways for progressing holes or carrying out tests. The RE should be open to alternatives but should always question the need to depart from the specification and ensure that the proposed new method will produce data of the same or better quality than the specified method.

A site investigation is only of use to a project if the data obtained are of good quality and sufficient quantity of information is obtained. The RE should be familiar with the codes and standards governing the site investigation operations. Documents relevant to most site investigations in the UK are listed below; however, the RE should be aware of other codes or guidance documents relating to more unusual site investigations or works in other countries.

- British Standard 5930 + A2:2010: *Code of Practice for Site Investigations*;
- British Standard EN1997-2:2007: *Eurocode 7. Geotechnical design. Ground Investigation and Testing*;
- NA to BS EN1997-2:2007: *UK National Annex to Eurocode 7. Geotechnical Design. Ground Investigation and Testing*;
- *Site Investigation in Construction Series, Documents 1 to 4*.

The RE should observe the works carefully to ensure that they are confident that these standards are being achieved by the contractor (refer to Chapter 13 *The ground profile and its genesis* and Section 4 *Site investigation* of this manual). The RE should also check early on that the specified sampling and testing frequencies are being achieved.

There will often be significant pressure for the RE to supervise the site works in a part-time capacity. In this instance the RE needs to time site visits carefully to ensure that they are present at the most critical times. This will vary from site to site but as a general rule the RE should be present to agree all exploratory hole locations and to observe the early works to gain confidence in the capability of the contractor. Where numerous crews are working on the site, the RE should ensure they observe the early works of each crew to check for



Figure 96.5 Cable percussive drilling rig in use
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Figure 96.6 Logging core recovered from rotary boreholes
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variations in quality and working practices. If the RE is on site as a person who will later prepare the geotechnical interpretative report (as is preferable), then they also need to observe and understand the nature of the soils exposed and should time their site visits so that soils from each stratum are observed.

Because site investigations often occur in the early stages of a project and because the site teams are normally relatively small, welfare provisions and security for site investigations can often be less well established than for other construction works. The RE should nevertheless ensure that adequate welfare, security and health and safety provisions are made on site and should insist that inadequate conditions are rectified immediately.

Box 96.6 Key points: supervision of site investigation works

The RE should:

- observe the works carefully to ensure that the required quality and quantity of data are being achieved in line with the specification and relevant codes of practice;
- check the findings of the site investigation throughout the field-work and adjust the scope of work if the ground conditions are shown to be significantly different to what was expected;
- ensure they are on site at critical times, if their role is part time.

96.7 Supervision of piling works

The precise nature of the RE's supervision activities will depend on the type of piling being witnessed and the design requirements. Sections 5 *Design of foundations* and 8 *Construction processes* of this manual describe the many types of piling that are commonly used in geotechnical engineering today. Additionally, a list of codes and guidance documents is provided below:

- BS EN1997-1:2004: *Eurocode 7. Geotechnical Design. General Rules*.
- NA to BS EN1997-1:2004: *UK National Annex to Eurocode 7. Geotechnical Design. General Rules*.
- BS EN1536: *Execution of Special Geotechnical Work. Bored Piles*.
- BS EN1583: *Execution of Special Geotechnical Work. Diaphragm Walls*.
- BS EN12063: *Execution of Special Geotechnical Work. Sheet Pile Walls*.
- BS EN14199: *Execution of Special Geotechnical Works. Micropiles*.
- BS EN12699: *Execution of Special Geotechnical Works. Displacement Piles*.
- Tomlinson and Woodward (2009).
- Various CIRIA guides, see CIRIA website www.ciria.org.

The RE should be clear on the level of checking required, the construction details to be checked and the criteria for acceptance. This will depend largely on the design. For example, if single piles are used to support critical elements of structure then it would be expected that every such pile is rigorously checked due to the serious consequences of failure of a defective pile. However, it may be acceptable to check a proportion of piles for a design using pile groups to support a structure due to the inherent redundancy in the pile group. The complexity of the construction technique and relative experience of the contractor will also be factors, for example more rigorous checking of new or complex piling methods would be expected over proven and relatively straightforward piling methods.

It is also important that the RE is familiar with the practical elements of the piling technique before they take sole responsibility for the site supervision. Ideally this would be achieved by visiting another site using the same piling technique and assisting the RE there. Otherwise an experienced engineer should initially accompany the RE to site to assist in understanding the piling process and gaining confidence in the checking procedure.

The RE should ensure that they clearly understand the contractor's piling method and also the consequences of varying that method on the pile design.

There will be times on site when circumstances beyond the contractor's control disrupt the piling works, for example breakdown of plant, unforeseen ground conditions or severe weather. At such times, the contractor and the RE will come under pressure to resolve the situation as quickly as possible and with minimal effect on the pile design. It is therefore useful for the RE in conjunction with the contractor to consider the possible situations where things could go wrong and plan their actions in advance.

Where the piling works are repetitive, it is even more important than usual for the RE to establish the required quality standard at the start of the works. The RE also needs to ensure that standards are maintained throughout the work, particularly on long projects where personnel may change over the course of the contract. The RE will often be a young face on site compared to the site operatives and should recognise that these people can be very knowledgeable. The RE should consider their suggestions carefully but should also be aware that just because the crew have 'always done it like that' does not mean that it is the right solution for the project.



Figure 96.7 Typical bored piling site in Central London

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On piling projects, particularly those with high pile production rates, it can be tempting for the RE and contractor to fall behind with their record keeping. It is important that the RE does not let this happen and insists on records being produced in the timeframes specified in the contract.

Timely and accurate production of close-out reports are increasingly important on piling projects (refer to Chapter 101 *Close-out reports*). Close-out reports should be comprehensive and detail not only the as-built pile details but also record any problems encountered during the works and the chosen resolution. This will aid the re-use of the piles in the future and the RE should not consider their duties fulfilled on site until these have been completed.

Box 96.7 Key points: supervision of piling works

The RE should:

- be familiar with the practical elements of the piling technique (through previous experience or visits to other sites using the same technique);
- be clear on the level of checking required, construction details to be checked and the criteria for acceptance;
- keep up to date with records, especially if production rates are high;
- have considered the possible situations where things can go wrong and (with the contractor) plan their actions in advance;
- ensure that standards are maintained throughout the work, particularly on long projects where personnel may change over the course of the contract.

96.8 Supervision of earthworks

Earthworks construction projects can range from the very small (excavation of pad foundations for a low-level building) to the very large (major road infrastructure construction). However, the level of supervision required should be decided upon based on the complexity of the project as well as its scale.

For detailed information on design of earthworks, refer to Section 7 *Design of earthworks, slopes and pavements* of this manual. Additionally, a list of codes and guidance documents is provided below:

- BS EN1997-1:2004: *Eurocode 7. Geotechnical Design. General Rules*.
- NA to BS EN1997-1:2004: *UK National Annex to Eurocode 7. Geotechnical Design. General Rules*.
- BS 6031: *Code of Practice for Earthworks*.
- Various CIRIA guides, see CIRIA website www.ciria.org.

It is important that the RE understands the intended use of the earthworks structure as this will influence the level of inspection and the decisions they are required to make. For example, criteria for a structure required to carry heavy loads or plant with strict movement tolerances will be much more stringent than for a non-structural fill providing a platform for soft landscaping.

Prior to the site works commencing it is important to pre-classify the materials on site through site investigation (refer to section 96.6 above and Section 4 *Site investigation* of this manual). This allows the design and site teams to identify the appropriate type of plant to use for excavation, the angle of slopes that can be achieved and for filling the level of compaction that can be achieved. In addition the RE should make themselves familiar with the types of compaction plant that may be suitable for the material on site and how they function.

With the exception of the smallest earthworks projects, it is usually only possible to inspect a selection of the works. It is important that the RE ensures that the required frequency of testing (either *in situ* or laboratory-based) is obtained and that the samples are representative of the site as a whole and not, for example, just a limited area of the site or soil type. It is normally preferable for the RE to choose the testing locations to ensure these are independent of the contractor and focused on the key activities. Alternatively the contractor may propose a sampling and testing schedule for agreement with the RE. In addition procedures for dealing with contaminated material should be agreed prior to testing in case any such material is encountered during the works.

Where testing of the samples is required, the RE must ensure that the laboratories have achieved the specified certification for the particular tests that have been carried out. This is particularly important for site-based laboratories. For off-site laboratories it is important to ensure that samples are stored and transported in such a way as to have minimal effect on their behaviour.

Detailed daily recording of the weather is essential to earthworks operations as this could affect the performance of the soils used. Additionally, the possible effects of inclement weather on the performance of the materials should be considered by the design team and understood by both the RE and the contractor. The RE should be aware of the contractual implications of inclement weather and if it is counted as 'extraordinary', in which case the client could be liable for any costs incurred or if it is within the contractor's liability.

Where earthworks are inspected, the RE should keep good quality annotated photographic records complemented by



Figure 96.8 Typical earthworks site
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detailed descriptions of the works they have observed and the type and location of compliance testing. The contractor will often have to produce a validation report at the end of the works, which the RE may be asked to comment on. The RE should ensure that the contractor is providing elements of this report throughout the site works to prevent problems occurring at the end of the works if the information is inconsistent or incomplete. The RE should also produce a comprehensive and timely close-out report (refer to Chapter 101 *Close-out reports*) to summarise the works and to collate their independent records.

Box 96.8 Key points: supervision of earthworks

The RE should:

- understand the intended use of the earthworks and how that influences their supervisory role;
- have a thorough understanding of the site investigation and therefore the site soils;
- ensure that the sampling frequency and quality of testing are achieved and that samples are representative of the site as a whole;
- keep detailed records of the weather and understand how conditions could affect the performance of the soils;
- keep good quality annotated photographic records.

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96.9.3 Useful websites

British Standards Institution (BSI); www.bsigroup.co.uk

Construction Industry Research and Information Association (CIRIA); www.ciria.org

Construction Skills Certification Scheme; www.cscs.uk.com

Health and Safety Executive (HSE); www.hse.gov.uk/construction

It is recommended this chapter is read in conjunction with

- Chapter 8 *Health and safety in geotechnical engineering*
- Chapter 78 *Procurement and specification*
- Chapter 100 *Observational method*
- Chapter 101 *Close-out reports*
- Section 8 *Construction processes*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 97

Pile integrity testing

Simon French Testconsult Limited, Warrington, UK
Michael Turner Applied Geotechnical Engineering Ltd, Steeple Claydon, UK

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A pile integrity test is defined as a non-destructive test (NDT) which examines the response of a pile to some form of indirect physical scanning technique; such a test may employ acoustic shock waves, electrical energy, nuclear radiation or other input to excite the pile (CIRIA 144, 1997). Analysis of the response of the pile may allow an interpretation of the 'integrity' of construction of the pile body, typified by the homogeneity of the construction concrete or the uniformity of its external shape. Such tests are commonly used as part of a pre-planned quality control programme, as a retrospective testing programme, or as an aid to the evaluation of existing foundations.

The most common non-destructive pile testing techniques are described with particular reference to low-strain integrity tests and cross-hole sonic logging. Guidance is given on the analysis, interpretation and reporting of the results of such tests, including computer-aided signal matching and modelling techniques to enhance the interpretation of the signal responses obtained.

Parallel seismic testing and high-strain integrity testing are also reviewed. The former is commonly used for retrospective testing of foundation piles when they are to be re-used and the latter is a by-product of dynamic load testing.

97.1 Introduction

CIRIA Report 144 (CIRIA 144, 1997) defines a pile integrity test as a non-destructive test (NDT) which examines the response of a pile to some form of indirect physical scanning technique. Integrity testing is most commonly used as part of a pre-planned quality control programme for a particular site. It can also be used as part of a retrospective testing programme, where a problem has become apparent during or subsequent to the construction of the piled foundations. It can also be used as an aid to the evaluation of existing foundations: where they might be required to be re-used as part of a new development for example.

Non-destructive, or integrity testing of foundation piles has become routine on most UK construction sites, but is often considered something of a 'black art' by many engineers and industry professionals. Such tests employ indirect testing techniques to examine the structure of the pile. These techniques include the use of acoustic shock waves from a hammer blow; ultrasonic pulses; electrical resistivity; nuclear radiation such as gamma or neutron scanning; or other input to excite the pile. The response of the pile allows interpretation to be made and conclusions to be drawn on the 'integrity' of construction of the pile body, typically related to its internal construction or the uniformity of its external shape.

Pile integrity testing and its principal methods have evolved since their introduction in the 1970s to become an important part of the quality control process. They can provide additional information on the as-built construction of piles, as well as enabling further evaluation of piling systems. Recent advances in computer technology have provided convenient solutions for compact and portable site acquisition equipment, and developments of sophisticated analysis programs have led to their widespread use by the construction industry.

This chapter will provide an insight into the main pile testing methods currently in use, together with background theory and guidelines on the choice of the most appropriate method. Specifying the most appropriate test is not universally understood and the test method applied is often driven by varying factors. A wider understanding is therefore essential, and all engineers with responsibility in this area should have a working knowledge of the appropriateness of the various test methods, their limitations and the interpretation of the data obtained from such tests.

The main test techniques currently in use in the UK, which will be discussed in this chapter, can be identified as:

- low-strain integrity testing;
- cross-hole sonic logging;
- parallel seismic testing;
- high-strain dynamic testing.

Electrical and nuclear, or radiometric, tests are not commonly used in general site practice. Further information on these and other test techniques can be found in Fleming *et al*. (2009) and CIRIA 144 (1997).

The principal features of these four main test techniques can be summarised as follows:

(a) *Low-strain integrity testing* is the examination of the response of the pile to an external impulsive force: usually imparted by striking the head of the pile with a light,

hand-held hammer, generally fitted with a hard plastic tip. The shock wave from the hammer blow travels down the pile. Part or all of the energy from this blow may be reflected or affected by a change in pile properties: the pile toe being an obvious source of such an effect. An analysis of the response of the pile (measured by a receiving sensor at the pile head) to both the initial blow and any returning reflections from depth allows inferences or conclusions to be drawn on the properties of the buried pile.

(b) *Cross-hole sonic logging (CSL)* is the examination of the response of the pile to a sonic or ultrasonic pulse travelling between a number of vertical ducts cast into the pile concrete. A signal transmitter or emitter is lowered to the bottom of one duct, and a receiver is lowered to the bottom of an adjacent duct. Both transducer and receiver are then raised in tandem at a predetermined rate. Changes in the characteristics of the pile or its constituent materials can affect the travel time or quality of the signal arriving at the receiver.

(c) *Parallel seismic testing* is a method of estimating the acoustic length of a pile where no direct access can be gained to the pile head (a pile within an existing foundation for example). A vertical tube, such as a borehole casing, is driven or bored into the ground alongside the pile. A sensor is lowered sequentially down the tube while the foundation above the pile is struck with a small hand-held hammer. The transit time of the shock wave to travel through the pile and the intervening ground to the sensor is measured. Once the sensor passes the end of the pile, the shock wave must pass through an increasing distance of soil to reach the sensor and the transit time will increase markedly.

(d) *High-strain integrity testing* is an ancillary use of dynamic pile testing systems such as those as described in Chapter 98 *Pile capacity testing*. The test involves striking the pile with a large falling weight, such as a piling hammer. Sensors attached to the pile measure the impulsive force imparted into the pile by the falling weight and the resulting acceleration and velocity of the pile as it is driven downwards. Analysis of the signals from these sensors can allow the identification of pile damage such as a broken pile.

97.2 The history and development of non-destructive pile testing

97.2.1 Low-strain integrity testing

Most of the integrity testing of piles in the UK involves measuring the acoustic properties of the pile by striking it or causing some other acoustic vibration within it. This mechanical impulse generates 'shock' or 'acoustic' waves, which travel along or through the pile at a velocity determined by the characteristics of the pile material. The transmission of such waves through the pile is most readily represented or analysed by stress-wave propagation theory using the principles of one-dimensional wave mechanics in elastic rods.

During the 1960s low-strain integrity testing research was pursued in France by Paquet, working at the Centre Experimental de Recherché et d'Etudes du Batiment et des Travaux Publics (CEBTP). This centred upon the response of a pile to an external impulse in both the time ('sonic echo') and frequency ('vibrational') domains. This work led to the conclusion by CEBTP (Paquet, 1968) that there were limitations to the time-based 'echo' methods and that the assessment of piles by the 'vibration method' was preferable, particularly when defects in the upper few metres of the pile were of concern. These developments are discussed in detail in Hertlein and Davis (2006). Contemporary technology made practical on-site application of the theories discussed in the paper difficult to apply, and it was not until the early 1970s that appropriate equipment became commercially available.

At that time, Davis and Dunn (1974) described the application, theory and interpretation of results gathered by the frequency-based vibration method, using an electrodynamic vibrator or 'shaker' attached to the pile head and measuring the response of the pile to the spectrum of vibration frequencies. The significant downside to the method at this time was the extensive pile head preparation necessary to perform the test and the cumbersome equipment required.

Over the same period research was also being undertaken at Institute TNO in the Netherlands, and Case Western University in Texas, for example. These projects were generally focused on pile driving analysis using the wave equation (Smith, 1960). These researchers also investigated the use of the wave equation to examine pile integrity based upon 'low-strain' test techniques using a small hand-held test hammer. These forms of analysis therefore concentrated upon time-based 'sonic echo'-type techniques.

During the early 1980s the CEBTP 'vibration' test method was further developed by substituting the vibrator with a broad spectrum load cell which was struck with a small hand-held hammer to generate the necessary range of frequencies to accommodate the test. This was further developed into an integrated hammer which incorporated the load cell (Stain, 1982). After 1982, the test method was characterised by CEBTP as the Transient Dynamic Response (TDR) method. Other specialists use terms such as the impulse response method. In the following text such tests are described under the generic name of frequency response tests.

Parallel developments in the 1980s by both research and commercial organisations saw the development from analogue to digital processing, recording and analytical techniques, and, by the mid-1990s site equipment had become truly portable, being both compact and battery powered. Such systems can store several hundred pile records and, if necessary, download them via a modem link to an office base for speedier processing and report production. Advances in software and analysis programming with the advent of greater computing power

have also enabled result simulation and modelling techniques to be used to aid result interpretation, as described later.

97.2.2 Cross-hole sonic logging and parallel seismic testing

The history of the development of cross-hole sonic logging (CSL) and the parallel seismic test broadly follows a similar path to low-strain integrity testing, as outlined above. Cross-hole sonic logging, in particular, was developed by CEBTP in the late 1960s (Paquet, 1969) to counter one of the flaws with low-strain integrity testing, which was that the response of the pile to the pile head excitation is depth-limited. Cross-hole logging utilises sonic transmitter and receiver probes which are lowered down ducts within the pile and allow the construction characteristics of the pile to be examined at depth.

Parallel seismic testing (Paquet and Briard, 1976) utilises a duct placed outside a previously constructed pile (which might have been already incorporated into the as-built structure, for example) to examine the length or characteristics of the pile.

As with low-strain testing, the use of these systems was limited by the technology available at that time. Advances in portable computers throughout the 1980s and the introduction of improved analysis equipment saw widespread use of CSL on major projects in Europe, the Far East and Africa.

97.3 A Review of defects in piles in the context of NDT

97.3.1 Introduction

The foundation system is selected by the designer of the piles and/or the overall scheme designer from a range of possible alternatives to suit the ground conditions and the imposed structural loads. In selecting the system both parties need to consider the means by which the as-constructed works are checked against the design intentions and assumptions. Ideally, these design and performance requirements should be outlined within the project specification.

To a large extent, each pile is unique and, because it is buried in the ground, it cannot be examined in the same way as the structure it supports. Additionally, the foundation system cannot normally be subjected to pre-loading to confirm its adequacy before being put into service (except in a case such as a storage tank or silo). Hence, the evaluation of the foundation pile system usually requires inference and extrapolation from a combination of direct and indirect testing and, if necessary, the targeted examination of individual piles. It should be understood that a key point in any such evaluation process is the accurate recording of construction information for each pile, usually by the pile installer. Guidance on essential construction records are listed, for example, in the ICE SPERW document (ICE, 2007).

NDT is therefore undertaken as part of the contractual evaluation system, to provide additional information on aspects of the pile construction, as an aid to the evaluation of the foundation system.

97.3.2 An overview of pile defects and NDT

Problems that may arise in the construction of various types of pile are reviewed in Chapter 82 *Piling problems*. In the context of the use of NDT, the features arising from the problems, or potential problems, identified in Chapter 82 *Piling problems* fall into four general categories (CIRIA 144, 1997). These are identified as:

Type A. Changes to the intended shape of the pile body itself.

Type B. Total rupture of the pile, in the form of a transverse crack or break across the body of the pile.

Type C. Changes in the internal properties of the pile.

Type D. Features that affect the interaction of the pile with its environment and, consequently, its ability to transfer the design load into the surrounding soil or rock.

Type A features are usually associated with some aspect of the pile-forming process, either in the pile construction or post-construction phase. Features that affect the shape of the pile body were generally identified by CIRIA 144 (1997) by the terms necks, waists, bulbs, expansions, steps or bites, as illustrated in **Figure 97.1**.

Type B features are linked to the action of unplanned external tensile or lateral forces acting upon the hardened pile material, before (as in the case of pre-formed or precast piles), during or after installation. The effect of such an external force is typically to produce a transverse fracture perpendicular or at an acute angle to the long axis of the

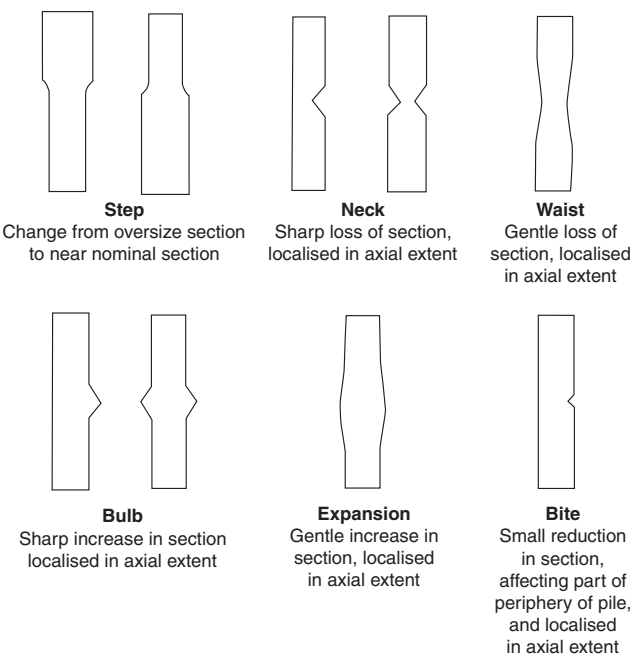


Figure 97.1 Typical type 'A' features in a pile
Reproduced with permission from CIRIA R144 (1997), www.ciria.org

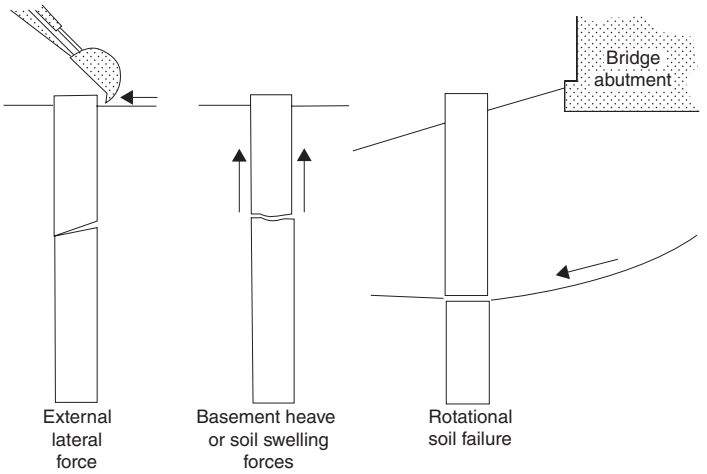


Figure 97.2 Typical external causes resulting in type 'B' fractures
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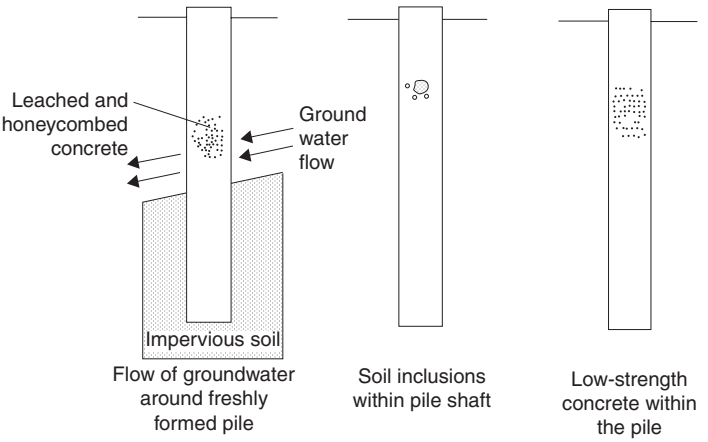


Figure 97.3 Typical type 'C' features within a pile
Reproduced with permission from CIRIA R144 (1997), www.ciria.org

pile, as illustrated in **Figure 97.2**. Transverse cracks can also be formed by shrinkage of the concrete or by the effects of ground heave.

Type C features might be caused by faulty concrete or concreting processes, or the use of sub-standard materials. They therefore reflect internal changes in the properties of the pile, as illustrated in **Figure 97.3**. The changes in properties, such as concrete strength, could be gradational or sharp. Other examples are where some of the pile concrete is contaminated by suspended spoil (e.g. piles formed under bentonite) or variations in concrete cover to steel reinforcement.

Type D features include poor toe conditions and softened or degraded bore sides due to relaxation or water ingress. Type D features are therefore typically associated with deficiencies in the pile construction process at the boring or drilling stage, prior to concreting, or, in the case of driven piles, with pile heave in congested groups, or with relaxation effects after driving. Such factors can therefore directly affect the load-carrying capacity of the pile.

It is important to understand, however, that these 'features' are not necessarily defects: in the sense that a 'defect' might be considered to be something that would significantly affect the short- or long-term performances or the load-carrying capacity of the pile. The identification of a particular feature is not a judgement of whether a pile is defective, sub-standard or non-compliant. Further evaluation and, possibly, further investigation may need to be undertaken to determine whether the feature is an unacceptable 'defect'.

CIRIA 144 (1997) proposed the terms 'anomaly' and 'defect' where an anomaly was an irregular or unexpected response that might or might not represent a real feature of the pile shaft. In contrast, a defect was categorised as a feature in the pile that was not in accordance with its specified construction, and which might or might not affect the ability of the pile to perform as required.

In terms of the latter Amir (2001) proposed further that a defect which was not significant in terms of its effect on pile capacity or durability should be termed a 'flaw', reserving the term 'defect' for a feature that would affect the bearing capacity or likely durability of the pile. A 'defect' would require engineering evaluation: perhaps the pile could be accepted at a reduced capacity for instance.

Integrity testing therefore inherently requires a step-by-step process leading from the identification of a 'feature' or an 'anomaly' by the test to the evaluation of its significance and the decision on any further action required.

Section 97.4.6, below, discusses in more detail these analytical and interpretative processes associated with the evaluation of such 'features'.

It is important that the method of testing and evaluation should be chosen with reference to the pile type and construction method, pile layout, piling programme, subsoil conditions and level of site control, both during and after pile installation, to ensure it is suited to the detection of the type of fault which could exist.

A further point to understand is that, in general, non-destructive tests are better at highlighting sharp changes in pile properties, as opposed to gradational ones. Hence in **Figure 97.1** small changes such as 'waists' or 'expansions' or gradual changes in pile diameter, rather than 'steps', would typically be difficult or impossible to identify in practice.

97.4 Low-strain integrity testing

97.4.1 Time-based analysis

The general arrangements for a time-based integrity test are illustrated in **Figure 97.4**. These tests are often known as 'echo' or 'sonic echo' (time domain) tests.

To understand the essence of the test technique it is helpful to initially consider the case of a perfect free pile (i.e. a pile which has no lateral soil restraint or base support) of length L. In this case, the toe of the pile is said to be a 'free' end, as

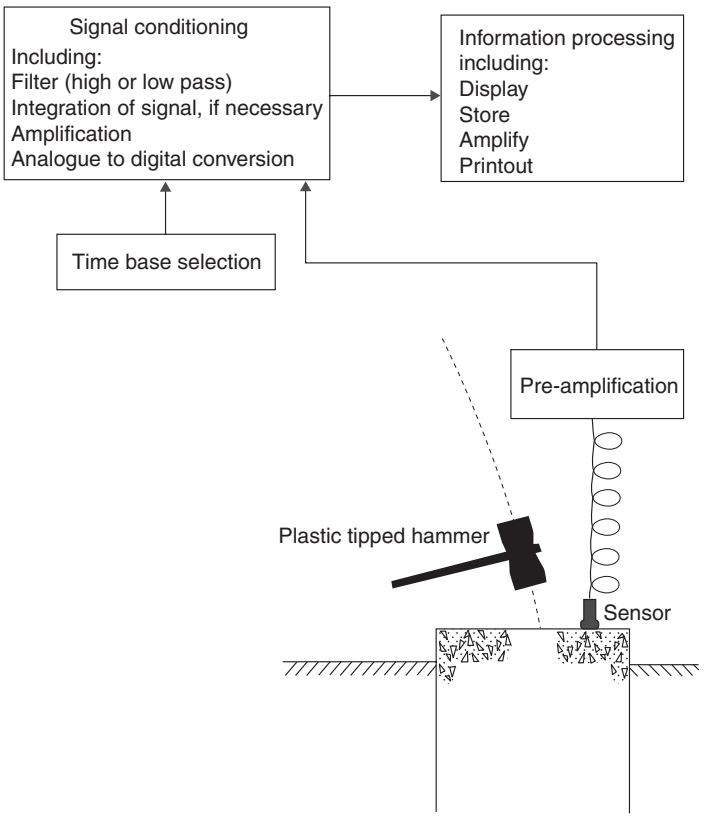


Figure 97.4 Schematic of a typical time-based 'sonic echo' test layout
Reproduced with permission from CIRIA R144 (1997), www.ciria.org

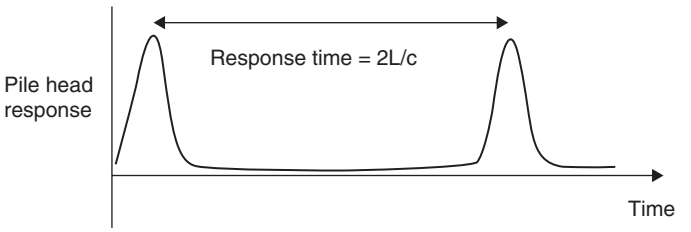


Figure 97.5 Graphical representation of hammer blow and 'echo' reflection observed at the pile head from a 'free' end at the pile toe

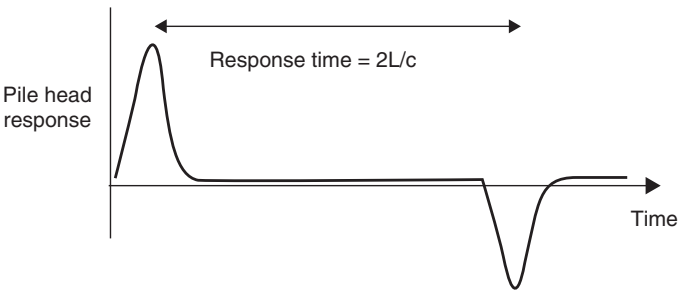


Figure 97.6 Hammer blow and 'echo' reflection observed at the pile head from a 'fixed' end at the pile toe, showing an upward-travelling compression wave, with its polarity opposite to the downward-travelling wave

it is free to move downwards in response to the downward-travelling stress wave from the hammer blow. Applying a short duration impact to the pile head, such as a blow from a hand-held hammer, causes a compression wave to propagate down the pile in the form of a longitudinal or bar wave (commonly termed a stress wave). Upon reaching the pile toe the wave reflects back to the pile head as a tension wave. The fundamental propagation velocity, c, for longitudinal waves is given by one dimensional wave theory as:

$$c = \sqrt{E/\rho} \qquad (97.1)$$

where E = material Young's modulus
and ρ = material density.
It can be seen that the time, t, taken for this wave to propagate from the pile head to the pile toe and back up to the head will given by:

$$t = 2L/c. \qquad (97.2)$$

This is illustrated in **Figure 97.5**, where the first peak is generated by the hammer blow and the second peak is the reflection or 'echo' of the wave returning from the 'free end' at the pile toe. It should be noted that on most of the illustrations of time-based (sonic echo) testing in this chapter the intial hammer blow is represented as an upward peak on the signal response graph. It will be realised that it could equally be represented as a downward-facing peak, and hence all other peaks or troughs will be similarly reversed, and other figures show this mode. Commercially available testing systems usually adopt one or the other method of data presentation.

Now consider the case of a similar pile, without skin friction but this time supported on an infinitely rigid base. The applied compression wave from the hammer blow upon reaching the supported base reflects in the form of a compression wave but one with an opposite polarity to the downward travelling wave as shown in **Figure 97.6**.

In practice, of course, the pile will be embedded in the ground. This has the effect of dissipating the energy from the stress wave as it propagates down the pile shaft. The degree of signal attenuation is a function of the pile diameter, the pile length and the stiffness of the surrounding soils. Generally speaking, piles with a length to diameter ratio in excess of 30:1, embedded in stiff to very stiff clays exhibit 100% signal attenuation: so that, effectively no return signal reflected from the pile toe reaches the pile head. Hence, in such a situation, it can be seen that the test may be of diminishing use in identifying the location of the toe of the pile, if this was of concern to the engineer. However, it must be appreciated that the test will still have application in examining the upper section of such a pile.

The attenuation of the low-strain integrity test signal from longer piles was also the reason that cross-hole sonic logging was developed (see section 97.4.5).

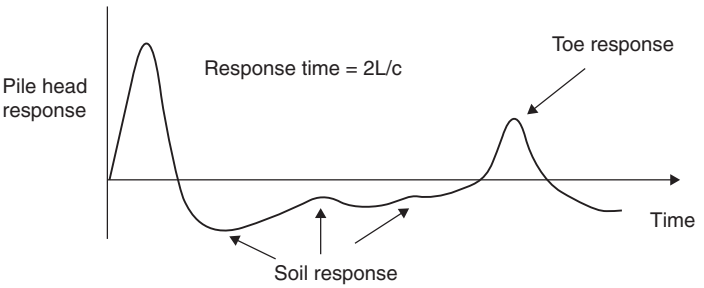


Figure 97.7 Pile head response in multiple soil horizons

Velocity of plane wave propagation in concrete	3500–4000 m/s
Density/unit weight of concrete	23–24 kN/m³
Young's modulus of concrete	approximately 38 GN/m²
Soil shear wave velocity	100–300 m/s
Soil density/unit weight	16–21 kN/m³

Table 97.1 Typical values of material properties

Changes in soil characteristics along the shaft will also generate partial reflections of the downward-travelling stress wave. At each of these soil horizons, a proportion of the stress wave will be reflected back to the pile head, the remaining portion continuing to the pile toe. Multiple soil horizons along the shaft can therefore have a major effect on the theoretical pile response, so that interpretation of the return signal becomes more complex, as shown in **Figure 97.7**.

Inevitably, the resultant response from an embedded pile is often not as straightforward to interpret as the theory may suggest. A more detailed explanation of wave propagation in piles is given in CIRA Report 144 (1997).

97.4.2 Typical pile and soil properties

As discussed in 97.4.1 above, in order to evaluate the response of a pile to a low-strain integrity test impact it is necessary to consider the properties of both the material from which the pile is constructed and the soil surrounding the shaft. It can be appreciated that a sound pile shaft constructed in a very stiff soil, or socketed into rock, would produce a response significantly different to a pile floating in space with no skin friction or base support. The stiffness of the soil surrounding the embedded pile contributes to damping of the stress wave as it propagates down the shaft. This damping factor was first recognised by Briard (1970) and can be measured by the attenuation of the response from the pile base. Typical values of material properties are as shown in **Table 97.1**

97.4.3 Pile impedance

Any changes in pile properties along the shaft will also generate partial reflections of the stress wave as it passes the boundary at which the change occurs. Such property changes within the pile are defined by a parameter termed the impedance, Z, of the pile. The value of Z at any given location along the shaft is given by:

$$Z = \rho \,.\, c \,.\, A. \qquad (97.3)$$

An increase in pile impedance, such as an increase in the cross-sectional area, A, of the pile, will generate a compression wave at the level of the change. This will propagate back up to the pile head and produce a pile head response in the *opposite* direction to the hammer impact response: in a similar manner to the rigid toe reflection illustrated in **Figure 97.6**. Conversely, a reduction in pile impedance, such as a reduction in cross-sectional area, or a reduction in the density of the concrete forming the pile, will generate a tension wave: producing a response at the pile head in the same direction as the hammer impact response, in a similar manner to the 'free end' toe reflection in **Figure 97.5**.

It can be seen, therefore, that the reflection of the signal from the toe of the pile, and its interpretation as either a free end or a fixed end effect, would also be produced by other features within the pile, such as a change of section, for example. Typical features which might be expected to produce a free end response are illustrated in **Figure 97.8(a)**. Similarly, features which would be expected to produce fixed end responses are illustrated in **Figure 97.8(b)**.

In addition, the impedance change associated with a particular feature might only cause a partial reflection of the downward-travelling stress wave from the initial hammer blow on the pile head, while the remainder of the signal continues downwards towards the toe of the pile. An example of such a feature might be a bulge or a neck in the pile section. It will be appreciated that, in such a case, the part of the signal that reaches the pile toe and is reflected back towards the pile head will again suffer partial reflections, and hence attenuation or degradation, as it passes through the feature, so that it might be barely detectable at the pile head. The practical significance of such partial reflections is that the pile head response becomes more complex as a consequence of the effect of more than one impedance change upon the transmitted and reflected stress waves. **Figure 97.8(c)** illustrates the effect of the possible 'combined' response of piles with a neck or a bulb feature within the pile section. In addition, **Figure 97.8(d)** illustrates the effect of the attenuation of the signal response as a result of soil skin friction upon the pile. At the limit, for a long pile embedded in a stiff soil, no toe reflection would return to the sensor at the pile head, and only the initial impulse from the hammer blow would be recorded, as illustrated in **Figure 97.8(e)**.

It will be appreciated that a relatively small impedance change would only reflect a small percentage of the downward-travelling wave, so that most of the signal would continue downward towards the pile toe. As the impedance change becomes greater, an increasing proportion of the downward-travelling wave would be reflected. Ellway (1987) suggested

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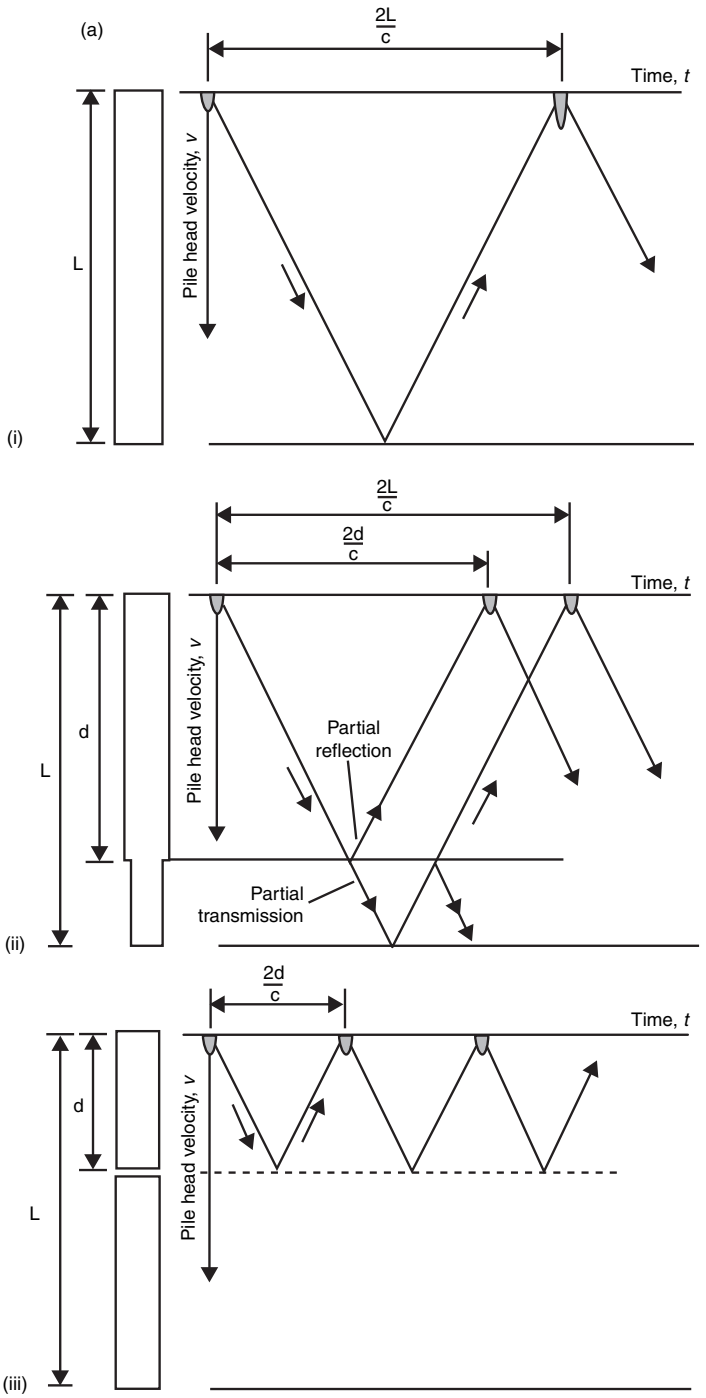


Figure 97.8 (a) Typical simplified free end responses for time-based tests:
(i) Toe of pile (note: repeated reflection at time intervals of 2L/c);
(ii) Intermediate decrease in cross-section (partial reflection at change of section, reduced toe reflection);
(iii) Broken pile/complete loss of section/crack (free end reflection from break, repeated at time intervals of 2d/c)
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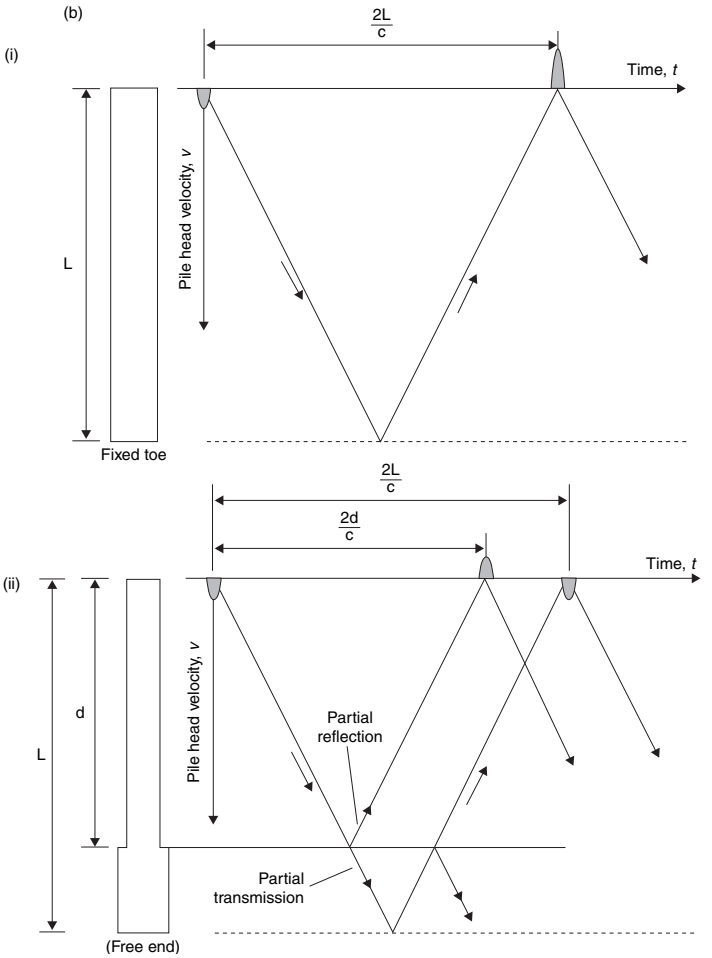


Figure 97.8 (b) Typical simplified fixed end responses for time-based tests:
(i) Toe of pile (note: repeated reflection at time intervals of 2L/c);
(ii) Intermediate increase in cross-section (partial reflection at change of section, reduced toe reflection)
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that when the ratio Z_1/Z_2 or Z_2/Z_1 is greater than 4 the downward-travelling wave will be effectively completely reflected at the interface between Z_1 and Z_2, so that no return signal would be received from below this level.

97.4.4 Frequency-based analysis

Initially, as discussed in section 97.2.1, for the 'vibration' test method employed by Paquet (1968) and Davis and Dunn (1974), the pile head was subjected to a cyclically varying excitation force using an electro-dynamic 'shaker' attached to the pile head. By this means the pile head force was applied in discrete frequency steps, gradually sweeping through a frequency range from 0–1000 Hz. By applying this vibration at constant amplitude and force F_o and observing the maximum velocity V_o at the head of the pile within this frequency range it will be seen that the discrete frequencies at which resonance occurs are equally spaced along the frequency spectrum. The first resonating peak

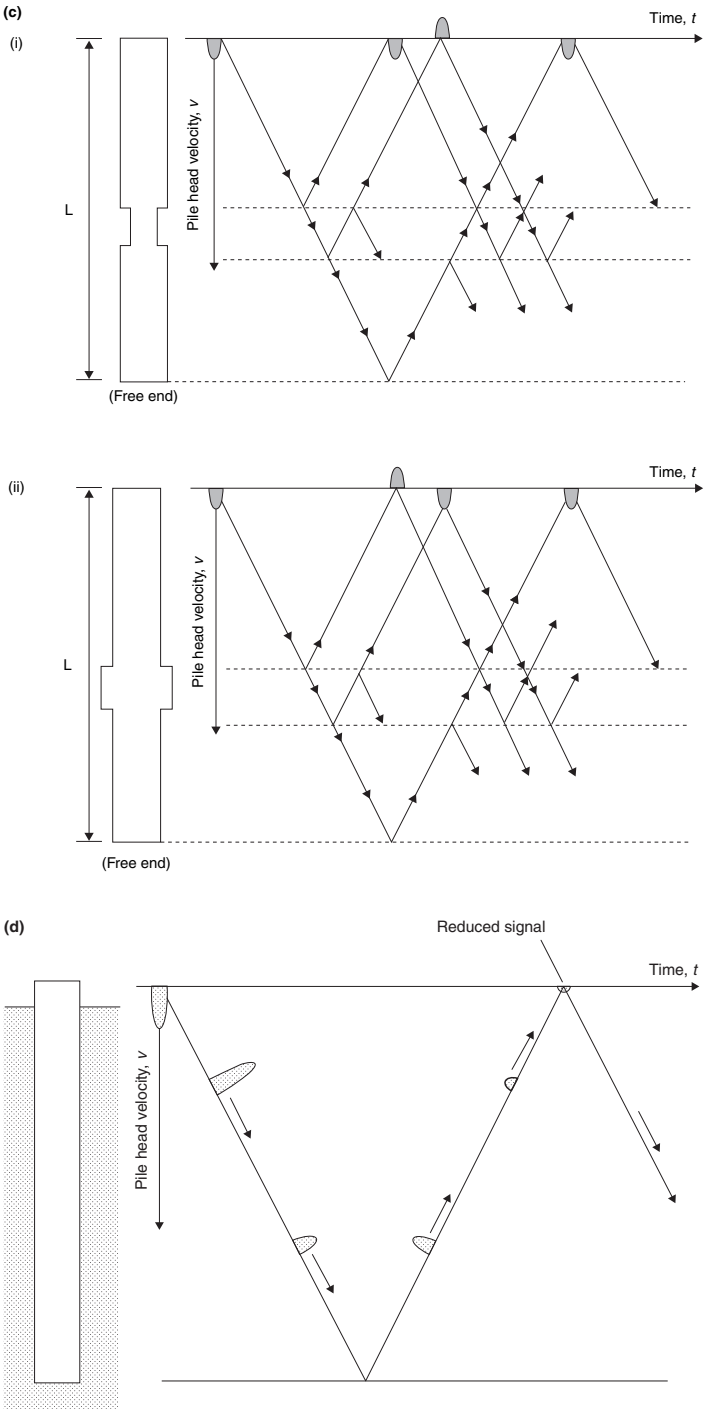


Figure 97.8 (c) Typical 'combined' responses for time-based tests:
(i) Free end with neck (partial reflections at both changes in section, reduced toe reflection);
(ii) Free end with bulb (partial reflection at both changes in section, reduced toe reflection);
(d) Attenuation of stress wave due to skin friction
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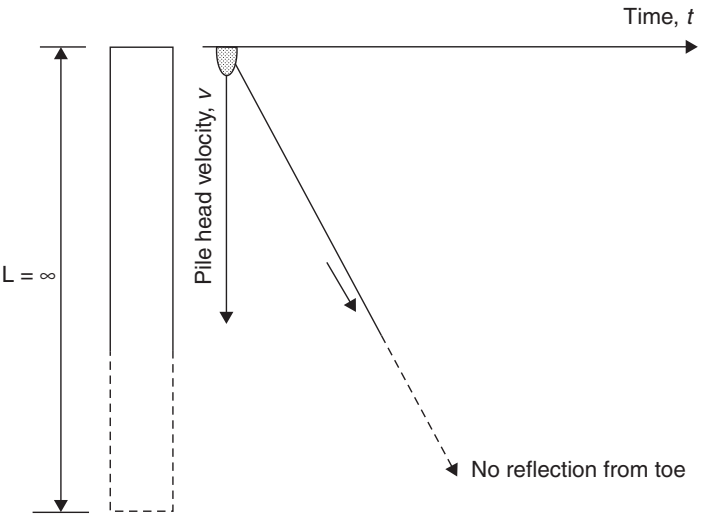


Figure 97.8 (e) Signal response for an infinite pile (no toe reflection)
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in the series is the natural frequency of the pile and subsequent peaks are harmonics of the natural frequency.

As described earlier, later developments utilised a load cell attached to the pile head, which was then struck with a small hand-held hammer fitted with a suitable impact head capable of generating the same bandwidth of frequencies used for the vibration tests. Current systems use a hammer incorporating the load cell as a single complete unit.

The current frequency-based test techniques are commonly known as 'impulse' response and by such proprietary names as Transient Dynamic Response (TDR) methods. Within this text they will be described under the generic term of 'frequency response' tests.

An important difference between the current time- and frequency-based techniques is that the latter always measures the force applied to the pile head by the hand-held test hammer. A schematic of the general arangement of this test method is shown in **Figure 97.9**.

This method can best be understood by referring back to the original 'vibration' test technique using an electro-dynamic shaker. For any discrete frequency of excitation generated at the pile head, the returning wave and consequential pile head velocity will be the vector sum of the phase of the returning wave and the original input wave. For example, if the returning wave is in phase with the input wave then the pile head velocity will be at a maximum; if out of phase by 180 degrees the resultant pile head velocity will be at a minimum.

From standard wave theory relating to vibrations in long slender rods, the distance between resonating peaks, Δf, is given by:

$$\Delta f = c / 2L \tag{97.4}$$

where c is the velocity of plane wave propagation along the pile and L is the length of the pile.

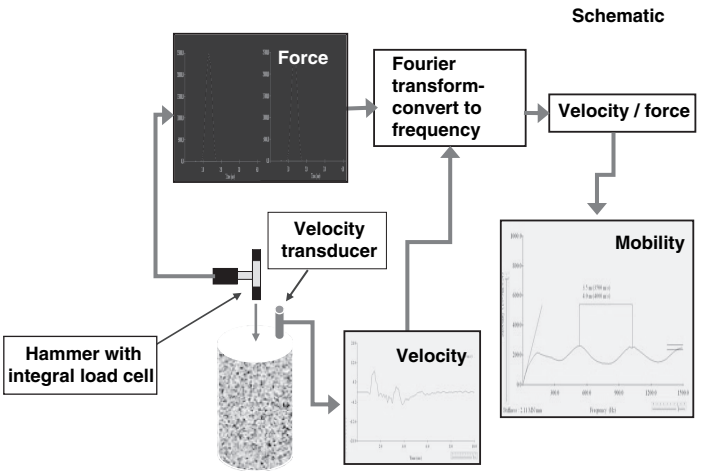


Figure 97.9 Schematic of general arrangements for the frequency response method
Courtesy of Testconsult Limited

To 'normalise' the response from the pile impact and produce a unique pile signature the output from the pile head velocity transducer (V_o) is divided by the input force (F_o), where V_o / F_o is the mobility, M_o (also termed the mechanical admittance), of the pile at the particular frequency. The units for mobility are therefore m/s/N. The pile head response is thus plotted in terms of pile head mobility versus frequency.

In the case where the elastic base is infinitely rigid (see section 97.4.1 above), the lowest frequency of resonance has a value of *c/4L* as shown in **Figure 97.10(a)**.

By contrast, when a pile rests on an infinitely compressible base, resonance first occurs at a very low frequency (**Figure 97.10(b)**). When the base is an elastic one of normal compressibility the lowest frequency of resonance lies in an intermediate position between that of the rigid and infinitely compressible bases as shown in **Figure 97.10(c)**.

In a similar manner to that described above for time-based analyses, a free or fixed end effect could also be produced by other features within the pile: such as a reduction or increase of the pile section, for example, or a broken or cracked pile.

As described by Ellway (1987), the sharpness of the pile shaft resonances depends upon the relative amount of the energy of the stress wave that is either transmitted or dissipated each time it is reflected from a boundary layer, such as the pile toe. A short pile in free air would have little energy loss, and would be expected to exhibit the sharp resonant peaks illustrated in **Figures 97.10(a)** to **97.10(c)**. However, where a pile is embedded within a soil then the propagation of the stress wave will be dissipated, or *damped* by the soil. The idealised signal response curve of **Figure 97.10** would therefore be modified to the form shown in **Figure 97.11(a)**. This form is typical of most frequency response signal curves met in practice.

In an exactly similar way to the 'echo' test, if the soil was sufficiently stiff, all of the stress wave would be attenuated and no

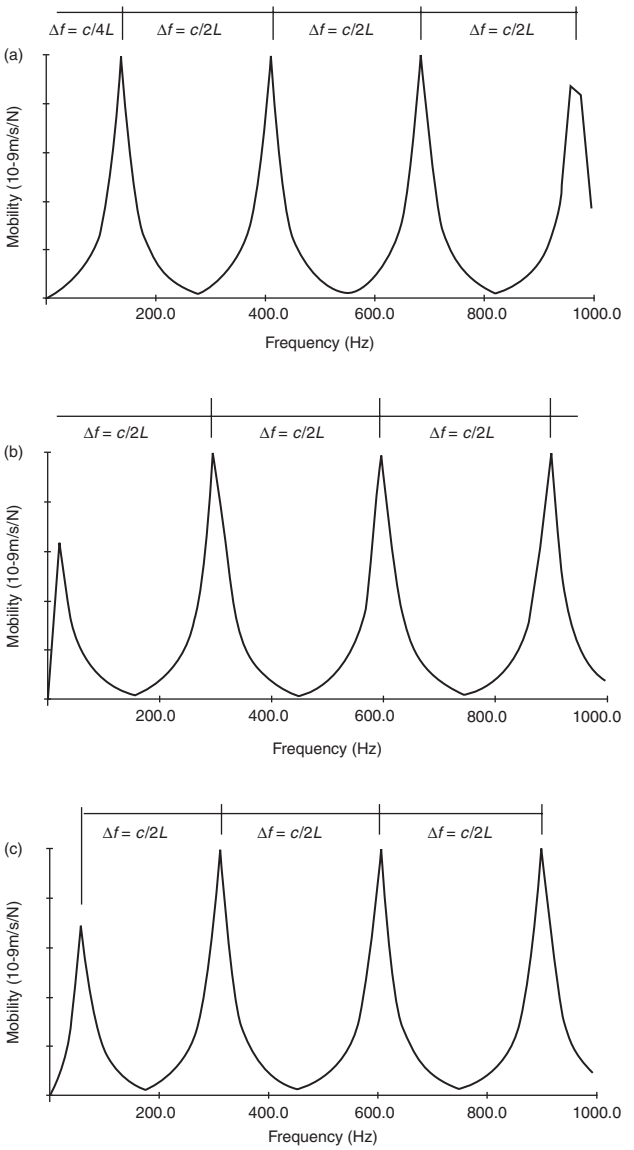


Figure 97.10 Pile with (a) rigid base support; (b) no base support; (c) normal base support

reflection would be received from the toe of the pile by the sensor at the pile head. Hence, no resonances or frequency peaks will be registered, and the signal response will tend towards a single value of mobility, as illustrated in **Figure 97.11(b)**.

The stress wave can be attenuated not only by the surrounding soil, but also by a very long pile, where the wave has to travel through the pile to the toe, before returning to the sensor at the pile head. It can be seen that an infinitely long pile of uniform cross-section would produce no reflection from the toe, and there would be no intermediate impedance changes to cause reflections of the signal. Thus the frequency response curve of a very long pile would therefore also tend to that illustrated in **Figure 97.11(b)**, and the value of the mobility would

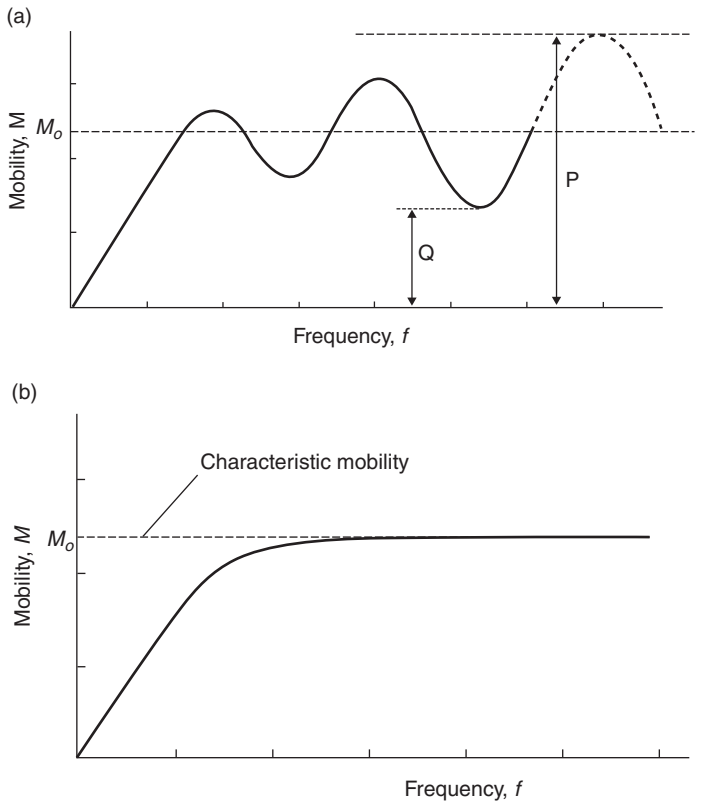


Figure 97.11 (a) Effect of soil damping on signal response curve; (b) Idealised frequency response curve for an infinitely long pile
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reach a constant value. This value is termed the characteristic mobility, M_o, of the pile, and is given by:

$$M_o = L / \rho\, c\, A. \qquad (97.5)$$

The characteristic mobility, M_o, for a particular pile therefore depends only upon its internal properties, and is a unique calculable value. The calculated theoretical value of M_o for a pile tested in the field is often annotated onto commercial signal response curves as an aid to interpretation and comparison.

From a consideration of Equation (97.5), it can be seen that a test value of M_o greater than its theoretical value could be caused by one or more of the pile parameters σ, c or A being lower than its design or intended value. Conversely, a lower value of M_o would imply σ, c or A being greater than their design value: a larger diameter pile, for example.

The above text and figures also clarify that the attenuation of the frequency response curve can be the result of either a shorter pile embedded within a stiffer soil, or a longer pile in a weaker soil. In both cases, the difference in amplitude between maxima and minima (indicated by P and Q in **Figure 97.11(a)**) is reduced, so that the response curve gradually approaches the value of the characteristic mobility of the pile.

In addition, the initial straight line portion of the mobility versus frequency curve is a measure of the *dynamic stiffness*, E, of the pile head. E would be expected to be a specific value for a group or 'class' of piles on a site having the same cross-sectional area, length and unit weight, installed into the same soil stratum. Hence, a pile with a lower dynamic stiffness than its fellows might be considered for further examination or test. Further description of this feature is given in Hertlein and Davis (2006) and CIRIA 144 (1997).

97.4.4.1 Pile impedance and partial reflection

In a similar manner to analysis in the time domain discussed in sections 97.4.2 and 97.4.3, it would be expected that an intermediate feature in the pile, such as a reduction or increase in diameter, which caused only a partial reflection of the stress wave, would be identifiable by its characteristic resonant frequency in addition to the frequency peaks due to, say, the toe of the pile. Hence, the two signals might be superimposed onto the response registered for the pile. This is illustrated in **Figure 97.12**, taken from CIRIA 144 (1997), after Ellway (1987). Ellway noted that the reflection coefficient (the proportion of the signal either reflected or transmitted) across the change of impedance (shown as a bulb in **Figure 97.12**) is also governed by the frequency of the vibration wave. In general, this coefficient increases with frequency, so that a greater proportion of the vibration energy is reflected from the change in impedance interface at higher frequencies. Thus, as seen in **Figure 97.12**, at lower frequencies the full length of the pile can be discerned. At higher frequencies the intermediate impedance change becomes increasingly discernible. Ellway suggested that, for this reason, a frequency-based test should examine the response of the pile over a minimum frequency range of 0–2500 Hz.

Figure 97.13 further illustrates the return signal from a pile with a major impedance change at shallow depth. Section 97.4.5 discusses the interpretation of site data in further detail.

97.4.5 Features detected by low-strain integrity tests

97.4.5.1 Features detected using time-based sonic echo test methods

Figure 97.14 illustrates a typical response from a pile with a previously placed 'neck' defect (from a test site at Blyth, in north-east England, as described by Lilley *et al.* (1987). The first upwards peak is from the hammer impact and the second from the reflection or 'echo' from a change in pile or soil impedance. As the echo is of positive polarity (relative to the hammer impact peak) we know that the returning wave is a tension wave and is therefore a result of a reduction in impedance due to a change in pile or soil properties. The magnitude of the reflection would suggest a shaft feature rather than a soil feature and this is most likely to take the form of a reduction in section or 'necking' of the pile shaft.

The separation of the two peaks is 1.4 ms, which, based upon a wave speed within the concrete pile of between 3500

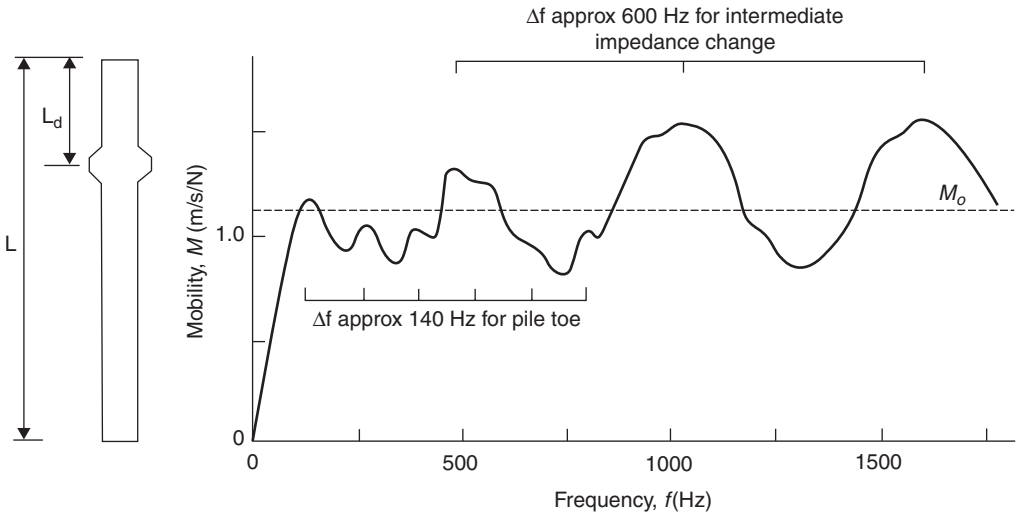


Figure 97.12 Frequency response test, illustrating the effect of an intermediate impedance change
Reproduced with permission from CIRIA R144 (1997), www.ciria.org (after Ellway, 1987)

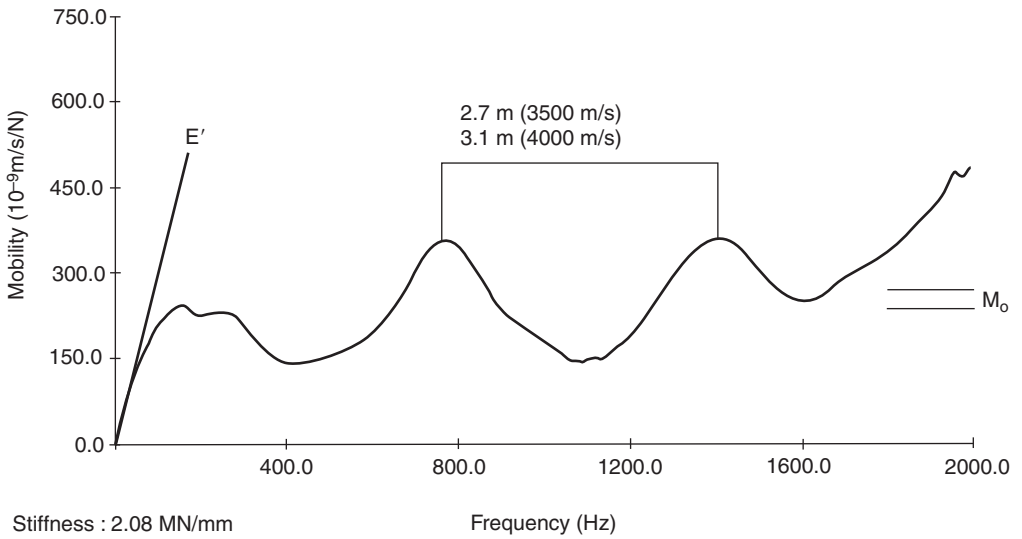


Figure 97.13 Typical site trace of a frequency response test on a pile with a neck defect, showing interpretation of pile head dynamic stiffness, *E′*, and characteristic mobility, M_o
Courtesy of Testconsult Limited

and 4000 m/s, is equivalent to a depth of 2.4–2.8 m below pile head level. The signal response may also suggest an increase in impedance very shortly after (probably as the pile returns to normal section).

There is no obvious toe reflection from the pile, which was recorded as 11.3 m long and installed within very stiff to hard Glacial Till (Boulder Clay). Again, based on a wave speed of between 3500 and 4000 m/s the pile toe would be expected on the signal trace at around 5.5 to 6.5 m/s from the hammer peak. The toe is not clearly discernible because of attenuation of the signal by the strong soil.

Caution should always be taken with this type of result as the reduction in pile impedance suggested may also be associated with an oversize pile section reducing to nominal diameter at the indicated depth. If this is suspected then reference should be made to pile construction records, concrete volume and geotechnical conditions.

The normal course of action with this pile result (once reference to construction records has eliminated an oversize upper section) would be to excavate to the level of the anomaly to inspect the pile shaft or undertake a performance test on the pile. This may take the form of either dynamic load test or static load test.

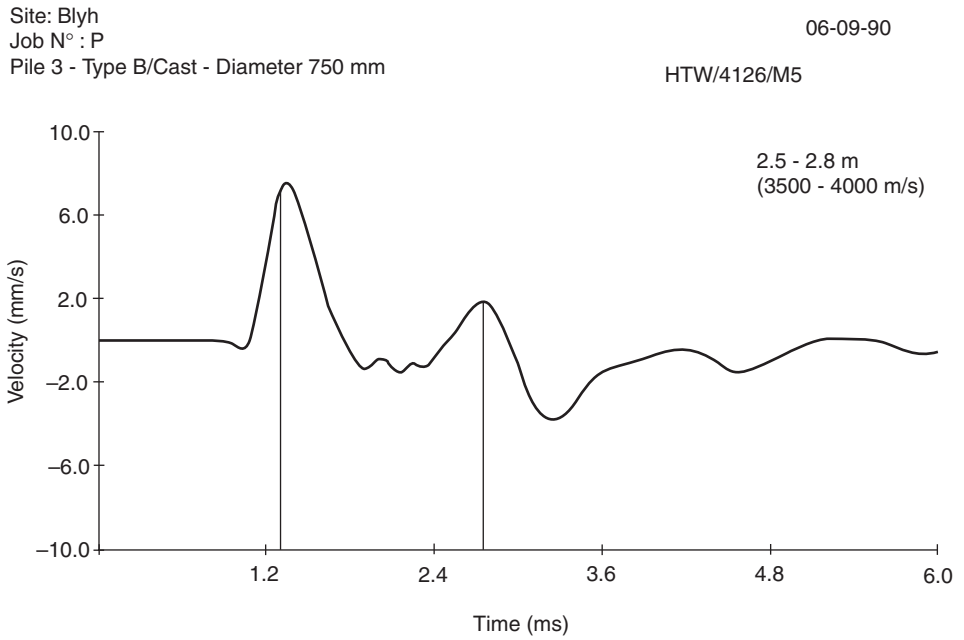


Figure 97.14 Sonic echo response from an anomaly at approximately 2.4–2.8 m
Courtesy of Testconsult Limited

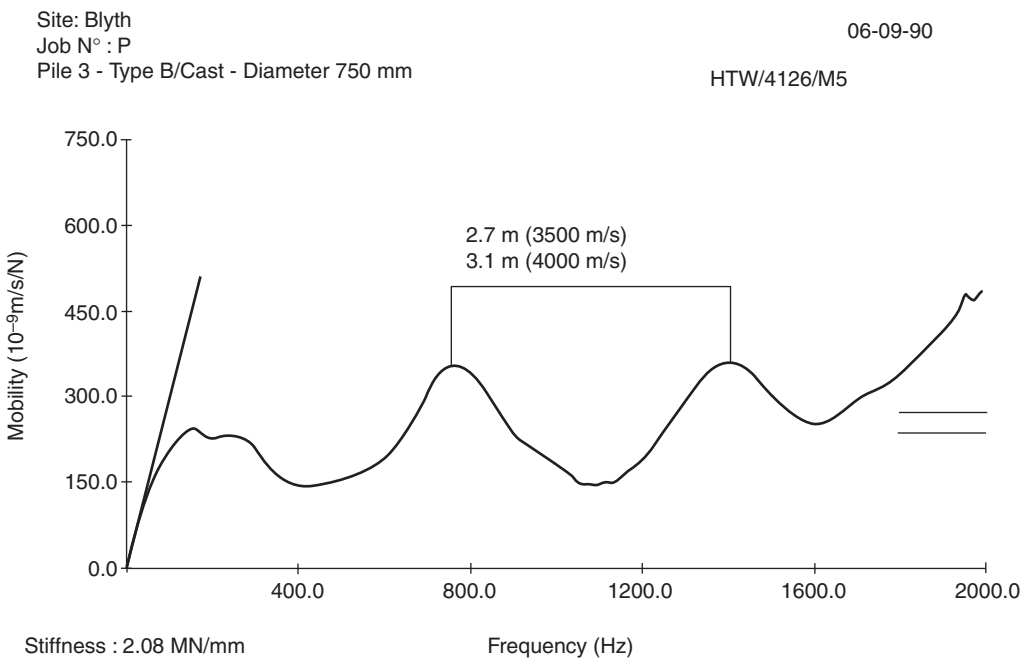


Figure 97.15 Impulse response curve from anomaly at approximately 2.7–3.1 m
Courtesy of Testconsult Limited

97.4.5.2 Features detected using frequency response test methods

The site analyser processes the data from the hammer and velocity transducer and produces a signature response curve showing mobility against frequency. The response curve from the same 'defective' pile is shown in **Figure 97.15**. This also allows a direct comparison of the results when displayed in the time domain and frequency domain.

Interpretation of the response curve would initially be undertaken by measuring the frequency difference between adjacent

peaks. This shows a Δf frequency of approximately 650 Hz. By applying the formula $L = C/2\,\Delta f$, length measurements of approximately 2.7 and 3.1 m can be calculated for the wave propagation speeds (C) of 3500 and 4000 m/s. The phasing of the first harmonic peak is close to Δf frequency (650 Hz) which suggests a reduction in pile impedance. In addition, the pile head dynamic stiffness (E) is calculated to be 2.08 MN/mm and the characteristic mobility (M) calculated to be approximately 300×10^{-9} m/s per N which suggests that the pile has a comparable pile head stiffness to similar pile/soil installations and that pile section, concrete density and wave propagation velocity are within the normal range for the given pile diameter and concrete properties.

Further interpretation of the response curve can be undertaken using numeric simulation techniques. **Figure 97.16** shows a numeric simulation performed on the impulse response curve discussed above: proprietary software is, however, required for this. The simulation includes a reduction in pile section at 2.9 m and shows a close match between the real and simulated response following manual adjustment to both soil and pile variables. The resultant graphics confirm that the inputted localised necking of the pile shaft at approximately 3.0 m is the correct interpretation of the response, as any mismatch of the response curve would require differing pile and soil properties.

Further modelling of the pile section using impedance analysis of the impulse response was first postulated by Paquet (1992). Advances in computer technology have enabled this to become a reality and several equipment manufacturers now include modelling of the pile section in this way. The resulting pile model is termed an impedance-log. The impedance-log technique essentially produces a one-dimensional graphic representing changes in pile impedance.

97.4.5.3 Impedance-log analysis

As noted by Hertlein and Davis (2006) it has always been the ultimate aim of low-strain integrity test specialists to create an image of the as-constructed shape of the pile shaft 'in the ground'. They suggested that the closest to this ideal at that date is the impedance-log analysis method developed by Paquet (1991, 1992). This technique combines the amplified time-domain response from the echo-type test with the characteristic pile shaft impedance measured in the frequency response test. In common with other analytical tests, however, the method is based upon a one-dimensional model, so that an asymmetrical feature such as a partial neck on one side of the pile would not be identified as such. The technique utilises the data from both the time and frequency domains to remove the motion of the pile head caused by the initial hammer blow and the

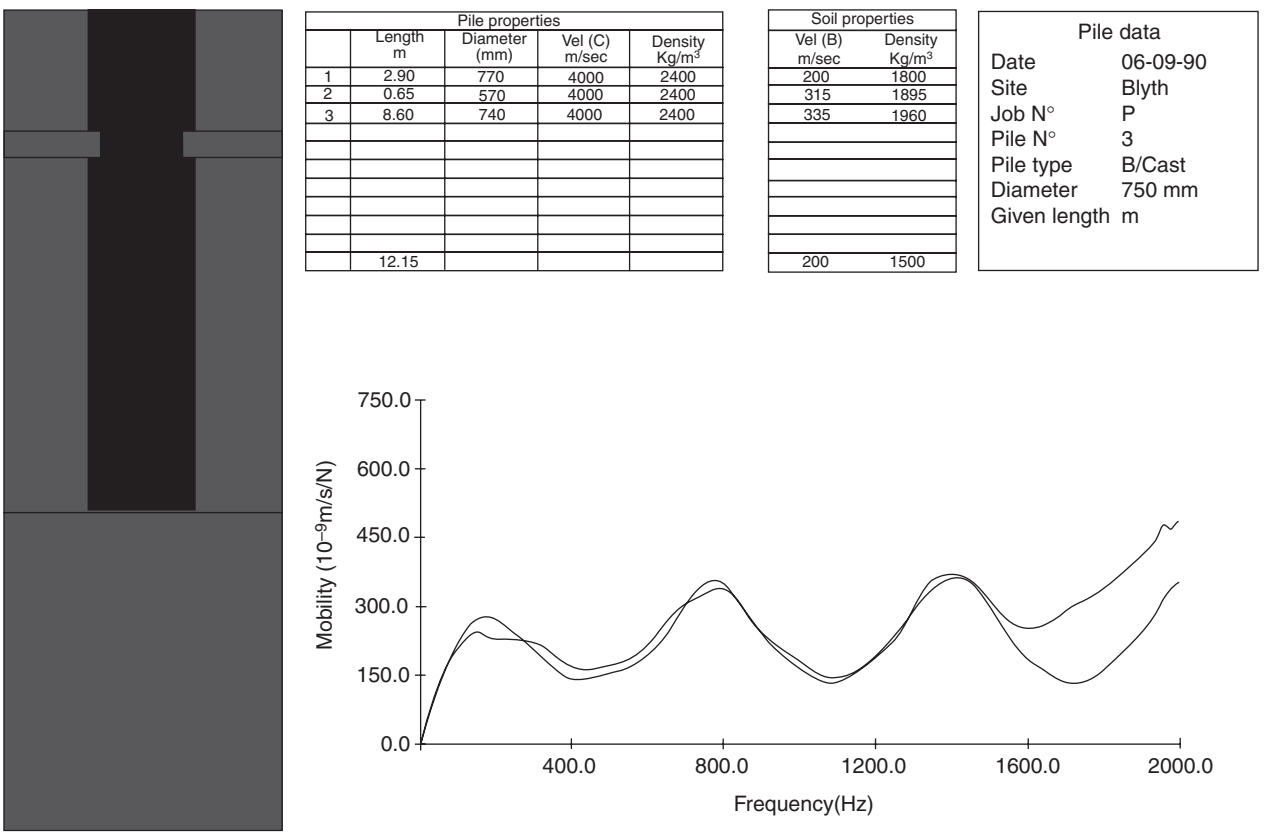


Figure 97.16 Simulation of mobility response and resultant pile section
Courtesy of Testconsult Limited

effect of impedance changes due to the surrounding soil. In principle, this is obtained by calculating a theoretical mobility plot for an infinitely long defect-free pile of the given diameter and predicted ground conditions from the available ground investigation information. In practice this model is typically derived from a simulation analysis on a pile at the site which is considered to be satisfactory. The resulting calculated soil characteristics are then used in the impedance-log model for other piles.

This idealised mobility plot is subtracted from the test response, resulting in a 'reflected' mobility response which contains information about changes in the pile shaft geometry and the surrounding soil. The response in the time domain is then calculated from this plot to produce a 'relative reflectogram', as illustrated in **Figure 97.17(b)**.

Further analysis allows the removal of the soil effects from the model, and the calculation of the impedance down the pile shaft as a function of time, which is equivalent to depth. As described in earlier parts of this chapter, if the density and wave-speed of the pile material are known, or can be reasonably approximated, changes in impedance correspond to changes in the cross-sectional area of the pile, and the resulting impedance-depth relationship can be plotted in terms of impedance versus either the diameter or cross-sectional area of the pile.

A resulting pile impedance-log analysis is shown in **Figure 97.17(c)**. This analysis was undertaken on a pile on a test site at San Jose, California, which was constructed with known defects. It can be appreciated that for such analyses it is most important to have full pile construction details and soil data.

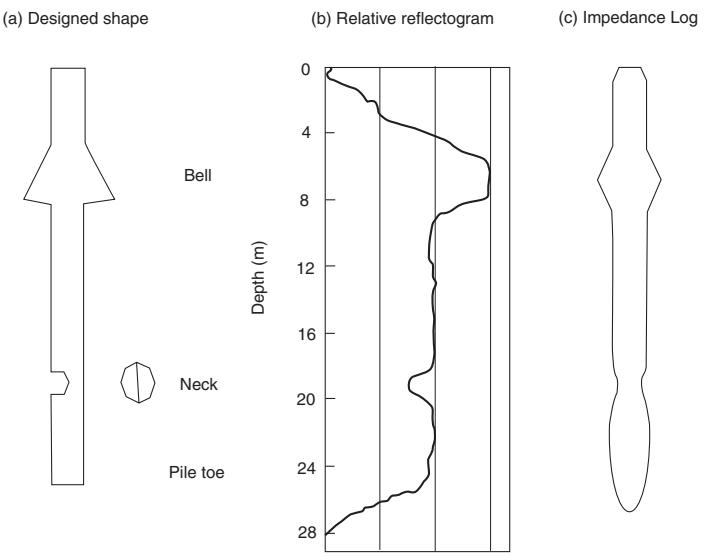


Figure 97.17 The impedance-log and relative reflectogram calculated from the velocity-time reflections for a pile constructed with artificial pile defects
Reproduced from Hertlein and Davis (2006), John Wiley

Further description of the technique is given in Hertlein and Davis (2006).

97.4.6 Analysis, interpretation and reporting of low-strain integrity tests

CIRIA 144 (1997) identified two main fields of use for low-strain integrity testing:

(a) Routine control testing, as part of a pre-planned site quality control regime.

(b) Retrospective investigations, where a problem has become apparent during the works: often while piles are being cut down ready for incorporation into the foundation system. A further use of such testing would be where it was proposed to re-use existing piles following demolition or alterations to an older structure (see Chapman *et al*., 2007 or Butcher *et al*., 2006, for example), or where the testing formed part of a forensic investigation into a possible foundation failure.

In the case of routine control testing, CIRIA 144 (1997) suggests that it should be reasonable to expect that most of the signals from such integrity tests should be understandable to other non-specialist engineers involved with the project. Since the results are of interest, and importance, to a number of contracting parties within the project, CIRIA 144 (1997) suggested that, for a testing programme to be useful, it was also important that the engineering advisers to those parties were able, with proper guidance, to agree to the validity of the basic information.

It is also imperative that such engineers are aware that the results of integrity testing should not serve as the sole basis for the acceptance or rejection of any particular pile. Other information, such as ground conditions, construction records and on-site observations, should also be considered. Crucially, if the interpretation of the test data is perceived by others to be subjective, or even whimsical, then there will be a corresponding decrease of confidence in its use as a quality control tool.

Again, it is important that the non-specialist engineer should be aware that, in practical terms, in some situations the data will allow no clear interpretation.

In an attempt to resolve this problem CIRIA 144 (1997) proposed a signal classification system for both sonic echo and frequency response tests. This was intended to differentiate the simpler signal responses from those that are more complex.

97.4.6.1 Classification of signal responses

CIRIA 144 (1997) classification

To attempt to resolve this problem, CIRIA 144 (1997) suggested a three-fold classification of signal responses:

Type 0 – no distinct return signal received, i.e. no significant impedance change within the pile, for the depth penetrated by the test;

Type 1 – one clear major response, i.e. one significant impedance change within the pile;

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Type 2 – more than one major response, i.e. two or more impedance changes within the pile.

The features of these three categories can be summarised as follows:

Type 0 signal

A Type 0 signal would be one in which the damping effect of the surrounding soil has attenuated any return signals to such an extent that the toe cannot be discerned. Therefore, there is no significant impedance change within the effective penetration depth of the system. The non-specialist engineer would easily appreciate the reason for this, provided that the basic principle of the test was understood.

Figure 97.18 illustrates typical examples of Type 0 signals for time-based and frequency-based systems.

Type 1 signal

A Type 1 signal would contain one clear, major response, indicating that the shaft was responding as a single acoustic unit. This would represent a pile containing a single major impedance change, either from the pile toe or some significant intervening feature. No other significant response would be visible on the recorded trace. Type 1 signal responses should be very similar to the theoretical simple signal expected from the test and be easily recognisable. Hertlein and Davis (2006) also suggested that such a signal response would also be easily simulated by computer modelling.

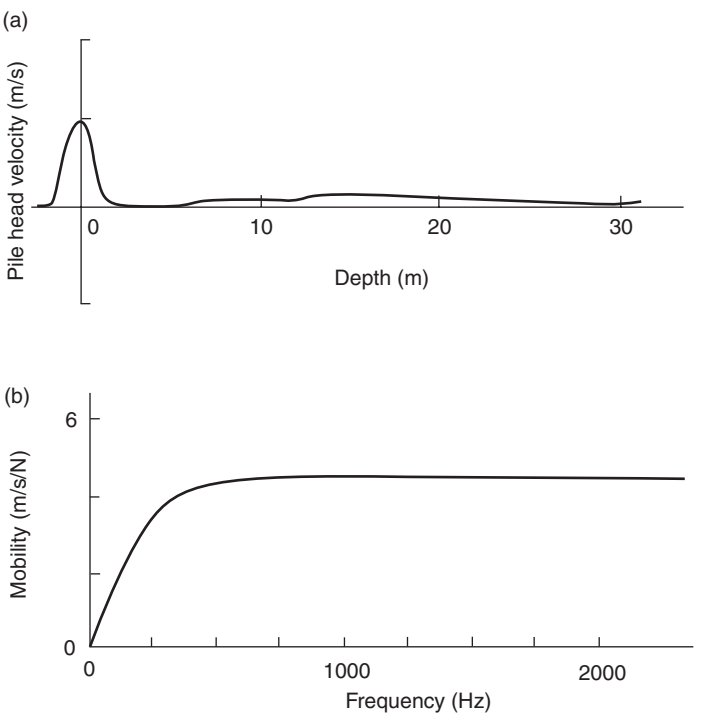


Figure 97.18 Examples of Type 0 signal response: (a) time-based (sonic echo) test; (b) frequency-based (frequency response) test
Reproduced with permission from CIRIA R144 (1997), www.ciria.org

Typical Type 1 signal responses would be like those shown in **Figure 97.19**.

Type 2 signal

A Type 2 signal would be one containing more than one major response, so that the interaction of overlapping responses from different levels within the shaft would make interpretation of the resulting response a complex matter. At one extreme Type 2 signals might display a clear major response from the length of the shaft responding as a major acoustic unit, but with intermediate responses due to local changes in shaft impedance within that acoustic unit, as shown in **Figure 97.20** and in **Figure 97.12**. At the other extreme, Type 2 signals might contain no clear major response to indicate if part of the shaft is responding as a single acoustic unit, as indicated in **Figure 97.21**.

A Type 2 signal would require interpretation by a specialist, because simple models do not easily explain the response. A summary of the CIRIA 144 (1997) classification is provided in **Table 97.2**.

Classification of the signal responses obtained from the piles on an individual site in this way will help to assess how much

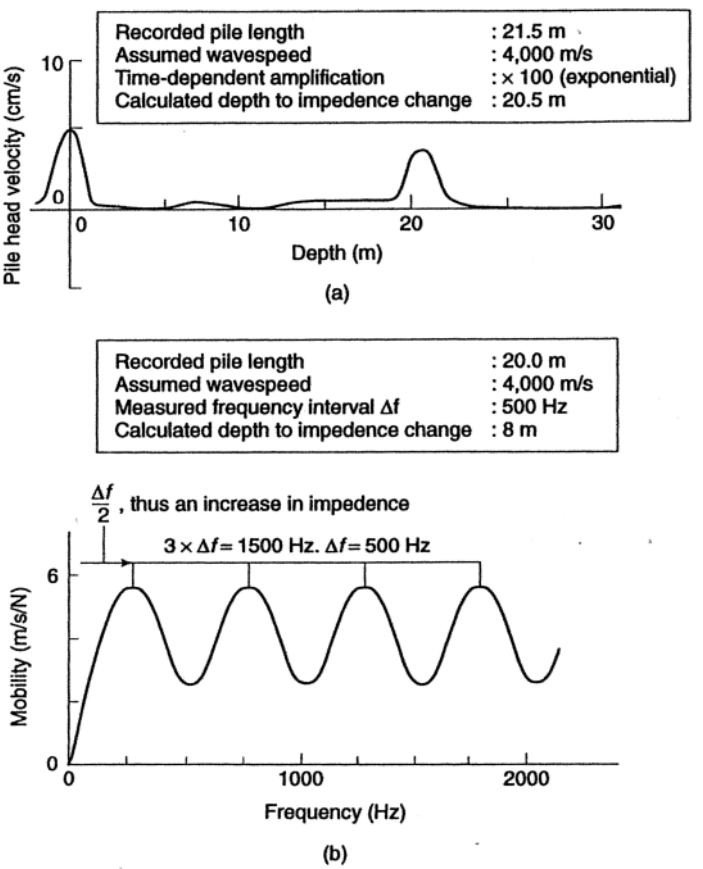


Figure 97.19 Examples of Type 1 signal response: (a) time-based (sonic echo) test; (b) frequency-based (frequency response) test
Reproduced with permission from CIRIA R144 (1997), www.ciria.org

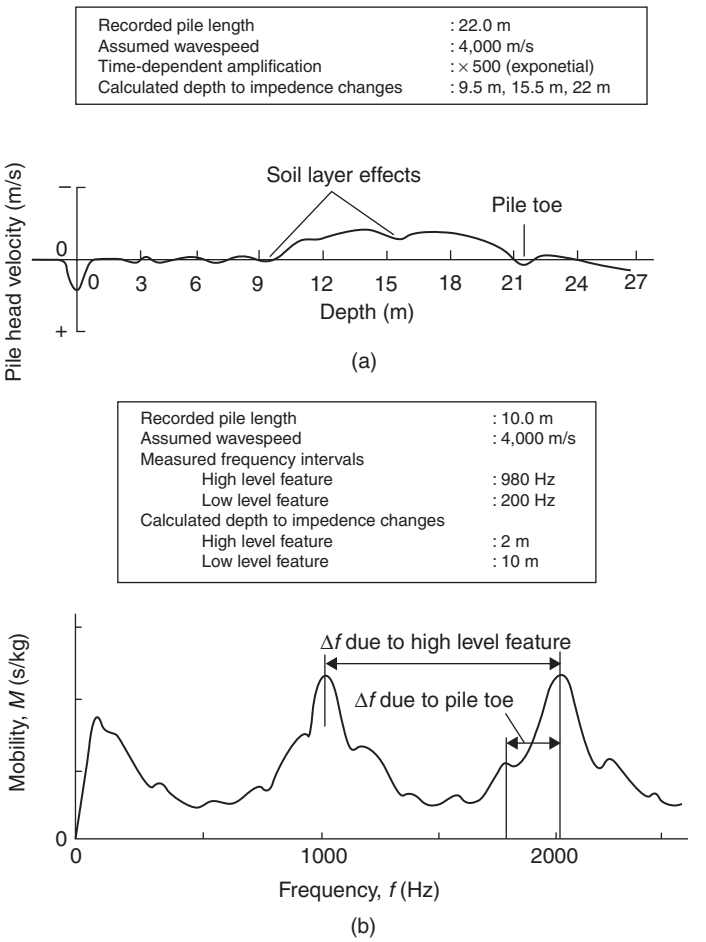


Figure 97.20 Examples of 'clear' Type 2 signal response, clear toe response and additional secondary response: (a) time-based (sonic echo) test; (b) frequency-based (frequency response) test (note: sonic echo test shows inversion of y-axis, so that initial hammer blow response is shown pointing downwards. This is a feature of the particular test system)

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weight should be given to the test results for the piles on that site. The quality of the test results will depend upon:

- The characteristics of the test system, particularly its dynamic range (the ratio of the specified maximum value and the minimum detectable value of the parameter being measured), its resolution and its signal-to-noise ratio.
- The pile characteristics, especially the length/diameter ratio, the quality of the pile material and the shape of the pile.
- The nature of the surrounding soil: the stiffer the soil, the greater the signal attenuation. In addition, a boundary between soils of different relative stiffness acts as a reflective layer or impedance change within the shaft–soil system.
- The quality and standard of the pile head preparation for the test.

It is important to note that CIRIA 144 highlights that there is a distinction between the analysis of the signal (i.e. what it is

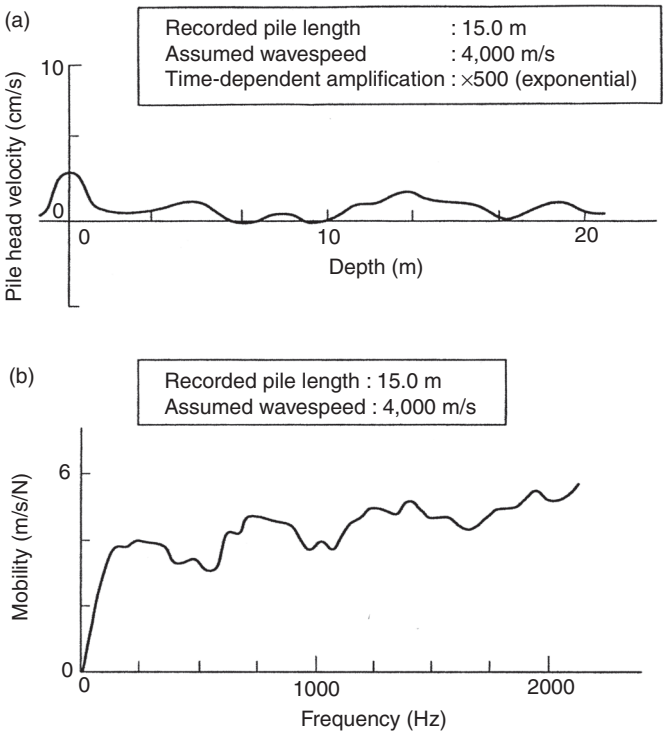


Figure 97.21 Type 2 signal response, no clear response

showing or saying) and its interpretation (i.e. what it means). The CIRIA (1997) classification 0, 1 or 2 is intended as a preliminary *analysis* of the characteristics of the signal: so that the tester can identify to the non-specialist engineer what are the general features of the signal response for each pile. The next step in the process is the tester's *interpretation* of the signal (what it means, and its significance).

An example of this *interpretative* aspect of the analysis process, which has been developed by Testconsult Ltd, in the UK, is outlined below.

97.4.6.2 Testconsult interpretative classification

The interpretative classification system developed by Testconsult seeks to group results from frequency response testing into seven categories as below:

- CAT 1 – Full pile length measured;
- CAT 2 – Damped response;
- CAT 3 – Pile with overbreak;
- CAT 4 – Oversize pile reducing to nominal diameter;
- CAT 5 – Pile with crack in upper shaft;
- CAT 6 – Pile with reduction in properties, i.e. necking;
- CAT 7 – Invalid result due to poor head condition.

It can be seen that a combined CIRIA/Testconsult classification might be, for instance a CIRIA 1 + Cat 1 signal response.

Signal type	Characteristics of signal	Signal responses obtained from		Remarks
		Remote end	Intermediate sources	
0	No impedance change within depth of penetration of signal	Not distinguishable because of soil damping or other effects	Not distinguishable because of soil damping or other effects, or not present	The reason for the response is readily understandable to the non-specialist engineer. Caution required in evaluating depth of penetration of signal
1	One clear impedance change	Clear major signal, indicating the pile system is responding as a single acoustic unit	Absent or very subordinate. Less than 50% of relative magnitude of remote end signal	The response is readily understandable to the non-specialist engineer
2	More than one impedance change	(1) Clearly discernible, reasonably prominent signal	Moderate to strong signals, but not completely obscuring the remote-end response	The response is understandable to a non-specialist engineer only with expert assistance
		(2) Not clearly discernible, because it is similar to or weaker than other parts of signal from intermediate levels	Signals of similar strength to or stronger than remote-end response	The response is not capable of interpretation by a non-specialist engineer

Table 97.2 Summary of CIRIA classification of low-strain integrity test signals
Reproduced with permission from CIRIA R144 (1997), www.ciria.org

97.4.6.3 Analysis and interpretation

As outlined above, CIRIA 144 (1997) emphasises that there are two stages to the interpretation process. These can be summarised as:

(1) analysis of the acoustical data;

(2) interpretation of the significance of this analysis, taking account of all other relevant information for the pile.

The analysis of the acoustical data includes the identification and depth of impedance changes within the signal response curve, and the evaluation of these to determine whether they are relative increases or decreases in impedance.

The interpretation should take account of the site geology and ground conditions, the pile construction records, and particular features of the piling system (such as the use of permanent casing to the upper section of the pile) or the like.

CIRIA 144 (1997) provides further insight into the way this information should be used by the interpreter to provide a reasoned explanation of the condition of the piles. It emphasises that the purpose of such testing is to help evaluate the pile, not the testing system, so that as much information as possible should be given to the specialist to assist in interpretation.

97.4.7 Testing within the contract

CIRIA 144 provides extensive guidance on the use of control testing within the framework of the contract and the constraints of an active working site. It emphasises that many parties have an interest in the results of such testing but the lines of responsibility and communication are not always clear. This applies especially to low-strain integrity testing, but has application to cross-hole sonic logging also.

Hertlein and Davis (2006) also highlight this problem from US experience, and point out that NDT, when used in quality assurance testing of piles and other foundation systems, can be viewed from two extreme viewpoints. They suggest that NDT is either

- to assist *engineers* to confirm that the design and performance criteria have been met, or
- to check that the *contractor* has supplied the *owner* with the product that the owner has paid for (material quality, minimal geometric requirements, etc.).

From the first point of view, certain deviations from construction specifications can be tolerated and compensated for, provided that there is strong interaction between *engineer*, *contractor* and *tester*. In the second case, no margin for error outside the contract specifications can be allowed, since they are effectively 'set in stone'.

Davis (1998) contends that the NDT methods now available are more suited to the first scenario (performance specification). This is because the variables inherent in drawing conclusions from NDT on piles are numerous, and it is not always possible to clearly define these variables to reach a complete answer to satisfy the materials specification.

In UK practice, there is typically a view that such testing is intended to 'verify' that the construction of the piles is acceptable. However, there is almost always no contractual acceptance criterion against which 'verification' can be judged. Hence acceptability or otherwise of a particular pile or piles becomes a matter of argument or 'discussion'.

UK practice has also tended towards the specifier of the test being the technical representatives of the employer, owner, developer or funder of the project. Unfortunately, in the authors' opinion, the NDT contract is almost always included

within the piling contract. This means that the test contractor must report test results directly to the entity most likely to suffer monetary cost and disruption if adverse results are reported. It is not unusual that the test contractor is specifically instructed not to report to or discuss the test results directly with the employer's technical representative: either by the piling contractor or the general contractor to whom the piling contractor is sub-contracted.

This procurement method has led not only to a 'least cost' rather than a 'best specialist' approach, but also to the production of specialist reports which tend to give as little information and comment as possible about the results obtained.

97.4.8 Guidance on the specification of low-strain integrity testing

In the UK, the Institution of Civil Engineers' (2007) SPERW document provides some useful guidance on the specification for the integrity testing of piles and embedded retaining walls, together with some accompanying guidance notes. Unfortunately, there is some confusion in the document on the identification of the various test methods: partly due to the use of proprietary names rather than the generic terms adopted in CIRIA 144 (1997). In addition, practice, equipment and techniques have advanced further since CIRIA report 144 was first drafted.

Both CIRIA 144 (1997) and Hertlein and Davis (2006) provide guidance on the specification of low-strain testing. Hertlein and Davis in particular provide sample specifications for both low-strain testing and cross-hole sonic logging.

It is not appropriate in this particular chapter to provide a detailed specification, but is probably useful to highlight crucial points to be borne in mind when specifying such tests.

97.4.8.1 Type of test

It can be seen from the earlier discussions that there is a general hierarchy of technical quality as a result of the methodology and techniques employed. In increasing sophistication these can be identified as:

- time-based 'echo' techniques;
- frequency-based techniques;
- frequency-based techniques with simulation modelling, including impedance-log analysis.

In this context it should be appreciated that time-based techniques only examine data in the time domain. Frequency-based techniques can examine the data in both the time and frequency domains and also monitor the input force from the test hammer. Simulation modelling techniques attempt to remove extraneous information from the signal: such as that caused by the influence of the ground within which the pile is embedded, for example.

It is important that the specifier is aware of and appreciates these differences. The more knowledgeable the specifier the better they can target the testing. It is also important to realise that a more sophisticated test, with a correspondingly more complex analysis, should be expected to be more expensive. Hence, the selection of a test organisation on price alone may not be appropriate in many cases.

An important difference between time-based and frequency-based techniques, which may be of major interest to the end user, is that the examination of the data in the frequency domain can assist in examining anomalies close to the pile head. This is because the stress wave imparted by the hammer blow would typically be 1.0–1.5 ms long, which is equivalent to a wavelength of around 2–3 m in a concrete pile. Hence reflections returning from a near-surface anomaly can overlap with part of the downward-travelling wave from the initial hammer blow. The resulting complex signal can be difficult to differentiate and interpret in the time domain. By comparison, in the frequency domain, such a feature would be expected to be visible at a frequency interval of 666–1000 Hz, and might be more readily discernible (see **Figures 97.12** or **97.20(b)**, for example).

As a generality, a defect close to the pile head would often be of greater concern to the design engineer, especially if the bulk of the pile load is carried by shaft friction rather than in end-bearing.

97.4.8.2 Experience of testing specialist

Research testing in the US (e.g. Baker *et al*., 1993; Iskander *et al*., 2001) indicated that integrity testing methods were quite dependent upon the skill and experience of the test operator. Hertlein and Davis (2006) suggest that field testing and preliminary interpretation should be undertaken by an experienced operator with at least one year's experience with the particular method employed. They further suggest that final interpretation and reporting should be performed by, or under the direct supervision of, an engineer or senior technician with at least three years' experience with the specific method being employed.

The testing specialist should be expected to have such experienced and qualified personnel on its staff. It should also be able to provide computer simulations of the anticipated response of the piles at the subject site, given details of pile characteristics (length, diameter, concrete and reinforcement details), cut-off levels and ground conditions from test level.

It should also be able to provide a sample test report, with full explanatory annotation as necessary. The selection of a suitable test specialist is of paramount importance to all parties, and the depth and coverage of a sample test report can be a guide to the test organisation's understanding and ability in this highly specialist field.

97.4.8.3 Pile head preparation

Pile head preparation is most important to the quality of the data acquired for subsequent analysis. It is to be expected that most integrity test houses can provide guidance on pile head

preparation, and for best results this advice should normally be followed.

97.4.8.4 Equipment

Current recommendations suggest that the measuring and data acquisition systems should have certified dynamic ranges of at least 60 dB and flat frequency responses over a minimum bandwidth of 20–2500 Hz, and the analogue to digital resolution should be no less than 16 bits.

All equipment should be provided with calibration certificates traceable to UKAS standards. Impulse hammers, velocity transducers and the data acquisition system should have re-calibration checks at intervals of no more than 12 months.

97.4.8.5 Reporting of test results

A reporting timescale should be specified. This would typically include preliminary 'overview' reporting at the time of test or within 24 hours, with the issue of a formal analysis and evaluation report within three to five working days.

The testing specialist's report should include signal response graphs for each pile, as applicable, and an analysis of the response of each pile.

Analysis and interpretation of the test data for each pile by the testing specialist should include:

(1) whether a clear response has been detected consistent with the level of the pile toe, or any other level within the pile;

(2) whether any secondary responses have been detected;

(3) whether the responses are due to an increase or decrease in pile impedance, and a view of their magnitude and importance;

(4) an interpretation of the cause and significance of these responses or 'features';

(5) recommendations for any further investigation or measures if required.

97.4.8.6 The review of acceptance or rejection criteria for the piles, based upon integrity testing

It is important that the specifying engineer, the main contractor and the piling contractor have in mind how they are going to deal with the results obtained from the testing, and the specification should address this point. It should be appreciated that this is not a minor matter. There is often a major cost implication in terms of delay and disruption to the contract progress, quite apart from the costs of remedial works to the pile(s).

Hertlein and Davis (2006) suggest the following approach:

- If a clear response consistent with the level of the pile toe is detected, with no significant secondary responses from levels above this, then the pile would be deemed acceptable in terms of the integrity testing.
- If no clear response from the pile toe is detected, with no significant secondary intermediate responses above the level of the pile toe, then the supervising engineer of the test house as identified in section 97.4.8.2 above (the experience of the testing specialist), should report what length of the foundation pile may be considered to be proven, based on their interpretation of the test data.
- Reflections interpreted to be from any level above the level of the toe of the pile should be analysed by the testing specialist to determine whether the reflection is from a significant reduction or increase in impedance, since these are consistent, in the main, with a corresponding reduction or increase in the pile cross-section. The probable/possible nature and significance of the reflector should then be assessed, taking into consideration the intent behind the design of the pile. The decision to accept or reject the pile or to require any further investigative works should be made by the engineer responsible to the client for the overall design of the structure.

If the test result is deemed to be inconclusive, the pile head may be trimmed back and the pile re-tested to try and provide more conclusive data. If this is impractical or is not successful, additional testing by other means may be instructed. Such measures would include excavation to expose the area of concern, coring the pile, or undertaking dynamic or static load-testing. Such measures are reviewed in CIRIA 144 (1997).

97.4.8.7 Timing of the testing

CIRIA 144 (1997) discusses the potential clash of interests between the contractor, who might to prefer to test early, and the client, who is really most concerned about the pile immediately before it is incorporated in the works. However, the later the test, the greater is the potential disruption to the overall contract if problems are found. The specifier should address this point, and the specification should be clear on the requirements. As a guide it is not unusual to require that the testing of any pile should be undertaken within 28 days of installation, and that all testing should be complete within 14 days of the installation of the last pile. For cast-in-place piles, it is usual to specify a minimum curing period before testing. This would typically be between three and seven days depending on the characteristics of the concrete mix.

97.5 Cross-hole sonic logging

97.5.1 Introduction

The method is a development of the ultrasonic pulse velocity (UPV) test method commonly used to determine ultrasonic transit time in concrete. The UPV test method is described in detail in BS EN12504-4:2004.

Vertical ducts are cast into the pile over its full length. Two probes, one a transmitter and one a receiver, are lowered to the bottom of adjacent ducts. An ultrasonic pulse from the transmitter is detected by the receiver and the signal is captured by the data acquisition system, as illustrated in **Figure 97.22**.

A continuous series of such measurements is taken as the probes are simultaneously raised up the ducts, so that a

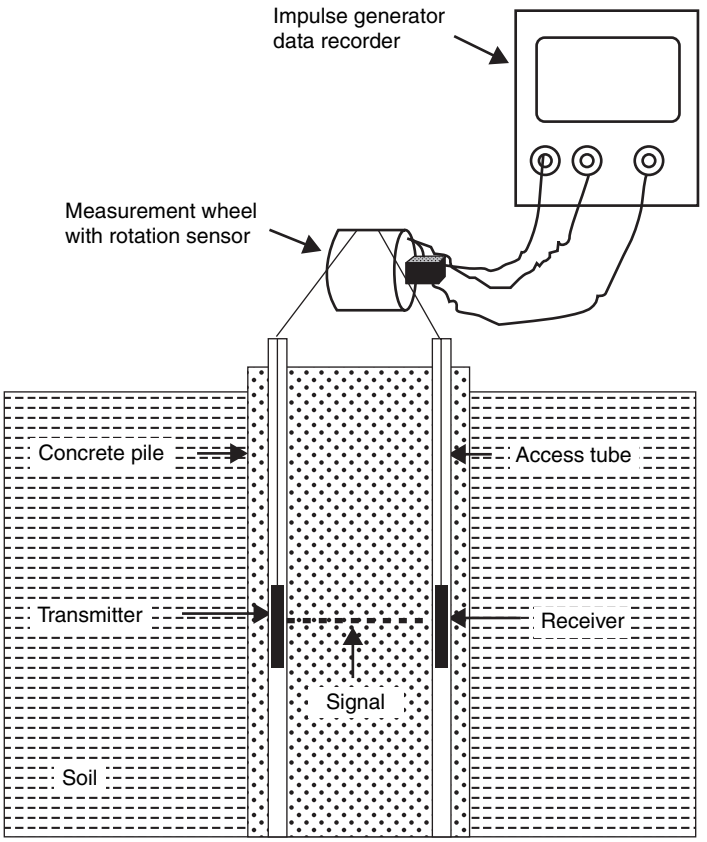


Figure 97.22 Schematic of cross-hole sonic logging system
Reproduced from Hertlein and Davis (2006), John Wiley

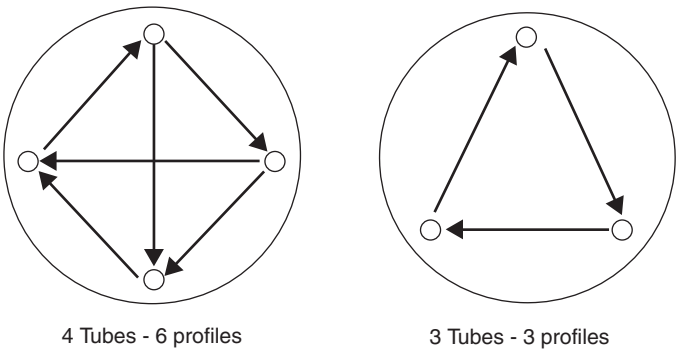


Figure 97.23 Plan view of cross-hole sonic logging access tube configuration

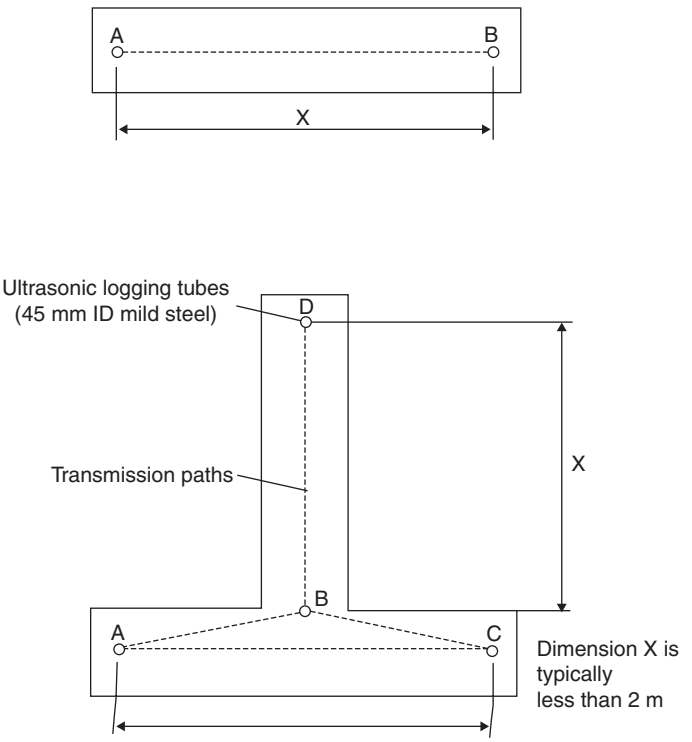


Figure 97.24 Plan view of sonic logging ducts within diaphragm wall panels
Reproduced from Stain and Johns (1987), Deep Foundations Institute

continuous profile of the transit time and characteristics of the wave-train generated by each pulse is built up at successive levels in the pile.

97.5.2 Description of the technique

Cross-hole sonic logging is usually used on piles with a diameter of 600 mm or greater. Steel or, less usually, plastic access tubes are cast into the pile during the construction phase. However, plastic tubes can often suffer from debonding from the concrete, rendering the test ineffective. Hence steel tubes are preferred in most cases. They are typically attached to the inside of the reinforcing cage over the full length of the pile and extend above ground level by approximately 500 mm. A minimum of three tubes are used, usually increasing to a minimum of four for pile diameters of 750 mm or more. The tubes are usually equispaced around the circumference of the reinforcing cage to provide a minimum of three longitudinal 'profiles' of the concrete between the tubes for a three-tube configuration, and up to six for a four-tube configuration as illustrated in **Figure 97.23**. The steel tubes are generally approximately 40 mm in internal diameter to facilitate the passage of transmitter and receiver transducers.

Prior to the test, the tubes are filled with clean water to act as a coupling medium for the transducers.

The technique is also suitable for use in diaphragm wall panels or barettes. A typical layout of ducts within diaphragm wall panels is illustrated in **Figure 97.24**.

The transit time (t) of the wave-train from each pulse is a function of the distance between the tubes (L) and the propagation velocity (c) of the concrete between the piles. In turn, the propagation velocity of the pulse is a function of the modulus, E, density, ρ, and Poisson's ratio, ν, of the pile concrete. Hence

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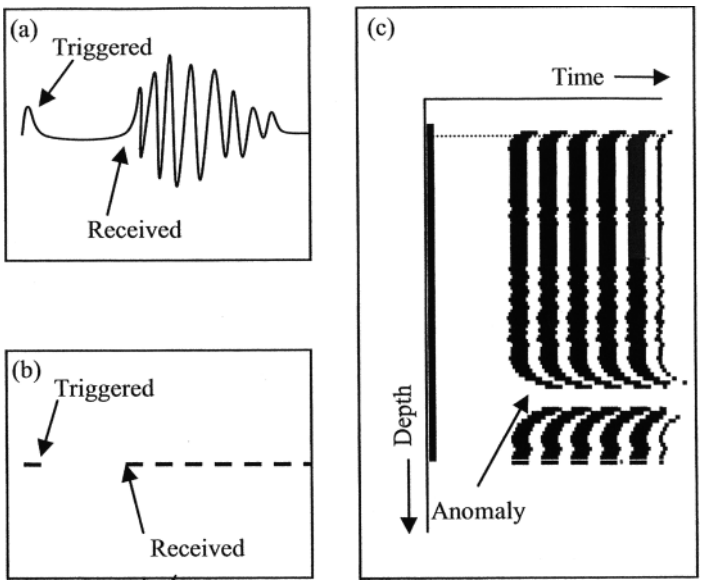


Figure 97.25 Compilation of a cross-hole sonic logging plot: (a) Original single pulse; (b) Pulse data modulated to form a single positive/negative dashed line; (c) Modulated pulses stacked to form the complete 'waterfall' plot

Reproduced from Hertlein and Davis (2006), John Wiley

anomalies within the pile, such as voids, soil inclusions or low-modulus concrete, would be expected to give an increase in the transit time: provided that the access tubes remain at a constant relative position apart (i.e. L remains constant). In addition, if the anomaly was caused by voided or poorly compacted material, for instance, this might also lead to an attenuation of the signal.

The two probes (transmitter and receiver) are slowly raised up the tubes, and a reading is taken every 20 mm. Each received ultrasonic pulse is collected and the wave-train would have the form illustrated in **Figure 97.25(a)**.

With the system developed by CEBTP, each received ultrasonic pulse is processed by the data acquisition system so that each 'positive' peak of the wave-train is printed as a black line with a width matching that of the original peak; conversely each 'negative' trough is printed as a gap. Hence a dashed line is created as illustrated in **Figure 97.25(b)**, which contains all of the timing and much of the amplitude information of the original record. Each consecutive pulse is printed contiguously to form a vertical profile of the data for that particular pair of tubes, as illustrated in **Figure 97.25(c)**. This profile is often known as a 'waterfall plot' or a 'sonic profile'. In this way any changes in transit time can readily be distinguished from the mean value and identified for further investigation.

Other cross-hole sonic logging systems measure the arrival time of the first peak in the ultrasonic pulse wave-train, known as the first arrival time, or FAT, together with the overall amplitude of the early part of the pulse, as illustrated in **Figure 97.26**.

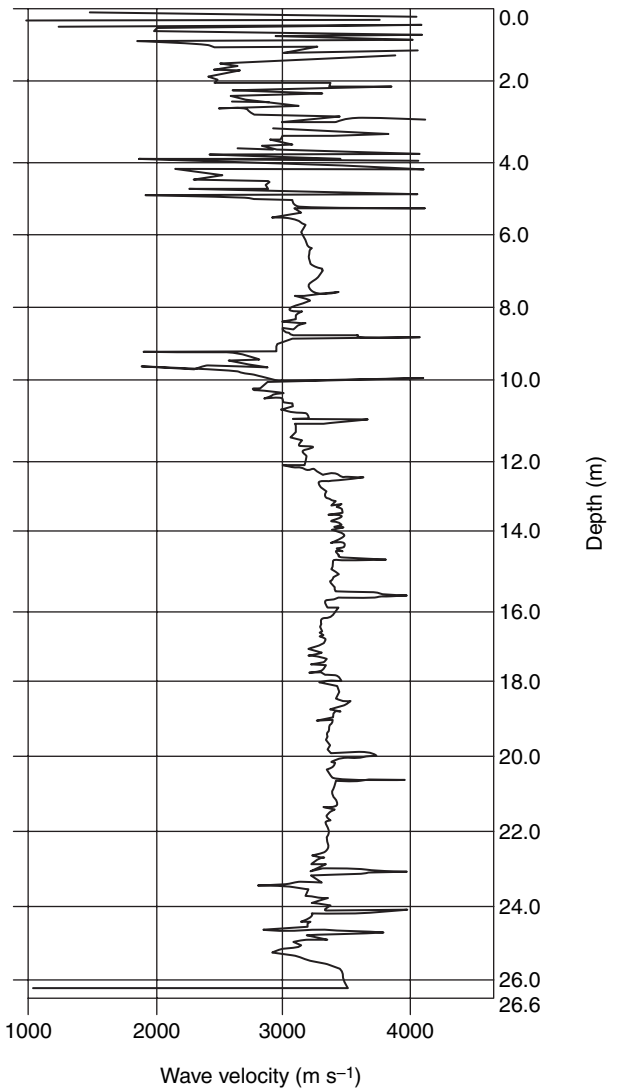


Figure 97.26 Typical FAT (first arrival time) cross-hole sonic logging plot

Reproduced from Hertlein and Davis (2006), John Wiley

Hertlein and Davis (2006) suggest that a drawback of this method is that the system has to detect the first arrival peak in a digitised wave-train, and has to be able to differentiate between background noise and the arriving signal. Thus a threshold level has to be set that eliminates the background noise, as illustrated in **Figure 97.27**. Unfortunately the signal pulse is a relatively small 'noise', and construction sites tend to be noisy. Hence if the first or second peak in the arriving wave-train falls below the threshold level, the computer will identify the 'first arrival' as the first peak that exceeds the specified threshold, be it the second, third or even later arrival.

97.5.3 The detection of defects

As noted in CIRIA 144 (1997), the test identifies the shortest acoustic path between the transmitter and receiver.

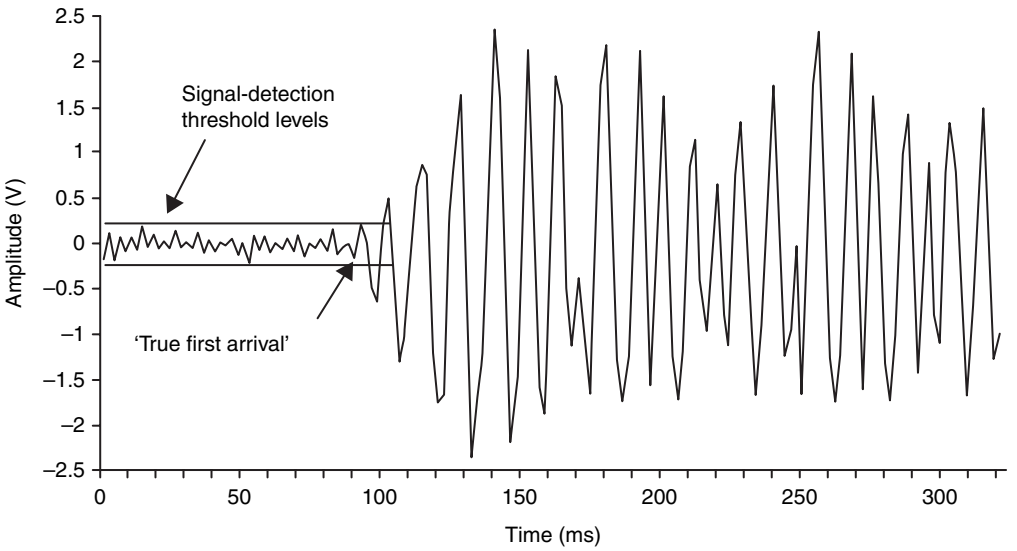


Figure 97.27 Typical cross-hole sonic logging pulse, with illustration of signal threshold levels and 'first arrival time'
Reproduced from Hertlein and Davis (2006), John Wiley

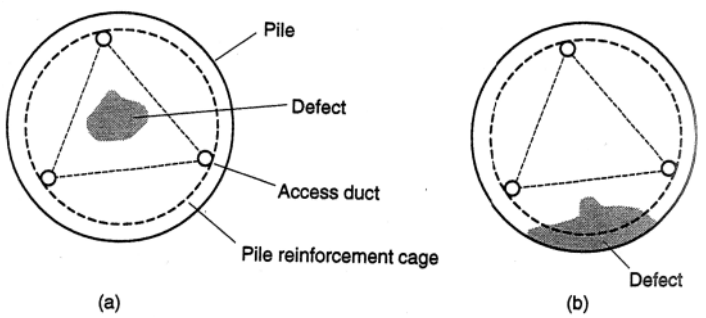


Figure 97.28 Cross-hole sonic logging. Examples of possible defects off the shortest direct path between probes: (a) defect in centre; (b) defect on perimeter
Reproduced with permission from CIRIA R144 (1997), www.ciria.org

With a three-tube system, for example, it is possible that a feature at the centre of the pile would not be detected, as illustrated in **Figure 97.28(a)**. Similarly, since the access ducts are usually attached to the pile reinforcement, a defect such as loss of concrete cover would not typically be detectable as in **Figure 97.28(b)**.

97.5.4 Features detected using the cross-hole sonic logging method

A typical test result from a defective diaphragm wall section is given in **Figure 97.29**.

Interpretation of the results is generally straightforward when signal loss increases dramatically but the threshold for what is considered to be anomalous concrete is less clear. In practice an increase in transit time of less than 10% is not considered to be significant unless all profiles are affected. If this is the case then further investigation may be required which could include coring the pile, reference to the pile construction records, test cube analysis and further clarification of axial loading and bending moments on the pile.

The association between ultrasonic propagation velocity and concrete strength has been advocated in several technical papers and an argument does exist to suggest that this may be applied to data collected from CSL data. Tube spacing is, however, an additional variable which needs to be taken into consideration. For further reading on the correlation between concrete strength and ultrasonic pulse velocity, refer to Turgut (2004).

Signal damping is often a feature of the CSL results and this may be due to debonding of the access tubes or a thin layer of contaminated concrete surrounding one or more of the tubes. Signal damping with no associated increase in transit time is not considered to be significant.

Three-dimensional computer tomography has been introduced by several equipment manufacturers since 2000. As noted by Fleming *et al*. (2009), this can model the extent and form of a defect or anomaly within the pile in 3D form by utilising and combining, for example, the six 2D profiles obtained from a four-tube layout. The vertical position of such anomalies, together with the significance of relative increases in transit time between adjacent pairs of tubes is colour-weighted to produce a 3D colour-coded graphical representation of the pile and help visualise the potential anomalies.

The nature of these anomalies and their structural significance is, however, still subject to the analysis and interpretation of the test specialist. An example of computer tomography is presented in **Figure 97.30**.

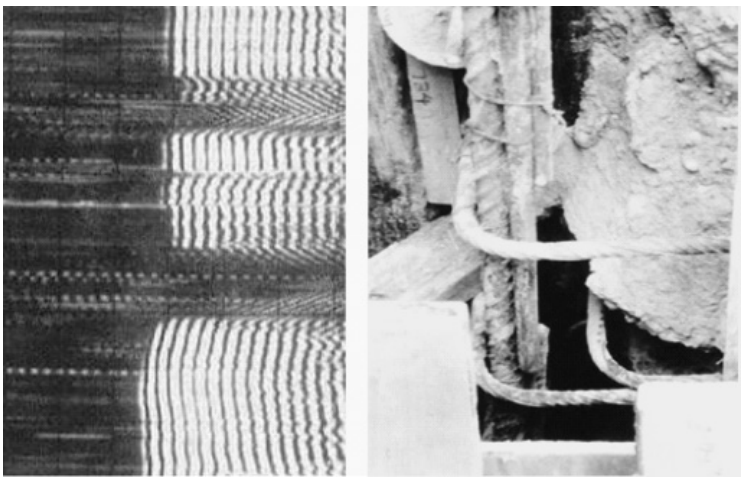


Figure 97.29 Cross-hole sonic logging profile and excavated diaphragm wall
Reproduced from Stain and Willaims (1991), Deep Foundations Institute

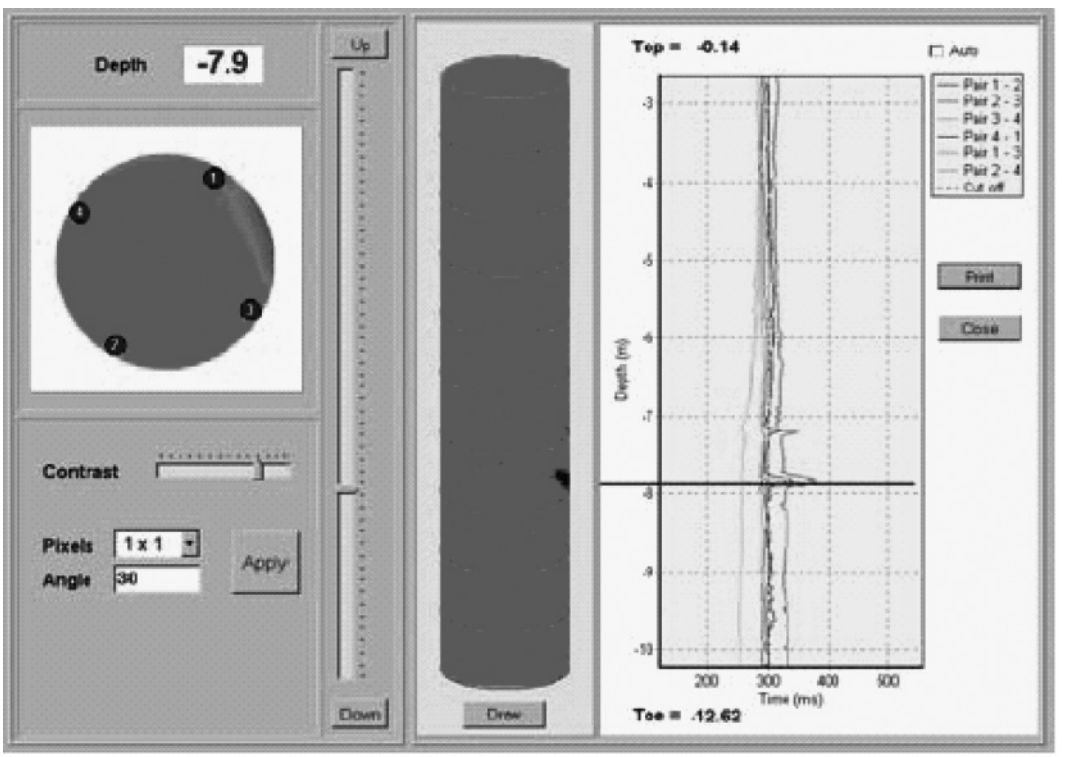


Figure 97.30 Example of computer tomography
Courtesy of Testconsult Limited

97.5.5 The specification of CSL

Similar consideration to those highlighted in section 97.4.8 for low-strain integrity testing would also apply to the specification of CSL testing. Again, the ICE's (2007) SPERW document, and both Turner (1997) and Hertlein and Davis (2006) provide guidance on this issue. The latter document also references the French AFNOR and US ASTM D6760 standards.

In a similar manner to that proposed for low-strain testing, Hertlein and Davis (2006) suggest that the person responsible

for analysis and reporting on the CSL test data should be able to demonstrate a minimum of, say, five years' experience in CSL testing, with at least three CSL projects in each of those years. Similarly they recommend that field personnel responsible for performing or supervising the performance of the CSL test shall be able to demonstrate at least two years' experience with CSL testing, with a minimum of three projects in each of those years

Hertlein and Davis also recommend that the text of the CSL report should include a description of any anomalous zones identified by the test data and a discussion of the apparent difference in pulse velocity. Account should be taken of the sources of possible variation. One particular point that is highlighted is that when assessing the likely significance of an anomaly, any available information concerning test strengths or other actual measurements of concrete properties should be included in the report. These authors highlight that it is not unusual for actual concrete strength to significantly exceed the design strength, in which case the concrete in an anomaly that shows a 15–20% reduction in pulse velocity may still meet or exceed project specifications.

With regard to the review and acceptance/rejection of the piles, a time period (typically three working days) should be specified for the engineer to evaluate the CSL report and determine whether or not the tested piles are acceptable. If the pile is not acceptable, the engineer should decide what additional testing, investigation or analysis is necessary in order to properly characterise the anomaly and determine whether or not the pile needs any repair or remediation. In such an eventuality the repair technique to provide satisfactory remediation must be proposed and agreed between the parties.

97.6 Parallel seismic testing

The parallel seismic test was developed to address the problem of assessing pile integrity once the pile under investigation had been incorporated into the foundation system. This might be as a result of proposed re-use of the foundation or to augment quality control on a construction project after the option for more normal testing has passed.

The test involves installing an access tube adjacent to the pile to a depth exceeding the expected depth of the pile toe level. The tube can either be plastic or steel and, if it is not in intimate contact with the soil, it must be grouted in place to form a good acoustic coupling with the surrounding soils. Prior to the test, the tube is filled with water: again to ensure an acoustic coupling. The arrangements for the test are illustrated in **Figure 97.31.**

The principle of the test is to measure the time taken for a stress wave to travel from the structure to a receiver probe as it is lowered down the access tube. This is achieved by striking the structure adjacent to the pile with a small hand-held hammer and recording the transit time of the resultant stress wave. The test is repeated by lowering the probe incrementally (usually in steps of 500 mm) and recording the transit time for the impact-induced stress wave. A graph of probe depth versus

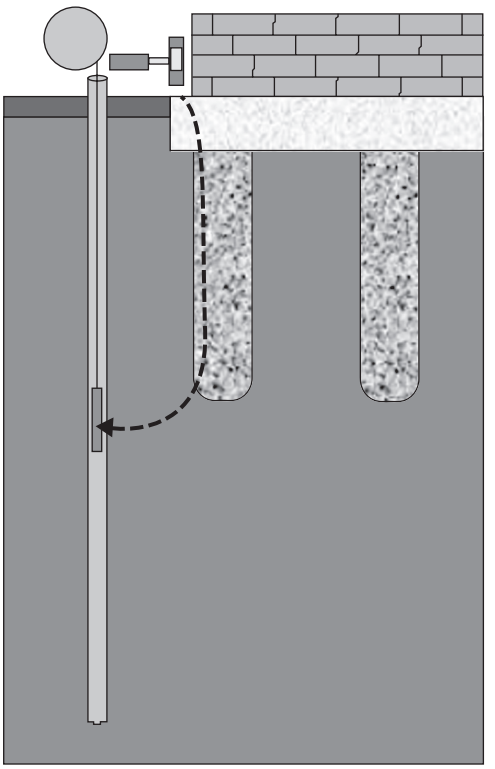


Figure 97.31 Arrangements for parallel seismic test
Courtesy of Testconsult Limited

first arrival time is built up from the data series as shown in **Figure 97.32**.

The test utilises the fact that the propagation velocity of the stress wave through soil is considerably lower than through concrete or steel. As the probe passes the toe of the pile, or a defect such as a discontinuity in the pile shaft, the path length of the stress wave will be increasingly through the soil, and the transit time will correspondingly increase at a greater rate, as indicated in **Figure 97.32**. A change in gradient of the first arrival time at the probe depth should therefore indicate the base of the pile (or, alternatively, of course, it could be indicative of a discontinuity in the pile shaft).

97.6.1 Limitations

A limitation of the test is that a borehole must be drilled alongside the pile to provide a duct for the receiver. This needs to be sensibly parallel to the pile (usually vertical) and typically within one metre of the pile under test. The method is ineffective in dry or loose granular material or unconsolidated fill.

The impact point on the structure for the test hammer must have a good mechanical coupling to the top of the foundation and be as close as possible to the axis of the pile.

97.7 High-strain integrity testing

High strain dynamic load testing is described in Chapter 98 *Pile capacity testing* of this manual. Incidental to predictions of the

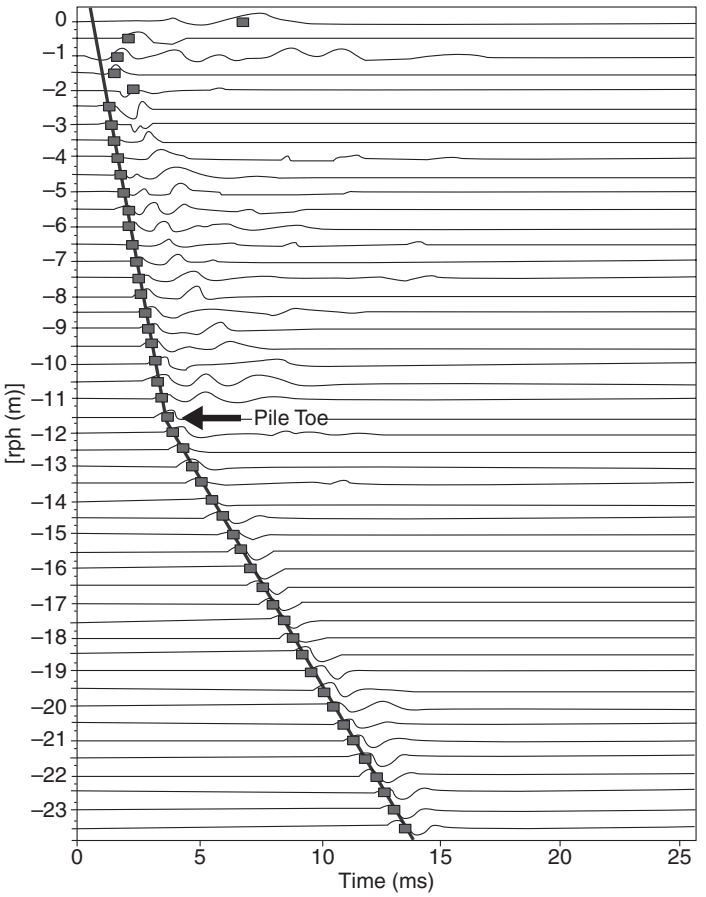


Figure 97.32 Typical parallel seismic test result
Courtesy of Testconsult Limited

load-carrying capacity of the pile is the possibility of detecting significant flaws in the pile which may be as a result of overdriving, or construction defects in the case of a cast *in situ* pile.

The dynamic load test (DLT) involves instrumenting the pile head with strain gauges and accelerometers to measure both upward and downward forces in the pile shaft. The raw data from the instrumentation take the form of total force (F) from the strain gauges (F = strain × cross-sectional area × Young's modulus) and force measured by the processed accelerometer data (F = pile impedance, Z, times pile head velocity, V). If these two sets of data are plotted on the same graph, as illustrated in **Figure 97.33**, any divergence between the two data series will be as a result of changes in pile support from the surrounding ground or the impedance of the pile. In the example shown in **Figure 97.33** the divergence is shown as an increase in force derived from strain measurements (F_t) and a decrease in force based upon velocity measurements (Z.v). This has been generated as a result of shaft skin friction.

If the pile shaft has been damaged due to overdriving or constructed with a shaft defect then this will present itself as

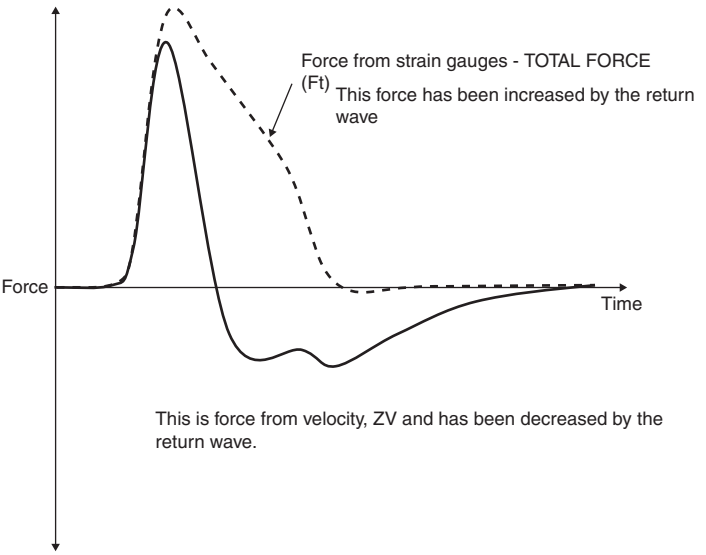


Figure 97.33 Force diagram from dynamic load test

a decrease in the F_t force and a corresponding increase in Z.v force at the point on the timescale representing the depth to the anomaly.

97.7.1 Features detected using high-strain dynamic load testing

The detection of pile defects using the high-strain DLT should be considered a by-product of the load test and should not be relied upon as the principal method for confirming pile integrity. The reason for this is that the duration of the hammer impact overlaps any return signal from defects close to the pile head, effectively masking them. In addition, if a defect is limited to a partial reduction in section as may be the case with a bored cast *in situ* pile, the stress wave will pass the anomaly and continue towards the pile toe. The partial returning signal under these circumstances may be mistakenly identified as a change in soil characteristics at the indicated depth.

This method is, however, useful in identifying damage to pre-cast driven piles which may be due to overdriving. A crack or fracture along the shaft and in particular close to the pile toe will exhibit a recognisable change in both total force and Z.v data. CIRIA 144 (1997) provides further information on interpretation and limitations of this method.

97.8 The reliability of pile integrity testing

97.8.1 Introduction

The reliability and even 'trustworthiness' of integrity testing as a quality control (QC) or quality assurance (QA) tool is a question that constantly arises within any discussion or dispute on the results being found on a particular site. Such a question is linked not only to the ability or reliability of the test method to detect an anomaly or defect within the pile or group of piles, but also to the issue of whether, as discussed in section 97.3.2,

such an 'anomaly' represents a 'defect', and whether such a defect could be

- minor, and not significant to the performance of the pile (a 'flaw' as described by Amir, 2001; see section 97.3.2 above), or
- major, and potentially detrimental to the pile (a true 'defect' according to Amir, 2001).

In the latter case, such a defect could lead to the rejection of a particular pile, or its acceptance at a reduced load or after some agreed secondary repair or investigations.

The following section reviews the implications of the 'reliability' of an integrity test method, and how this is linked to the question of how large a percentage of the total pile population should be tested to provide an acceptable level of assurance of the adequacy of the piled foundations. It also discusses the corresponding implications of providing for a degree of redundancy within the foundation design.

The planning and implementation of a test programme should also to take into account how many unacceptable defects might, on average, and on past experience, be anticipated within the pile population installed at a particular site, and this is also reviewed in this section.

97.8.2 Considerations on the reliability of the test

As highlighted by Cameron and Chapman (2004) the methods of integrity testing are not infallible. An erroneous classification of the quality or characteristics of a pile by an inaccurate integrity test, or an inaccurate interpretation of the test result, can lead to a defective pile not being detected (a false-negative result) and consequently incorporated into the foundation system. Alternatively, it could lead to a sound pile being wrongly condemned (a false-positive result) and subjected to unnecessary inspection or remediation work, or even rejection and replacement.

Hertlein and Davis (2006) identify the following 'links' in the chain of reliability for low-strain integrity testing methods:

(1) adequacy of pile head preparation for the pile head sensor, including trimming the pile to its test level;

(2) correct and recent calibration of load cells and sensors;

(3) suitable data acquisition, signal filtering and processing systems;

(4) trained and experienced site operatives;

(5) the degree of signal damping of the signal response: high length/diameter (l/d) ratios, stiff confining soils, and bulges in the upper portion of the shaft all cause high signal damping;

(6) the presence or absence of multiple anomalies down the pile length (where an anomaly can be a bulge, a neck, cracking, honeycombing or soil/laitance inclusions);

(7) the experience of the testing engineer in test data interpretation (the 'personal' component).

They suggest that these factors can be combined into a 'lumped' reliability model.

It can be appreciated that the breakdown of a single component in this model can throw the reliability of the system into question. Problems with links (1) to (4) can be improved by the test organisation and the site management. Problems associated with links (5) to (7) can minimised by improvements in both the testing hardware and software. Of particular note is the use of equipment capable of improved data analysis, using methods such as the impedance-log analysis described above.

Hertlein and Davis suggest that the present state-of-the-art test methods are very reliable if:

(1) l/d ratios are less than 30:1 in relatively soft soils;

(2) when significant defects are limited to the upper two thirds of the pile length.

Conversely, reliability decreases rapidly with:

(1) high l/d ratios in 'stiff' soils; and

(2) when defects are in the bottom third of the shaft; or

(3) there are multiple defects.

Further insight into these features has been given in the preceding text or can be seen in Hertlein and Davis (2006).

For cross-hole sonic logging the 'chain of reliability' can be identified as:

(1) the quality of the bond between the access tubes and the surrounding concrete;

(2) trained and experienced site operatives;

(3) distance between access tubes;

(4) number of tubes per unit area of shaft cross-section;

(5) distance of perimeter access tubes from the perimeter of the pile;

(6) experience of the testing engineer in the interpretation of test data (personal factor).

Item (2) was considered less important by Hertlein and Davies, but, as noted in CIRIA 144 (1997), cross-hole testing is also reliant upon the field expertise of the site operative: particularly if there is a need to examine an anomaly in greater detail.

97.8.3 The frequency of occurrence of potential flaws or defects within piles.

Cameron and Chapman (2004) reviewed the frequency of defective bored piles identified by low-strain integrity testing, as published in the technical literature. This review noted that within six published papers a range of 1.5–10% of such tests identified piles as being defective, or perhaps more strictly, potentially defective. One of these references, Fleming *et al.* (2009), which covered UK practice, reviewed the results of

time-based 'sonic echo'-type pile integrity testing undertaken in 1981 and 1982. The data suggested that defects were identified in between 1.5% and 1.9% of piles tested (on total numbers of piles tested of between 5000 and 4550 piles per year). Of these, the percentage due to pile construction defects was recorded at 0.4–0.6%. The balance, amounting to 60–80% of the defects detected by the test, was due to damage after the construction of the pile.

A further reference was Ellway (1987), who reported on the results of frequency-based integrity testing in the UK on some 4400 small diameter bored piles in 1985. He suggested that just over 4% of the piles showed signs of 'potentially significant structural faults'. Some one third of these were attributed to post-construction damage from mechanical plant or pile trimming techniques. A further third were due to weak or contaminated concrete within the upper 2 metres of the pile. The remaining third (1.4% of the piles) suggested evidence of soil contamination or loss of section below 2 m.

These two sets of data are now over 25 years old, and it appears to be the general view of the industry that the quality of pile construction has improved greatly since that time: particularly in the case of continuous flight auger (CFA) piles. The improvement in construction quality in the latter technique is largely driven by the now widespread use of instrumentation to monitor and record the pile construction process. The development of these systems also enabled users to better understand and gave a greater insight into the pile construction process.

Taking these factors into account, the available data probably suggest that perhaps 0.5–1.0% of bored cast-in-place piles might be expected to display imperfections within their acoustical response which *might* have some structural significance to the load-carrying capacity or durability of the pile. Although this is a relatively low frequency, the implication is still that, on a site containing a large number of piles, it is to be expected that some piles will be identified as requiring further engineering investigation and evaluation.

97.8.4 How many piles to test?

When electing to use low-strain integrity testing on a particular site it is necessary to also consider what should be the minimum number of tests to be undertaken on the piles. A difficulty is that defects or problems with piles are not always randomly distributed, but might be associated with more difficult ground conditions in a part of the site, or the poor workmanship of a particular rig operative, or some other factor.

CIRIA 144 (1997) suggested that it was the considered view of most practitioners that a statistical approach was often difficult to apply in practice. Hence the decision on whether to test or not on a particular site had to take account of all the circumstances at the site, such as the availability of other construction records or data, the consistency or variability of the ground conditions across the site, and the type of pile and piling systems. As a general recommendation, CIRIA 144 (1997) suggests that, in situations where low-strain integrity testing is considered to be worth doing, all the piles on the site or in a particular area of concern should be tested. However, if the picture emerging from the test results was sufficiently consistent, the designer might consider reducing the number of tests. At the same time, if no meaningful information was being obtained from the tests, the designer might choose to dispense with the testing. This advice was intended to highlight that the designer and the project team should be able to make informed decisions on the applicability and/or usefulness of the testing on a particular site, and to modify the test programme if this was considered appropriate.

Williams and Stain (1987) outlined a testing strategy or 'decision tree', as illustrated in **Figure 97.34**, that also served to give pragmatic guidance on 'when', and 'how many', to test. This suggested, as a guide, that a minimum of around 30 tests would give a useful guide to the condition of the piles on a site.

Cameron and Chapman (2004) have given further consideration to the statistical approaches that may be adopted in selecting the number of piles that should be tested to achieve pre-determined acceptable confidence limits. Such methods require an estimate of whether a percentage of defective piles can be tolerated within the foundation (in the case of a piled raft foundation, for example). Importantly, these authors also included within their review a consideration of the reliability of the testing system.

Clearly, the options available when planning an integrity testing programme are to examine all of the piles or none of them, or a selected sample. Cameron and Chapman (2004) give particular attention to the last option, where conventional sampling theory can allow conclusions to be drawn about the pile population on a particular site, based upon testing a sample of the piles. These authors also highlight the effects of planning the foundation design based upon a degree of redundancy within the overall foundation design. They compare the relative advantages and disadvantages of using either single piles or multiple piles in groups to support individual loads. In the former case, the occurrence of defects is clearly more critical to the structural performance of the foundation (**Figure 97.35**). A similar consideration would apply to piles constructed on strip or grid formations, as illustrated in **Figure 97.36**. Thus, if the interconnecting ground beams are capable of spanning to transfer the reduction in capacity within a structure due to a potentially faulty or unacceptable pile onto the adjacent piles, then such a robust foundation solution may justify a reduced integrity testing regime. At the least it can allow a reasoned view to be taken of the effect of such a predicted shortfall of capacity.

Cameron and Chapman (2004) also point out that such a comparative exercise can often highlight the perceived relative costs and savings of using single piles to support individual high column loads, compared with a notionally more expensive solution using multiple piles, providing a relatively high degree of redundancy. In the former case, the consequential programme

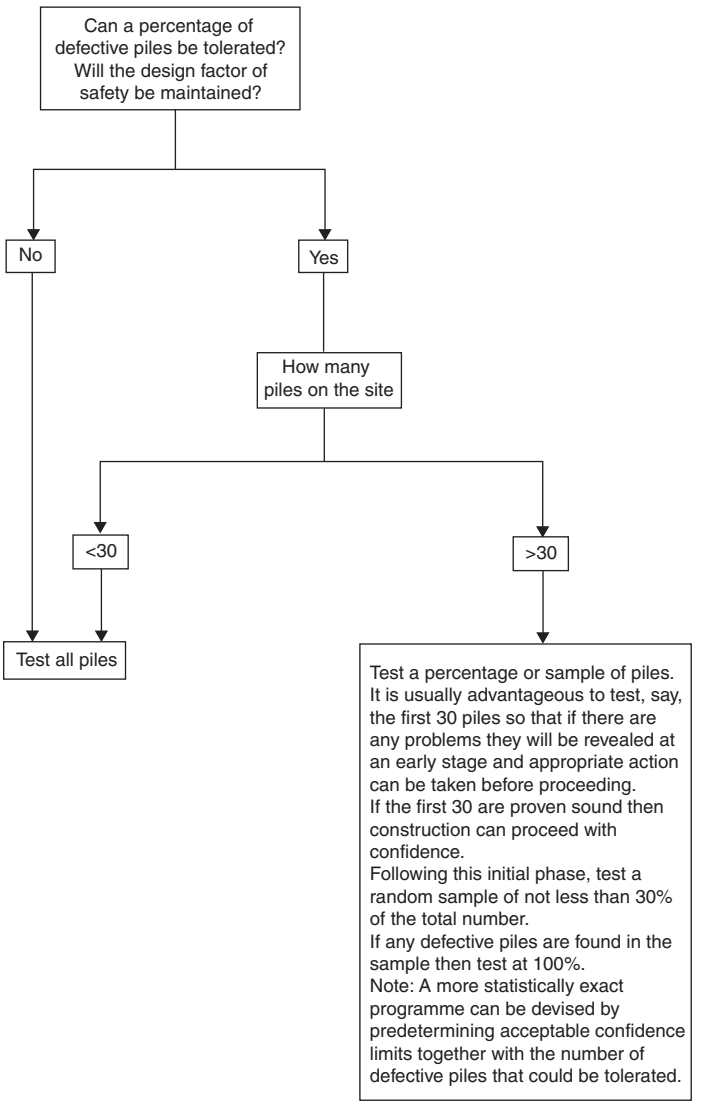


Figure 97.34 Evaluation of numbers of piles to test on a site
Reproduced from Williams and Stain (1987), Engineering Technics Press

risks of the failure or rejection of such a pile, including delays from re-testing or repairing such a pile, have to be considered. It is often not appreciated how, on a major, fast-track project, the overall costs of the piled foundations are often small compared with overall project and programme costs.

As noted earlier, however, if the reliability of the integrity testing system is also factored into the picture, then, even with 100% testing, it is not possible to be 100% certain that all the piles are defect-free.

Cameron and Chapman highlight a statistical approach developed by Preiss and Shapiro (1979) which gave quantitative guidance on the level of testing necessary to achieve a required degree of 'assurance' in the foundations. **Figure 97.37** shows the relationship between sample size, the percentage of defective piles which are acceptable within the pile population on that particular site and the statistical implications of the number of defective piles detected in the test sample.

As an example, consider that the foundation designer requires a confidence level of at least 90% that there will be less than a certain number of 'defective' piles among both the tested and non-tested piles in the population. Consider that 10% of the piles might be allowed to be defective. In such a case, from **Figure 97.37**:

- If 20% of the piles are tested and no defective piles are detected, then the 90% confidence limit is satisfied.
- If, however, one defective pile is identified, then 31% of the piles must be tested, with no further 'defectives'.
- If two defective piles are detected, the sample must be increased to over 40% of the piles, with no further 'defectives'.
- If five defective piles were detected the sample size would again need to increase: to around 70% of the piles.

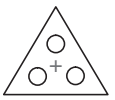
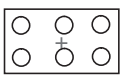
The exercise would clearly continue until either the number of tested piles, the sample size, and the number of defective piles identified within the sample size complied with the required degree of confidence, or all the piles had been tested.

However, Cameron and Chapman also highlight a further complication in the Preiss and Shapiro analysis: it is based upon the test being 100% accurate. If the reliability of the test is less than 100%, then this, of course, affects the confidence level of an accurate prediction. If a system was only 80% reliable, then even testing all the piles would not give an overall confidence level in the test results of higher than 80%.

The practical outcome from a less reliable test is that a greater number of false-positive, false-negative or 'inconclusive' results are likely to be obtained, which might have to be followed up by some other examination or review – such as coring the pile, or excavating alongside it to determine its true condition.

As an example, **Figure 97.38** illustrates the increase in confidence gained for every additional 10% of piles which are integrity tested within a pile population, using a test system which has an 80% 'reliability'. In this case the results are shown for a foundation system that can tolerate between 0% and 4% of defective piles. **Figure 97.38** illustrates that, where the foundation system can tolerate up to 4% of defective piles, the greatest benefit comes from the initial tests on the pile population. However, where the foundation system cannot tolerate any defective piles, then the full benefit only comes when testing the last 20% or so of the piles. Hence, the greater the reliance on each individual pile within the pile population, the more important it is to check the integrity of every single pile.

It should be borne in mind that, although statistically in most practical cases the most important piles are the first 20–40% of those tested, these do not necessarily equate to the first piles installed on the project. The sample must be representative of the whole population, and not skewed in some way: by different ground conditions, different piling operatives or equipment, or other considerations. This has particular relevance where there

	Non-redundant	Working load is equal to failure load Extremely unsafe with potentially very large settlements	Total loss of pile capacity Ultimate failure
	Non-redundant	Deficient pile on the verge of failure Very unsafe	Failure is likely
	Non-redundant	Just sufficient capacity to avoid failure by much tilting	Failure still likely as centre of support offset from centre of load
	Logically non-redundant	Support reduced to two piles providing full and two piles providing half support. Likely to be extra settlement and reduced factor safety	Support reduced to just two piles Factor of safety equals 1.0, therefore stability is marginal
	Clearly redundant	Minimal effect	Support reduced to potentially three or four piles, depending on location of deficient pile. Lowered factor of safety but performance may be adequate
	Clearly redundant	Minimal effect	Minimal effect

Legend: ○ Pile ▭ Pilecap + Position of column load

Figure 97.35 Degree of redundancy offered by different pile cap arrangements for FoS = 2
Reproduced from Cameron and Chapman (2004)

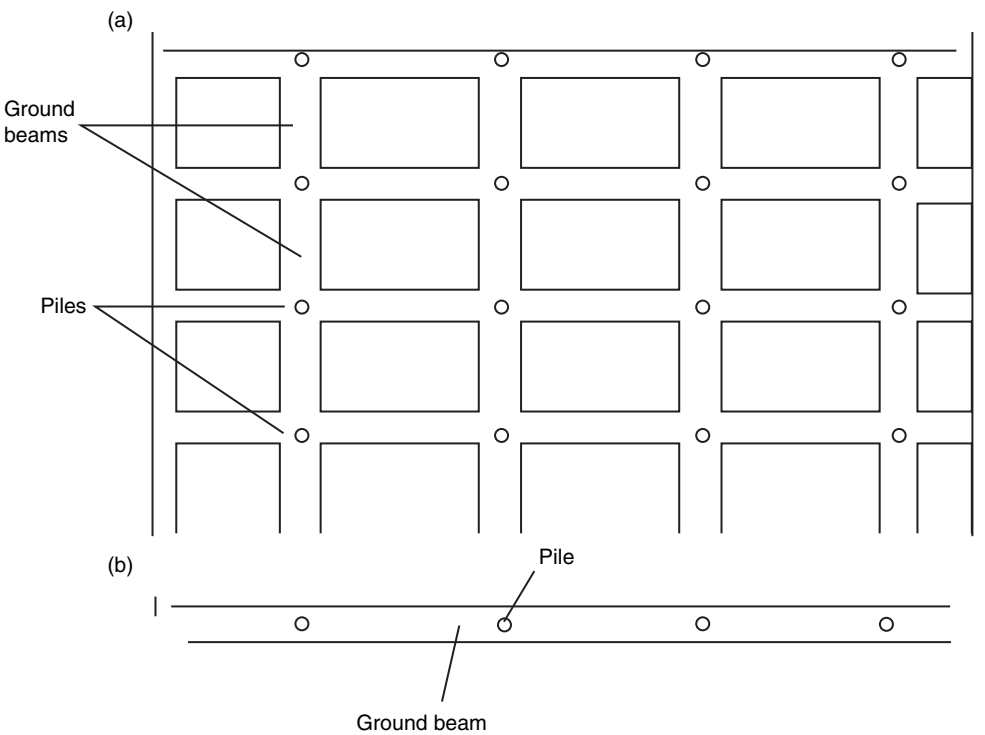


Figure 97.36 Potential redundancy of piled strip (b) and grid (a) foundations

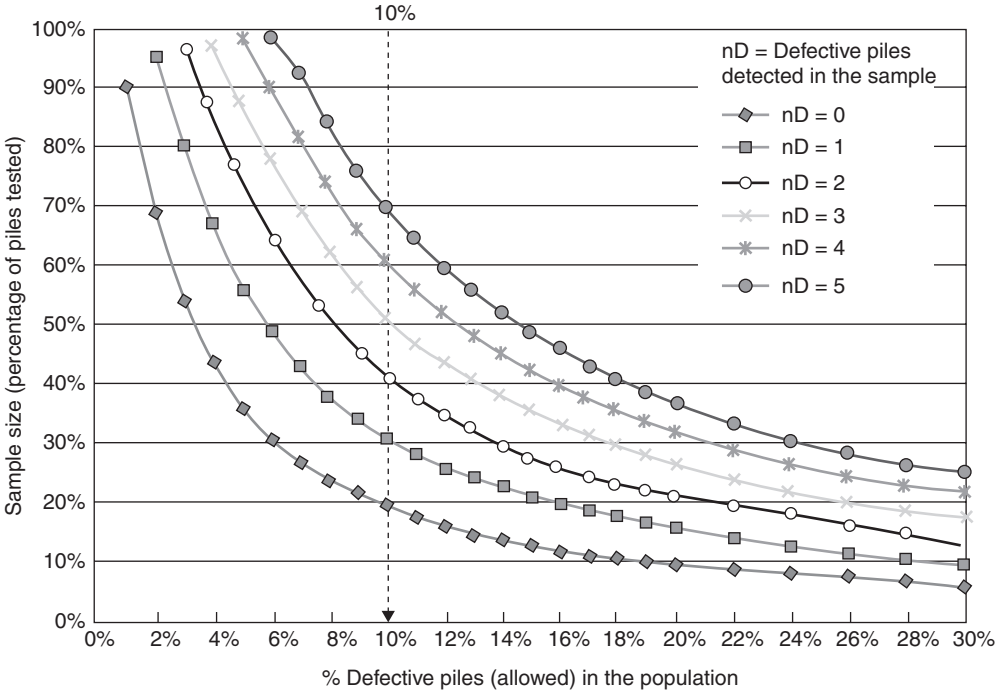


Figure 97.37 Number of piles to be integrity tested, for 90% confidence within a population of piles
Reproduced from Cameron and Chapman (2004)

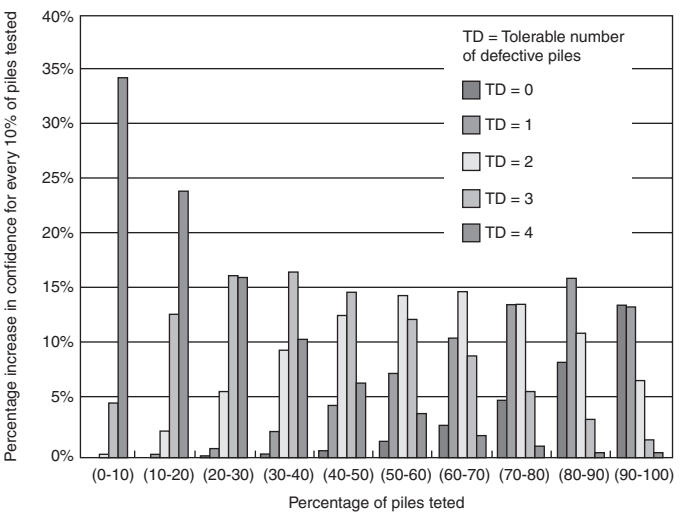


Figure 97.38 Additional increase in confidence by testing each additional 10% of the pile population, using a test system with an 80% reliability
Reproduced from Cameron and Chapman (2004)

is particular pressure in terms of cost and programme to omit some piles from testing.

Cameron and Chapman point out that in some cases the added value of testing the last piles (**Figure 97.38**) could be judged to not be worth the time and consequential cost. **Figure 97.38** also demonstrates that such an omission may be possible only if the tolerable proportion of defective piles is high. Hence there would be programme advantages in selecting a more redundant foundation system.

97.9 Selection of a suitable test method

Table 97.3 summarises the features of the four integrity test techniques outlined in this chapter.

For each technique the table outlines the property measured by the test, details of any pre-planning requirements, and the type of pile suited to the test.

In addition, the table gives a guide to the approximate relative cost of the particular test, its availability and its relative frequency of use. Any other relevant points are noted in a c omments section.

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Test method	Low-strain integrity tests	Cross-hole sonic logging	Parallel seismic tests	High-strain integrity tests
Property measured	Characteristics of the behaviour of acoustic shock waves or stress waves travelling through the pile	Transmission time of an ultrasonic wave through the pile material	Transmission time of acoustic shock waves or stress waves through the pile and intervening soil to a detector	Characteristics of the behaviour of stress waves travelling through the pile from a heavy impact
Pre-planning required?	None	Access ducts have to be cast into preselected piles	Sinking of measurement bore alongside pile	Not strictly necessary, but access for heavy plant may have to be provided
When can test be carried out?	After concrete has achieved design strength (usually 5–7 days min.)	After concrete has achieved design strength (usually 5–7 days min.)	After construction	After concrete has achieved design strength and typically 7 days min. after construction
Type of pile suited to test method	All types	Large diameter cast-in-place typically (usually 600 mm diameter or larger)	Any pile	All types
Approximate relative cost	Low	Low to medium	Medium to very high	Medium to high
Relative frequency of use:				
Control testing	5	4–5	Not applicable	1–2
Retrospective investigation	3–5	Not applicable	0–1	1–2
Availability	Readily available from specialist testing houses	Available from specialist testing houses	From specialist testing houses	Readily available from piling contractor and specialist test house
Test affected by pile length?	Yes, signals increasingly attenuated with depth	No	Yes	No, not within normal pile depth
Comments	Very common technique. Pile response is investigated in terms of time and/or frequency	Mainly used for large diameter cast-in-place piles, piers and barettes. Especially large single piles supporting high column loads. Not usually suitable for retrospective investigation because of necessity to install access ducts	Used for retrospective investigation only	Not commonly used in routine testing. Typically may be used to investigate a post-installation problem, such as pile damage

Key to symbols:
Relative frequency of use: Scale 0 to 5: 0: very rare. 1: rare 2: occasional 3: sometimes 4: common 5: very common
Approximate relative cost: Scale Low–Very high: Low <10% of pile cost. Medium: 10–50% of pile cost. High: 50–100%. Very high: >100% of pile cost
Note: relative cost excludes mobilisation costs.

Table 97.3 Summary of the suitability and applicability of the test methods
Reproduced with permission from CIRIA R144 (1997), www.ciria.org

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Turgat (2004). Research into the correlation between concrete strength and UPV values. NDT.net, **12**(12), December.
Williams, H. T. and Stain, R. T. (1987). Pile integrity testing: Horses for courses. In *Proceedings of the International Conference on Foundation and Tunnels*, 22–26 March. London: Engineering Technics Press, pp. 184–191.

97.10.1 Further reading

Although cited in the reference list above, the following publications are recommended for further reading and background information:

Fleming, K., Weltman, A., Randolph, M. and Elson, K. (2009). *Piling Engineering* (3rd Edition). London: Taylor & Francis.
Hertlein, B. and Davis, A. (2006). *Nondestructive Testing of Deep Foundations*. New York: John Wiley.
Institution of Civil Engineers (2007). *ICE Specification for Piling and Embedded Retaining Walls* (2nd Edition). London: Thomas Telford.
Turner, M. J. (1997) *Integrity Testing in Piling Practice*. CIRIA Report 144. London: Construction Industry Research and Information Association.

97.10.2 Useful websites

Corporate market research procurement services; for example, www.profound.com
Testing, monitoring, analysis and consulting services for deep foundations: Pile Dynamics, Inc. and GRL Engineers, Inc.; www.pile.com

It is recommended this chapter is read in conjunction with

- Chapter 82 *Piling problems*
- Chapter 93 *Quality assurance*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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Chapter 98

Pile capacity testing

Michael Brown University of Dundee, UK

As there is still uncertainty in accurately predicting the performance of piled foundations based upon design calculations there remains a need to test piles. Several methods of pile testing are available some of which have been in common usage for many years, while others are relatively recent developments. Static pile testing is a well understood and simple test technique that has been in use for many years. This approach has the benefit of directly producing test results but is hindered by the increasing size of associated testing infrastructure as pile capacity increases. A recent variation of classic top-down static testing is bi-directional testing which relies on the incorporation of a specialised loading jack or jacks in the pile shaft at some depth below ground surface. This technique has the ability to apply test loads that greatly exceed those possible in other pile test types. Alternative pile testing techniques come in the form of rapid and dynamic load tests that have the benefit of quick testing and reduced testing infrastructure but require more complicated analysis and interpretation techniques due to the rapid/dynamic nature of the tests.

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98.1 An introduction to pile testing

Although pile design has advanced in recent decades, the determination of axial pile capacity is still dependent on the use of empirical correlations. As a result, it is only possible to estimate capacity to ±30% in many soil types (Randolph, 2003). Owing to the uncertainty associated with predicting load-settlement behaviour using existing design methods, it is common practice to carry out pile load tests for verification.

The information obtained from pile load testing may be used in a number of ways (Poulos, 2000) including:

- construction and quality verification;
- verification of design information;
- to allow for a more refined or confident design with potential cost savings for subsequent piling works.

As pile testing may prove expensive (especially on small contracts), the need for pile testing may be considered in terms of risk reduction. On larger sites, where significant numbers of piles are being installed, pile testing may be considered from a perspective of potential cost saving. Through testing, improved design parameters may be determined resulting in optimisation of the piles through a reduction in length, for example. The pile testing strategy recommended by the *ICE Specification for Piles and Embedded Retaining Walls* (2007) (often referred to as SPERW) is shown in **Table 98.1**.

Eurocode 7 allows pile design based upon a pile load test(s) and states that pile tests should be undertaken in the high- to medium-risk situations highlighted in **Table 98.1**. Eurocode 7 also allows the characteristic resistance of axially loaded piles to be derived directly from pile load tests with the level of correlation increasing with increasing number of piles tests (from one to five). For five pile tests or more direct correlation is allowed with the measured test results which in turn are reduced by a partial factor to obtain the design resistance (see British Standards Institution, 2004, EC7, for further detail). This prescriptive approach to factoring may be considered an improvement over the *ad hoc* manner in which pile load tests have previously been used to influence safety factors. Further information, with respect to pile testing as part of the Eurocode framework, is available in BS EN1536, *Special Geotechnical Works: Bored Piles* and in the future through EN/ISO 22477-1, *Geotechnical Investigation and Testing: Testing of Geotechnical Structures – Part 1: Pile Load Test by Static Axially Loaded Compression* (currently in draft form). Piles may be tested at different stages throughout a project depending on how the outcome of the test will be used. For instance, preliminary or trial piles are tested prior to the main works or pre-contract and are designed to validate design and achievable performance along with checking construction techniques in certain ground types. It is important that preliminary piles are tested sufficiently in advance of the main works such that findings from the tests can be incorporated in working pile designs. Piles tested that will form part of the final structure are referred to as working piles.

As well as the stage of the project at which piles should be tested, it is also important to consider the time after installation at which piles should be tested. For instance, driven or displacement piles installed in clay cause significant disturbance of the ground. This results in changes to the local effective stress regime which with time equalise depending on the soil's permeability and the size of the pile. For instance, SPERW (2007) suggests a minimum of four days between installation and testing of any pile type, although it does go on to say that a 12 hour delay between installation and testing is adequate for driven piles installed in coarse-grained soils. In contrast, Fleming *et al*. (2009) suggest 1 to 3 weeks between installation and testing depending on soil type and experience.

Pile load testing methods include static tests, dynamic tests and kinematic or rapid load tests. There are also alternative static load test methods such as bi-directional testing in the form of the Osterberg Cell (O-Cell). Generally, static pile testing methods are expensive and time consuming (Fleming *et al*., 2009) becoming more so with increased load requirements, but have the advantage of simple analysis and interpretation. Conversely, dynamic and rapid load testing methods are quick to carry out and become cheaper than static tests with increased load requirements but require more specialised equipment and analysis. Comparisons of some of the typical pile test characterisitics are shown in **Table 98.2**. Note that this table is based upon current typical approaches to testing and does not dictate use of the techniques; for instance, dynamic tests may be undertaken on cast *in situ* piles but this may not be a frequent occurrence.

98.2 Static pile testing

98.2.1 What is static pile testing and what types of test are there?

The most common method of pile load testing, static testing, may take one of two forms. These are the maintained load test (MLT) and the constant rate of penetration test (CRP). The names of the two tests are derived from their methodology. Since the invention of tests such as the O-Cell method, these types of testing are often referred

Characteristics of the piling works	Risk level	Pile testing strategy
Complex or unknown ground conditions	High	Both preliminary and working pile tests essential
No previous pile test data		1 preliminary pile test per 250 piles
New piling technique or very limited relevant experience		1 working pile test per 100 piles
Consistent ground conditions	Medium	Pile tests essential
No previous pile test data		Either preliminary and/or working pile tests can be used
Limited experience of piling in similar ground		1 preliminary pile test per 500 piles
		1 working pile test per 100 piles
Previous pile test data available	Low	Pile tests not essential
Extensive experience of piling in similar ground		If using pile tests either preliminary and/or working pile tests can be used
		1 preliminary pile test per 500 piles
		1 working pile test per 100 piles

Table 98.1 Typical pile testing strategy based upon risk levels
Reproduced from Institution of Civil Engineers (2007) (SPERW, 2007)

	Type of pile test		
Characteristic	**Static[1]**	**Rapid**	**Dynamic**
Load duration	1–24 hours	100 milliseconds	7 milliseconds
Tests per day	1	2–6	8
Reaction mass required (as a percentage of pile capacity)	120%	5–10%	1.5–2%
Time needed for results	Directly	10 minutes	4 hours
Pile types tested			
precast	yes	yes	yes
cast *in situ*	yes	yes	yes
tubular steel	yes	yes	yes
Perceived reliability	high	experience too limited to assess	intermediate
Cost[2] (Pounds per kN of pile capacity)	0.42–0.75	0.45	0.05–0.1
Cost per pile	850–1500	400	50–200

(1) Static definition here does not include O-Cell (top-down).
(2) Based upon testing at 1000–2000 kN capacity piles on the same site, note dynamic test costs assume a pile driving rig is present on site. Costs given in GB pounds.

Table 98.2 Comparison of typical features of pile testing techniques
Data taken from Hoelscher and van Toll (2009)

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to as top-down as the loading is applied at the head of the pile.

98.2.2 Maintained load compression test (ML or MLT)

This type of test is often referred to using the acronyms ML or MLT. ML testing works by applying and maintaining increments of load to the head of the pile for a minimum specified time (**Table 98.3**) and until a specified rate of settlement criterion is satisfied (**Table 98.4**), at which point the load is either increased or reduced (Tomlinson and Woodward, 2008). The minimum time for holding of an increment typically varies from 30 minutes to 6 hours, with unloading increments typically held for 10 minutes. An example of the pile response seen during an ML test is shown in **Figure 98.1**.

Generally, this test method is not used to prove the pile ultimate load capacity or generate 'plunge' as it is difficult to maintain constant load at high settlement rates. Additionally, it is typical to increase the pile load in 25% increments (**Table 98.3**) of the working design verification load (DVL, see definition in **Table 98.3** and specification section later), which may mean the application of many hundreds of kN's between load increments. It is then possible for the actual ultimate load to be missed resulting in an underestimation of ultimate capacity as the pile plunges at a higher load increment. To avoid the underestimation, the load increment may be reduced to 10% DVL throughout the test or when load stability issues occur as is recommended for testing of preliminary piles (SPERW, 2007).

98.2.3 Constant rate of penetration testing (CRP)

The CRP test varies from the ML test in that a varying load is applied to the pile to maintain a constant rate of penetration. The rate of penetration is typically chosen to reflect the main soil type that the pile installation encounters (**Table 98.5**). Due to these penetration rates, tests are completed relatively quickly. For instance, a 600 mm diameter pile installed in clay pile can be taken to a penetration equal to 15% of the pile diameter (90 mm) in 2.5 hours. It can be seen in **Table 98.5** that the rates used for CRP in US practice may be 50% slower or faster than those specified for UK use. It should also be noted when comparing tests or correlations from the US that there is also a different ML test referred to as the quick load test method (QLT) where load increments are only held for 2.5 minutes (ASTM D1143–81:1994).

The CRP test is not considered appropriate for most testing situations and is typically reserved for research purposes. Although testing is faster, it may require greater capacity from

Load*	Minimum time of holding load for a single-cycle pile test	Minimum time of holding for a multi-cyclic pile test
25% DVL	30 minutes	30 minutes
50% DVL	30 minutes	30 minutes
75% DVL	30 minutes	30 minutes
100% DVL	6 hours	6 hours
75% DVL	n/a	10 minutes
50% DVL	n/a	10 minutes
25% DVL	n/a	10 minutes
0% DVL	n/a	1 hour
100% DVL	n/a	1 hour
100% DVL + 25% SWL	1 hour	1 hour
100% DVL + 50% SWL	6 hours	6 hours
100% DVL + 25% SWL	10 minutes	10 minutes
100% DVL	10 minutes	10 minutes
75% DVL	10 minutes	10 minutes
50% DVL	10 minutes	10 minutes
25% DVL	10 minutes	10 minutes
0% DVL	1 hour	1 hour

*SWL denotes Specified Working Load; DVL denotes Design Verification Load

Table 98.3 Minimum loading times for a maintained load compression proof test
Reproduced from Institution of Civil Engineers (2007) (SPERW, 2007)

Pile head displacement range	rate of settlement criteria
< 10 mm	≤0.1 mm/hour
between 10 and 24 mm	≤1% × pile head displacement/hour
> 24 mm	≤0.24 mm/hour

Table 98.4 Rate of settlement to be used along with minimum hold times
Data taken from Institution of Civil Engineers (2007) (SPERW, 2007)

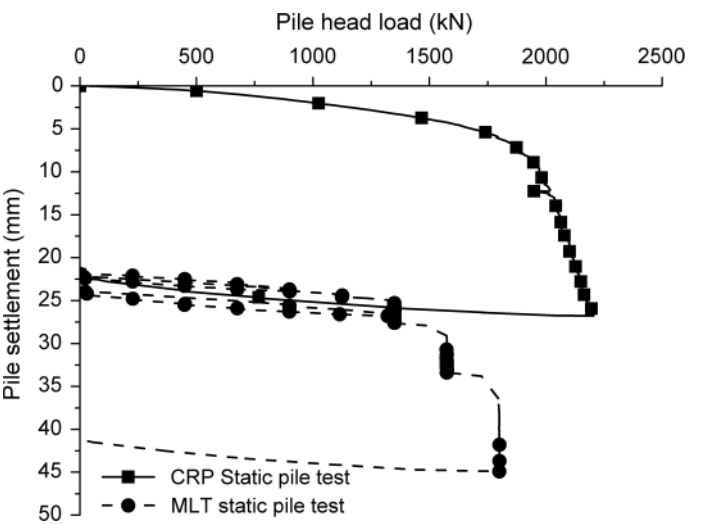


Figure 98.1 Example of CRP and MLT static load tests on a 12 m long, 600 mm diameter cast *in situ* bored pile in glacial till (see Brown *et al.*, 2006 for more information on the pile installation and testing)
Data taken from Brown *et al.* (2006)

the loading and reaction systems to produce plunge. There are also reservations about the relatively high penetration rates and short test duration (rate effects) especially where piles are installed in clay. It has been shown that as the penetration rates in CRP increase so does the ultimate pile capacity and stiffness (**Figure 98.1**). In terms of representing meaningful structural loading rates, it may be argued that the loading rate should be selected based upon the permeability of the soil specific to the pile installation and effective drainage path length. It should also reflect whether the desired behaviour to be proven is drained or undrained.

98.2.4 How it works and the various test set-ups

Static pile testing systems typically require a structure to react against to allow the application of load to the pile. A typical reaction or anchor pile type arrangement for pile testing is shown in **Figure 98.2**. Alternatively, the reaction to the hydraulic jack can be provided by placing kentledge (reaction mass) above the jacking arrangement (**Figure 98.3**). If the pile is a preliminary pile, i.e. one that is required to validate construction performance prior to construction of the working piles, then either arrangement of pile test would be suitable. Where load tests are required on piles that will form part of the final structure (working piles) then the test arrangement using kentledge would appear more appropriate. Unfortunately, if several working pile tests were needed, multiple individual test

	Rate of penetration mm/s (mm/min)			
Major soil type	SPERW 2007	BS8004	ASTM D1143–81	EN1536–2000
Fine-grained soils (e.g. clay)	0.01 (0.6)	0.0125 (0.75)	0.0042–0.021 (0.25–1.25)	0.01667 (1)
Coarse-grained soils (e.g. sand or gravel)	0.02 (1.2)	0.025 (1.5)	0.0125–0.042 (0.75–2.5)	0.01667 (1)

Table 98.5 Examples of different pile penetration rates specified for CRP testing
Note BS8004 has now been superseded



Figure 98.3 Top-down static pile testing arrangement utilising water-filled tanks as a reaction system. The testing arrangement is completely obscured by the tanks
Photograph courtesy of Deltares; all rights reserved

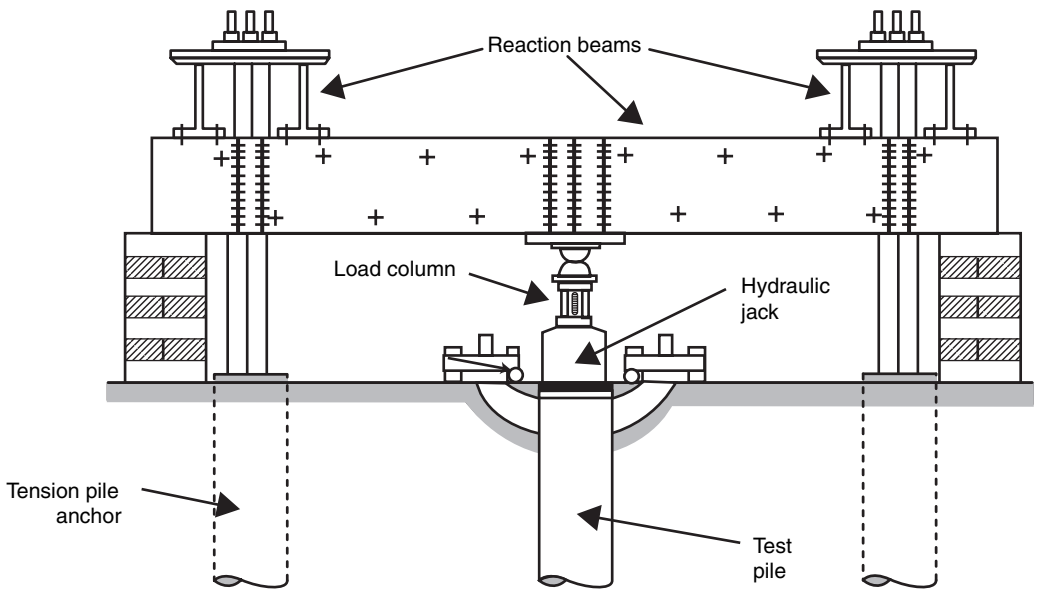


Figure 98.2 Top-down static pile testing arrangement utilising a tension pile reaction system
Modified with permission from CIRIA PG7 Weltman (1980), www.ciria.org

arrangements would be required to avoid time delays. Greater detail regarding pile testing procedure and equipment arrangements is given by Weltman (1980).

Application of the load to the test pile is usually undertaken using a large hydraulic jack with manual or automated control of hydraulic oil flow to allow either a maintained load to be applied or varying load for constant penetration rates (**Figure 98.4**). Historically load has been measured with a manually read calibrated load column placed between the jack and the reaction system (**Figure 98.4**). It is now more likely that an automated strain gauge-based load cell will be used which produces readings that can automatically be logged by a laptop PC. Similarly the piles' deflection has been measured using manually read dial gauges (mounted on a reference beam placed three diameters or 2 m from the test pile) whereas these have been replaced by displacement transducers that can be logged by a laptop (Fleming *et al*., 2009). Measurement by displacement transducers or dial gauges should also be verified by optical levelling to an accuracy of 0.2 mm. Developments in monitoring equipment along with reliable and cost-effective servo hydraulic control systems (which allow true stress and displacement controlled tests to be undertaken) have led to the development of fully automated test rigs that can be operated remotely (England, 2002). Automating such a system means that staff spend less time on an individual site and less time travelling between sites.

Another advantage of automation is that site operatives do not have to spend long periods close to highly stressed structures which in extreme cases have failed causing injury. There is also less need for shift working which may result in operatives working on deserted sites during the hours of darkness.

98.2.5 Static tension tests

Tension or uplift testing of piles utilises similar equipment and testing procedures to compression testing (**Figure 98.5**) although two jacks may be used to avoid inducing bending stresses in the pile. Connections to cast *in situ* piles are similar to those employed in tensile reaction piles whereas preformed steel piles may require additional welded brackets (Tomlinson and Woodward, 2008; Fleming *et al*., 2009). It is normal to test more than one pile that may be subject to tension and at least 2% of piles where a large number are subject to tension.

98.2.6 Static lateral load testing

As well as tensile load, piles are often subject to combined lateral, vertical and moment loading which in certain circumstances, such as foundations for offshore structures, may be cyclic in nature. It is not easy to achieve such complicated load paths in full-scale pile tests but both monotonic and cyclic lateral tests may be specified. Typically an adjacent pile(s) is used as reaction with a jack placed in between the reaction pile(s) and the test pile (**Figure 98.6**). Where cyclic loading is required automated jacks and monitoring systems are recommended. Similar load/deflection measuring techniques may be used as per axial pile load tests but it is also advisable to



Figure 98.4 Pile head loading arrangement showing loading jack placed on the pile with calibrated load column above with spacers to the reaction frame. Note the dial gauges for settlement measurement in contact with the head of the pile



Figure 98.5 Tension pile test. Note loading jack above the reaction beam

Photograph courtesy of Cementation Foundations Skanska Ltd; all rights reserved



Figure 98.6 Lateral load pile tests showing piles being jacked off each other. Displacement transducers (mounted on the white reference frame) to monitor lateral deflections can be seen against the test pile on the right hand side of the image

Photograph courtesy of Cementation Foundations Skanska Ltd; all rights reserved

measure relative movement of the test pile to the reaction piles and pile head rotations. In addition, valuable information for test analysis can be obtained by instrumenting the pile (Reese and van Impe, 2011) to allow determination of moments with depth. When selecting the number of lateral piles to be tested, close attention should be given to the variability of the ground over the top few metres of the test pile as this zone has significant effect on the lateral pile behaviour.

98.2.7 Advantages and disadvantages of static testing

The advantages of static testing lie in the simplicity of the test and it having been a long accepted technique. Generally, interpretation is simple and results can be produced quickly without the need for specialist interpretation or in some cases with only limited knowledge of the ground conditions.

Static tests are relatively slow. For instance, a typical ML test generally takes a minimum of 19 hours but may take much longer depending on the particular test specification. This also neglects the time required for setting up the test equipment. This has programme issues and implications regarding site safety with the need in some cases for 24-hour working.

The main disadvantages of static testing stem from the need to have enough reaction at ground level to apply loading. This results in infrastructure-intensive reaction frames connected to tension piles or kentledge (**Figures 98.2** and **98.3**). Such systems are time consuming to construct, expensive and may have significant space requirements both in terms of footprint and materials handling. The need for such large reaction (120% of anticipated ultimate capacity, **Table 98.2**) also means that the reaction system is highly stressed during testing, resulting in potential safety concerns.

One of the possible drawbacks of the anchor pile or kentledge-based reaction systems is that they interfere with the behaviour of the pile under test. Poulos (2000) suggests that the use of kentledge may cause an increase in test pile capacity and stiffness. Predictions in sand suggest that pile capacity and stiffness might be increased by 10–20% due to the presence of kentledge (Poulos and Davis, 1980). With this in mind, SPERW suggests that there should be a minimum separation of 1.3 m between the kentledge supporting arrangement and the test pile, BS EN1536 suggests a minimum of three times the test pile diameter. For a reaction pile arrangement where the reaction piles are shorter than the test pile, there should be a minimum separation of 2–3 m (depending on guidance consulted) or three times the test pile diameter. If the reaction piles are longer than (or inclined) the test pile the spacing should be increased to five times the diameter of the largest pile (test or reaction) unless the base capacity of the test pile is 20% less than that of the reaction piles. In the case of tension tests, any ground beams or adjacent compression piles used to support the reaction system should be at least three times the test pile diameter away from the test pile.

98.2.8 Interpretation of static tests

Inspection of the load-settlement curves determined from pile testing can be used in several ways. Firstly, by inspection the shapes of the curves may hint at the adequacy or integrity of the installation and highlight problems occurring during pile formation. Additionally, the result may be used to check performance criteria.

98.2.8.1 Settlement criteria

Pile design is typically based upon determining both ultimate and serviceability limit cases. When assessing performance from pile tests, it seems that settlement criteria at working loads is given more attention (in an onshore setting) than ultimate pile capacity. This is probably a result of ultimate capacity being associated with settlements that would cause serviceability damage to structures.

For working load tests, SPERW (2007) suggests that settlement criteria should be in the range of 5–10 mm at the design verification load (DVL) and 15–25 mm DVL + 0.5 SWL, although this should be assessed based on the specific building type.

98.2.8.2 Determination of ultimate capacity and test termination

Another potential barrier to using ultimate pile capacity to define performance seems to lie in the various definitions of what constitutes the ultimate pile load or capacity. For example, Tomlinsons and Woodward (2008) list seven different recognised techniques. These may be based upon the load at a certain settlement defined by the pile diameter (for instance, 10–15% of the test pile diameter) or a feature related to the shape of the

load-settlement curve such as the load at which settlement continues to increase without further increase in load. Definitions such as the latter may be problematic; for instance, in granular soils it is common to see gradual increases in capacity with increasing settlement, i.e. the criterion is never met.

EC7 seems relatively vague on this point but does state that where there is difficulty in determining the ultimate capacity a value equal to 10% of the diameter should be assumed. SPERW (2007) avoids the use of a specific settlement criterion and defines the ultimate capacity in MLT as the maximum load that can be applied whilst achieving the specific settlement rate criteria (**Table 98.4**). Conversely, though, CRP tests in SPERW (2007) are terminated on the basis where loads are reducing for at least 10 mm or a settlement equal to 15% of diameter is achieved.

If a preliminary pile does not reach ultimate capacity then results may need to be extrapolated to give an indication of the expected capacity. Methods proposed by Chin (1972) and Fleming (1992) referred to as hyperbolic methods are often used for this with varying degrees of performance for particular pile and soil types. The Chin method is based on the assumption that measured pile load-settlement behaviour can be represented by a hyperbolic function such that the ultimate capacity (U) of a rigid pile can be represented by

$$U = \frac{\Delta}{\left(\Delta/P\right) - K} \tag{98.1}$$

where Δ represents the pile head settlement at any pile head load (P), and K is the intercept on the horizontal axis (Fleming, 1992). Chin observed that if Δ/P was plotted against settlement (Δ) a straight line was obtained (**Figure 98.7**). The gradient of the straight line is the ultimate capacity of the pile (Equation 98.1). **Figure 98.7** also shows the CRP test previously presented in **Figure 98.1** with the ultimate pile capacity predicted from the Chin analysis shown as an asymptote at 2323 kN.

The form of the hyperbolic function was then improved upon by Fleming (1992) through incorporating parameters from back analysis of a large pile testing database and the effects of pile shortening. Fleming (1992) also separates consideration of pile shaft (denoted subscript s) and base (denoted subscript b) components such that:

$$\Delta_s = \frac{M_s D_s P_s}{U_s - P_s} \tag{98.2}$$

where the pile diameter (D_s) has been incorporated to recognise its influence on load settlement behaviour and M_s is a dimensionless flexibility factor with low variance with typical values between 0.001 and 0.0015 (Azizi, 2000). Pile base settlement is based upon the assumptions made for linear-elastic settlement of a footing and assessment of soil secant modulus (E_b) from a real load/settlement relationship under a footing at one quarter of the ultimate stress (U_b/4).

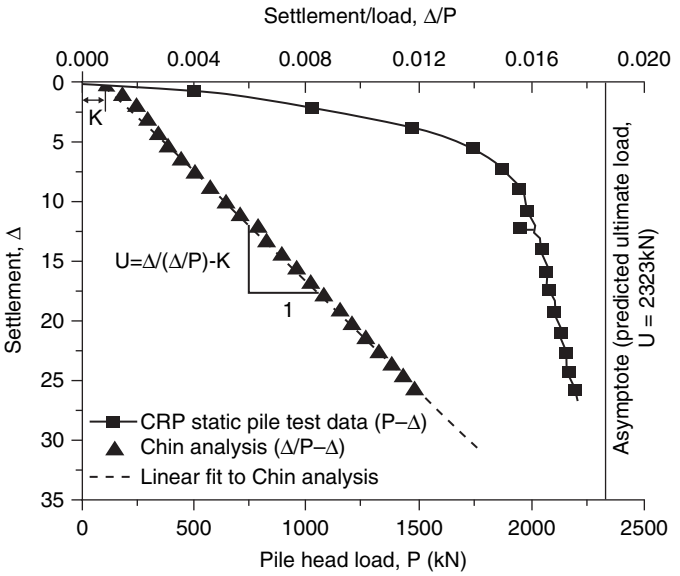


Figure 98.7 Chin analysis of a CRP static load tests on a 12 m long, 600 mm diameter cast *in situ* bored pile in glacial till

$$\Delta_b = \frac{0.6 U_b P_b}{D_b E_b \left(U_b - P_b\right)} \,. \tag{98.3}$$

Note that equations 98.2 and 98.3 above are for consideration of a rigid pile only. See Fleming (1992) for modifications that allow shortening to be incorporated. A useful summary of typical soil secant modulus values (E_b) can be found in Azizi (2000).

More detail of these techniques and interpretation of the results of static testing can be found in Fleming *et al*. (2009) and Tomlinson and Woodward (2008).

One of the aims of pile testing may be to verify or update design parameters and techniques. Where this requires specific knowledge of separate shaft and tip resistance and settlement it is recommended that the test pile is appropriately instrumented rather than attempting to derive such information based upon pile head measurements alone.

98.2.9 Specifying a static pile test

The magnitudes of the applied load increments for an ML test (**Table 98.3**) are chosen to verify the ability of the pile to carry the design loads associated with the structure and the resulting settlement. Typically, up to 10 mm of settlement is considered acceptable at the design verification load for insensitive buildings. The design verification load (DVL) referred to in SPERW is typically 40–50% of the anticipated or previously measured ultimate bearing capacity. The load a pile will actually carry in service is referred to as the specified working load (SWL). The DVL should account for the SWL and any specific differences that may occur between the pile load test and the pile in service, for example pile downdrag or variations in final levels

of the site. Working piles are typically loaded to the DVL plus 50% of their SWL (proof load test). Preliminary piles may be further subject to increments of 25% of their SWL. When calculating the anticipated static capacity it should be remembered that the calculated capacity should be as realistic as possible and avoid the adoption of unnecessarily conservative soil parameters.

98.3 Bi-directional pile testing

98.3.1 What is bi-directional load testing?

The bi-directional static load test is an alternative to the top-down static load tests described above (England, 2008). The method varies from the top-down load tests in that the major component of the system is a sacrificial, purpose-built high capacity jack (or jacks) cast in the pile length. The most common form of this type of system is referred to as the O-Cell, which is receiving increased use and acceptance.

The bi-directional O-Cell method of testing was originally developed by Professor Jorj Osterberg to load the pile from the pile base rather than from the head and more specifically for the evaluation of skin friction and end-bearing in rock sockets. On pressurising the jack(s), reaction is provided by the pile end-bearing capacity to mobilise the pile's skin resistance and *vice versa* until the capacity of either the jack or the upper or lower components of resistance are exceeded. More recently, individual jacks or multiple jacks have been installed at various levels within cast *in situ* piles and diaphragm walls (barrettes) to allow testing of different sections of the foundation length (Randolph, 2003; England, 2008; England and Cheeseman, 2010).

98.3.2 How does bi-directional testing work?

The system typically works by incorporating a purpose-built, low friction jacking device within a cast *in situ* pile (**Figure 98.8**). The loading arrangement is attached to the reinforcing steel or other support structure to ensure precise location and depth (**Figure 98.9**). When pile construction is complete and the concrete has attained sufficient strength, the jack is then pressurised using a hydraulic pump with the water pressure monitored by a pressure transducer attached to the hydraulic return line. The foundation is then effectively separated into two elements which are subjected to simultaneous loading.

During testing, the separation of the jack is monitored by displacement transducers (LVDTs or LVWDTs) mounted between the two faces of the jack. Telltales are also attached to the top of the jack that extend up to the head of the pile, which allow the compression of the pile shaft to be monitored and the location of the top and bottom of the jack(s) to be determined. The movement of the pile head is also monitored along with compression of the pile. On completion of testing, the jack(s) and the annulus around the jack(s) can be grouted up through the hydraulic circuit to allow the pile to be integrated into the structure as a working pile. The Osterberg Cells come in various diameters from 230 mm to 870 mm, with capacities from

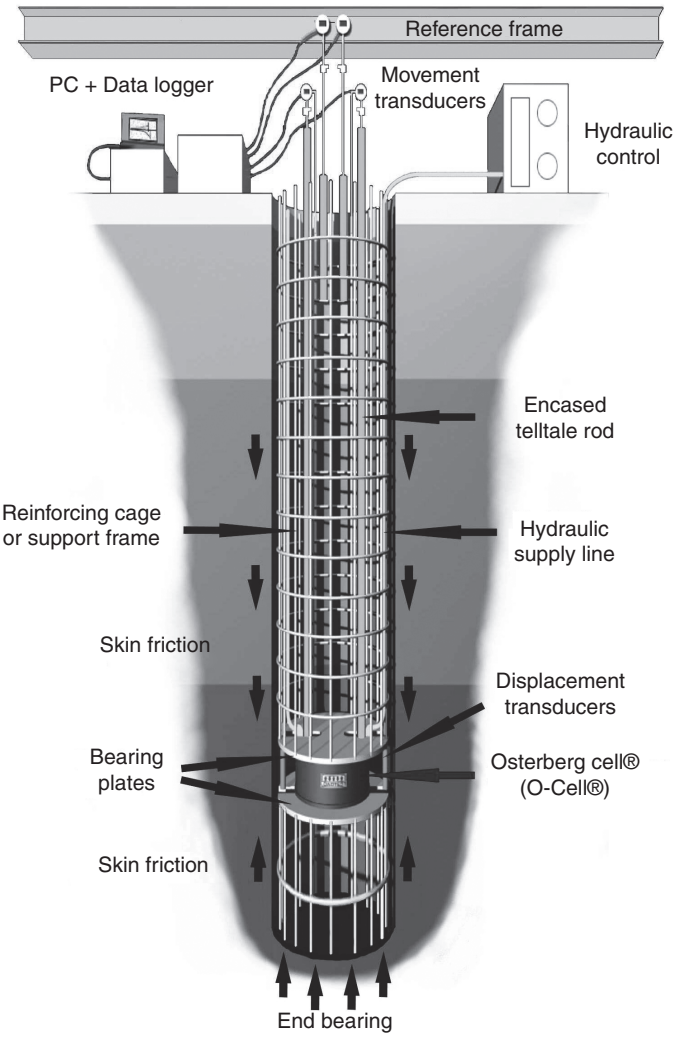


Figure 98.8 Schematic of single level O-Cell testing arrangement
Image supplied courtesy of Fugro Loadtest; all rights reserved



Figure 98.9 Insertion of the O-Cell in the reinforcing cage of a diaphragm wall
Photograph courtesy of Fugro Loadtest; all rights reserved

2 MN to 27 MN. Test loads as high as 320 MN have been applied using several O-Cells installed at the same level (www.loadtest.com).

98.3.3 The advantages and disadvantages of bi-directional testing

The bi-directional type of load test has several obvious advantages over static top-down methods. The systems require no large surface reaction such as kentledge or anchored steel reaction frames thus reducing space requirements, set-up time and transportation costs. The system is also safer with the loads being applied at depth. There are also reported cost savings with bi-directional load testing being comparable in cost with top-down static load tests at 5–10 MN (SPERW, 2007) but then becoming much more cost effective at higher loads. There is also the potential with the O-Cell to apply loads that cannot be achieved by top-down reaction or kentledge systems. Although the design of the O-Cell lends itself to use in auger bored cast *in situ* piles it has also been successfully been deployed in continuous flight auger piles (CFA), Fundex piles and both steel and concrete driven piles.

Another advantage is to use an O-Cell bi-directional test as a substitute for tension tests at high loads and push the pile up instead of attempting to pull the pile upwards from ground level.

One specific disadvantage associated with the bi-directional approach is that the jacking system needs to be pre-installed. This means it is not possible to select a random working pile for testing. The other disadvantage of the system is that the components installed within the pile are not recoverable after testing. This means the O-Cell and some of the instrumentation is sacrificial which has a cost implication.

98.3.4 Interpreting the data from bi-directional testing

Typical results from a single O-Cell installation similar to that shown in **Figure 98.8** are shown in **Figures 98.10** and **98.11**. To estimate the load-displacement behaviour at the head of the pile several techniques can be used. The two most direct methods are to sum the measured responses and include the additional elastic shortening or alternatively model the response of each of the pile elements using Cemsolve (Fleming, 1992) and then add together the geotechnical behaviour and corresponding total anticipated elastic shortening.

The total load-displacement responses can be obtained by adding together the loads measured above and below the O-Cell (at the same level of displacement). Where one section has mobilised less displacement than the other, for instance in the case of the movement of the upper section (**Figure 98.10**), it may be necessary to extrapolate the obtained results using methods such as the Chin approach (Chin, 1972) for a single hyperbolic function or the Cemsolve approach (Fleming, 1992) to obtain the sum of the measured response as shown in **Figure 98.11**.

During the bi-directional loading some elastic compression is contained within the results. During top-down static loading

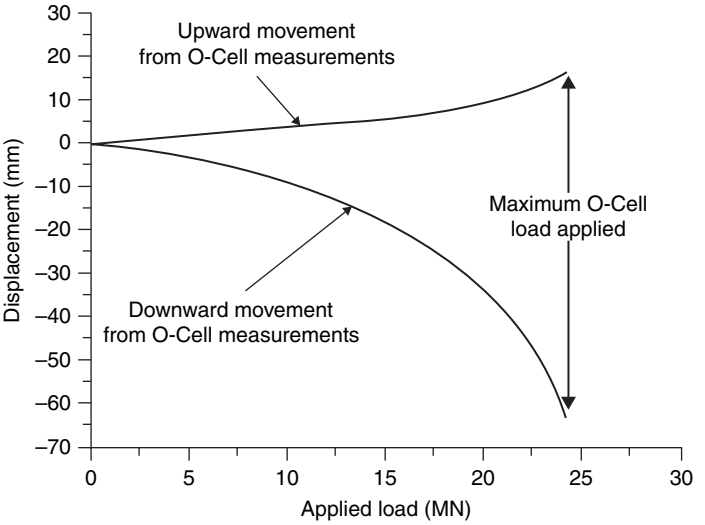


Figure 98.10 Example of results from a typical bi-directional load test
Data supplied courtesy of Fugro Loadtest

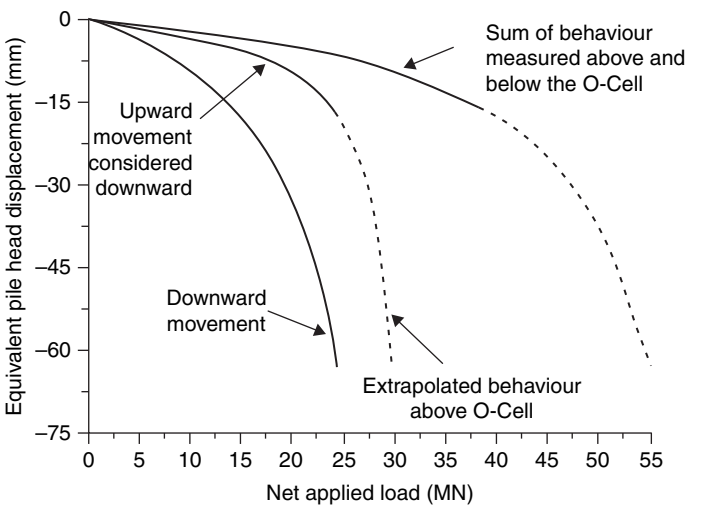


Figure 98.11 Sum of the measured responses from a bi-directional load test
Data supplied courtesy of Fugro Loadtest

where the full load is applied to the pile head additional elastic shortening would occur. This feature can be modelled and added to the results of bi-directional loading. Alternative approaches to analysis are discussed by England (2008).

98.3.5 Standardisation and guidance for bi-directional testing

As a relatively new method of pile testing bi-directional testing has historically suffered from a lack of guidance to aid specification; for instance, it is not specifically mentioned in Eurocode 7. This has been addressed recently in the UK by the second edition of the specification for piling and embedded

retaining walls (ICE, 2007). The Federation of Piling Specialists (FPS, www.fps.org.uk/) has also published guidance on use of the technique as part of a general pile testing guide. A more critical appraisal of the technique with case study comparisons can be found in a review of innovative pile testing techniques by Paikowsky (2004). For Eurocode 7 purposes (and other codes and specifications where bi-directional testing is not specifically mentioned), it may be adequate to class the O-Cell test as a full-scale static load test.

98.3.6 Specifying a bi-directional test

Care needs to be taken when specifying systems where the jack is to be installed in a preliminary test as the optimum level will be where the frictional capacity upwards (above the cell) matches the available reaction below the cell. In an ideal test, the skin friction below the cell (downwards) is fully mobilised with sufficient of the end-bearing mobilised to allow characterisation before the ultimate skin friction above the cell (upwards) is mobilised. Where elements of the pile are not fully mobilised, back analysis to reveal ultimate capacity may be used (as discussed above) where sufficient movement has occurred.

If the required test loads cannot be accommodated at a single level due to limitations on pile diameter it may be appropriate to consider multiple installation levels so that the entire foundation element may be mobilised in turn during phased testing. A specific advantage of using multiple level jack installations is that it allows individual pile sections to be mobilised separately. Cells placed close to the pile tip should also give due regard to the quality of the base and concrete at this level with cells typically placed one to two diameters above the base.

As the O-Cell applies load via jacking systems it is possible to specify load increments, hold periods and loading rates as per static tests.

98.4 High strain dynamic pile testing

98.4.1 What is high strain dynamic testing?

The previously discussed load application techniques used to measure pile capacity typically have load application durations in terms of hours. Dynamic pile testing differs in that loading the pile occurs by a drop weight applying a very short duration impact (≈5 ms) load at the head of the pile. The impact of the weight produces a stress wave that travels down the pile. Where the movement of the pile is resisted or there is a change in impedance such as at the pile tip, a wave will be reflected back up the pile. Based upon comparing the waves travelling up and down the pile it is possible to assess the dynamic pile reaction. Through further analysis it is possible to derive the equivalent static pile load-displacement behaviour.

98.4.2 Methods available for dynamic testing

Dynamic testing has the advantage that the main equipment required for monitoring the test is relatively compact and where the piles are being driven on site the normal pile-driving hammer can be used to produce the required stress wave. Piles are typically tested some time after installation which is referred to as re-strike. This involves the piling rig returning briefly to the test pile to deliver a series of hammer blows (typically five, depending on the approach taken). If the tests are being carried out on a cast *in situ* pile or are re-strike tests of driven piles where pile driving for installation is complete then it may be necessary to mobilise a separate drop weight system (**Figure 98.12**).

98.4.3 How does dynamic testing work?

Measurements are taken during the hammer impact from a pair of accelerometers and strain gauges that are attached above ground level to the pile head either during or after driving (**Figure 98.13**). The response of this instrumentation is logged during and after the weight impact on a specifically designed logging and analysis device, for example the Pile Driving Analyser (PDA) produced by Pile Dynamics Inc. Data from the instrumentation are used to derive the force (F) applied to



Figure 98.12 SIMBAT mini dynamic drop weight system (1000 kg hammer). Note pile head instrumentation installed on exposed concrete after local removal of temporary casing
Photograph courtesy of Testconsult Ltd; all rights reserved

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the pile by multiplying the measured strain by the cross-sectional rigidity of the pile and the velocity (v) from integration of the accelerometer readings.

The force measured by the strain gauges at the pile head due to the hammer impact is compared with the equivalent force derived from measurements made by the accelerometers (**Figure 98.14**). The data shown are for a re-strike test on a 250 mm square pre-cast concrete pile where the pile tip was 23.2 m below the strain gauges.

It is assumed, in stress wave theory, that both force and velocity are proportional as the wave travels through a material. Where:

$$v = \frac{F}{Z} \tag{98.4}$$

$$\text{or } F = v \times Z \tag{98.5}$$



Figure 98.13 Installation of instrumentation (strain gauge and accelerometer) on a pre-cast concrete driven pile (pile hammer visible at top of image)

$$\text{the pile impedance } Z = \frac{E_p A_p}{c} \tag{98.6}$$

where E_p is the Young's modulus of the pile, A_p is the piles' cross-sectional area and c is the wave velocity in the pile which can be found based upon the stiffness and density of the pile (ρ_p):

$$c = \sqrt{\frac{E_p}{\rho_p}}. \tag{98.7}$$

Where the movement of the pile is resisted, or there is a change in impedance such as at the pile tip, a wave will be reflected back up the pile (**Figure 98.15**). The total dynamic resistance of the pile (R) to the stress wave passing up and down the pile has been shown to equal the sum of the downward travelling force ($F_{(t_0)}$), force measured at maximum pile head velocity, ($v_{(t_0)}$) plus the upward travelling force that arrives back at the pile head ($F_{(t_0+2L/c)}$) at approximately $2L/c$ (where L is the pile length, **Figure 98.15**) after the initial peak load (Rausche *et al.*, 1985; Randolph, 2003).

$$R = \frac{F_{(t_0)} + F_{(t_0+2L/c)}}{2} + Z\left(v_{(t_0)} + v_{(t_0+2L/c)}\right). \tag{98.8}$$

98.4.4 Advantages and disadvantages of dynamic testing

The advantages of dynamic testing come from the simplicity of the equipment and the speed of testing. If driven piles are being tested it is typical to test at re-strike at some time after driving. The on-pile strain gauge and accelerometer are quickly bolted to the test pile (**Figure 98.13**) with logging undertaken on a small hand-held logging and analysis system (**Figure 98.16**). This means the system can be operated by a single operator with quick instrument installation causing little disruption to construction. As the piles can be tested during installation

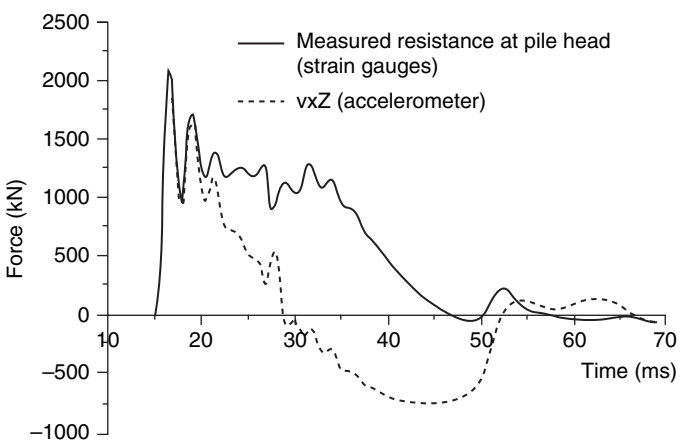


Figure 98.14 An example of measured pile stress wave data
Data provided by Technical Services Ltd

many working piles can also be tested without concerns over testing altering working performance. This data collection process has been improved recently with the development of wireless instrumentation and the option of replacing strain gauge load determination with direct pile head load cell readings.

If the piles are cast *in situ* piles or are driven piles tested after the original pile driving rig has been demobilised then it is possible to mobilise a drop weight system. Tests up to 36 MN have been undertaken by dropping 36 tonne masses. Typical loads achieved are 1 MN for 1000–1500 kg drop weight systems (typically 1.5–2% of the applied load is required) and 3 MN for 4000 kg drop weight systems. The ability to produce large loads with limited drop mass significantly reduces cost (**Figure 98.12**).

Other benefits associated with the nature of the test mean that it may be possible to verify the integrity of the pile during normal testing. Through analysis it is also possible to investigate the distribution of resistance down the pile and the split of base to shaft resistance without the need for additional instrumentation as is required in techniques where only pile head measurements are made.

Due to the nature of using the test during pile driving the test only gives an indication of the pile resistance during the installation process. Depending on the soil type the capacity of the pile may change significantly after driving due to, for instance, the dissipation of pore pressures which may lead to increases in capacity with time, which is often referred to as set-up. This may lead to the need for re-strike testing at various times after driving to verify pile capacity. This is not a problem specific to dynamic testing but may need to be considered for any pile testing method where displacement piles are being tested.

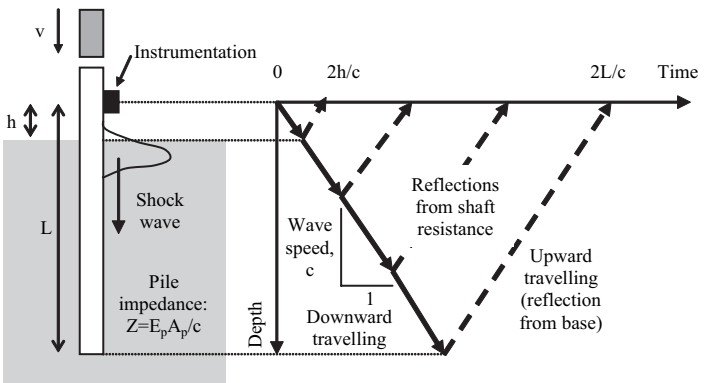


Figure 98.15 Schematic of stress wave travel in a pile
Reproduced from Randolph (2003)



Figure 98.16 Pile Driving Analyzer® capturing data during pile driving from wireless instrumentation
Photograph courtesy of Technical Services Ltd; all rights reserved

98.4.5 Interpretation of dynamic testing

It has previously been briefly explained how the dynamic pile resistance can be derived from pile head measurements during dynamic testing. It is then necessary to determine the ultimate static pile resistance from these measurements. Several methods are available to do this with the most common approaches based upon signal processing and numerical models (Holeyman, 1992).

98.4.5.1 CASE method

One of the earlier methods developed for signal processing is referred to as the CASE method (Rausche *et al.*, 1985). This simple technique may be used to quickly assess capacity in the field based upon an individual blow. It requires the determination of a damping force (R_d) which is assumed to be proportional to the pile tip velocity (v_{tip}) that can be removed from the total dynamic resistance to give the equivalent static resistance.

$$R_d = JZv_{tip} = J\left(F_{(t_0)} + Zv_{(t_0)} - R\right) \quad . \tag{98.9}$$

The performance of this technique is dependent on the soil-dependent CASE damping factor (J). Suggested values for the damping factor are shown in **Table 98.6**.

98.4.5.2 Signal matching

The most common methods of analysing dynamic load tests presently used are based upon lumped parameter finite

Soil type	Damping factor, J (s/m)
Sand	0.05–0.20
Silty sand/sandy silt	0.15–0.30
Silt	0.20–0.45
Silty clay/clayey silt	0.40–0.70
Clay	0.60–1.10

Table 98.6 Values for CASE damping coefficient
Data taken from Rausche *et al.* (1985)

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difference or finite element techniques where the pile is modelled as an assembly of interconnected masses with varying properties. These properties, predominantly soil parameters, are varied until computer-simulated pile head forces and velocities match those measured. Several computer packages have been developed that utilise this 'signal matching' method such as CAPWAP (Case pile wave analysis program), TNOWAVE and SIMBAT (Stain, 1992).

Common issues that may affect analysis are:

- insufficient pile settlement mobilised during testing to allow determination of ultimate pile capacity;
- variation in material properties or cross-section down the pile length;
- stresses locked in the pile as a result of pile driving.

It should be noted that these issues are not specific to dynamic testing and may cause problems to both static and rapid pile testing, especially where pile instrumentation is being used to supplement testing.

To allow the ultimate behaviour of a pile to be assessed from dynamic analysis it is necessary to mobilise adequate settlement in the pile and allow appropriate strain levels in the surrounding soil. Dynamic analysis based upon low settlement levels is likely to lead to a conservative assessment of pile performance, i.e. an underestimation of pile capacity. Where insufficient settlement is mobilised analysis is not compromised but reflects the mobilised soil resistance rather than ultimate pile capacity. Unfortunately, in the case of rapid load testing insufficient settlement may render analysis difficult. The situation of low mobilisation may come about due to ignorance of the importance of mobilising adequate settlement but more commonly it is a result of specifying rigs that are unable to apply the required load levels. Another reason for low mobilisation may be a limit on load/stress levels due to concerns over pile damage.

As dynamic test analysis is dependent on knowledge of the pile cross-sectional stiffness it is necessary to accurately know the cross-sectional profile of the pile and the stiffness of the pile material. These properties may be easy to determine for a steel or pre-cast concrete pile that is constructed to tight size tolerances and is cured in a factory environment. Where cast *in situ* piles are formed the profile of the pile may be less certain or complex, for instance in the case of a screw pile or under-reamed pile. There are also material issues, i.e. how does the inclusion of reinforcement affect the pile stiffness and does site-cured concrete have the same properties as laboratory-cured samples? Where piles have complex known geometry this can easily be incorporated in dynamic analysis if as mentioned the geometry is known. Where less information is available for the test, for instance in the case of pile re-use assessment where there is little information about the length, cross-section and quality of historic piles (Butcher *et al*., 2006), dynamic testing may not be appropriate.

98.4.6 Standardisation of dynamic testing

The dynamic pile testing method has been in common use for many years. In the UK, it is well documented in SPERW (2007) and the FPS general pile testing guide (FPS, 2007). In the US, there is an ASTM standard test method for high-strain dynamic testing of piles (ASTM D4945-08). One common thread throughout the guidance documents is the need for experience of the dynamic load testing in similar soils and on similar pile types (EC7). Where such experience does not exist or certain ground conditions prevail such as fine-grained and laminated soils it is recommended that the tests are calibrated against a site-specific static test (SPERW, 2007).

98.4.7 Specifying a dynamic test

It is important that the installation equipment being used for dynamic pile testing is first capable of driving the piles to the required depth or capacity and that it can mobilise the pile adequately during testing. Where time allows, details of the piles being driven and the driving equipment should be analysed prior to installation by the dynamic pile testing specialist. This wave equation analysis can be used to assess the ability of the proposed driving system to install the pile to the required capacity and desired penetration without exceeding allowable driving stresses in the pile material.

The other important decision to be made during dynamic testing is when to test, due to set-up and change of capacity with time. As mentioned, testing can be undertaken at the end of initial pile installation and through re-strike at any time after installation.

The piles to be tested dynamically should be designed to resist the driving stresses. They should also be designed to have at least 300 mm sticking up above ground at the end of pile testing to avoid sensor damage. To allow sensor installation it is important that there is adequate working room near the pile heads for sensor mounts to be installed prior to lifting piles. For cast *in situ* piles it may be necessary to include an extension to the top of the pile to approximately three pile diameters above ground level. This may be achieved by incorporating a section of casing above the pile head and filling with concrete while pouring the main pile concrete. Once the concrete has set windows may be cut in the casing to allow instruments to be attached to the concrete (**Figure 98.12**). Instrumentation is typically installed at a minimum of two pile diameters below the pile head.

98.5 Rapid load testing

98.5.1 What is rapid load testing?

The main difference between rapid load pile testing and static and dynamic testing is the duration of application of load. A rapid load test usually has a load duration of 90–250 ms which is approximately 30 to 40 times that of a dynamic pile load test (ASTM D7383-08; Holscher and van Tol, 2009). The duration of the loading has been designed such that piles less than 40 m in length remain in compression throughout the loading event

resulting in negligible stress wave effects and simpler analysis (**Figure 98.17**).

98.5.2 Methods available for rapid load testing

Several different techniques are available to produce rapid loading events. Statnamic testing works by rapid burning of a solid fuel that produces gas in a pressure chamber (**Figure 98.18**). The venting of this gas is used to accelerate a mass upward that in turn imparts a load on to the foundation pile below the Statnamic device. The load is applied and removed smoothly through the controlled venting of the gas. Alternatives to this approach typically take the form of a drop mass system where the load duration is increased through various cushioning techniques, for instance a spring or springs in the pseudo static pile load tester (Schellingerhout and Revoort, 1996) and spring hammer systems (Matsuzawa *et al*., 2008) and specialised cushioning in the hybrid dynamic test (Miyasaka *et al*., 2008).

98.5.3 What are the advantages and disadvantages of rapid load testing?

The main advantage of rapid load testing devices comes about as a result of the method of load application. For example, in the rapid load testing method known as Statnamic a reaction mass is accelerated upwards resulting in a load application of 3.5 MN from 18 tonne. Typically, for Statnamic testing the reaction mass is only 5% of the equivalent static load reaction mass required to produce the same load. This results in rapid load testing devices being significantly smaller and lighter than static equivalent set-ups. This means that they can be used on sites with space restrictions, are easier to mobilise and quicker to set up and test. For example, a 3 MN Statnamic rig (**Figure 98.19**) can be mobilised with one articulated truck and a 70 tonne crane. If the rig incorporates a hydraulic catch mechanism (catches the reaction mass) it can test up to 10 individual piles a day or allow multiple load cycles to be carried out on an individual pile with minutes between cycles.

Typically disadvantages of rapid load testing methods stem from the rapid nature of the load application which may result in piles displacing at rates measured in metres per second resulting in very high strain rates in the soil (**Figure 98.17**). In fine-grained soils, such as clay, such high strain rates have been shown to significantly increase the strength of the clay. This may result in pile ultimate capacity up to three times the measured or predicted static pile capacity (Holeyman, 1992). This effect appears to become more pronounced as the plasticity of the clay increases. It is important to be aware of this effect to

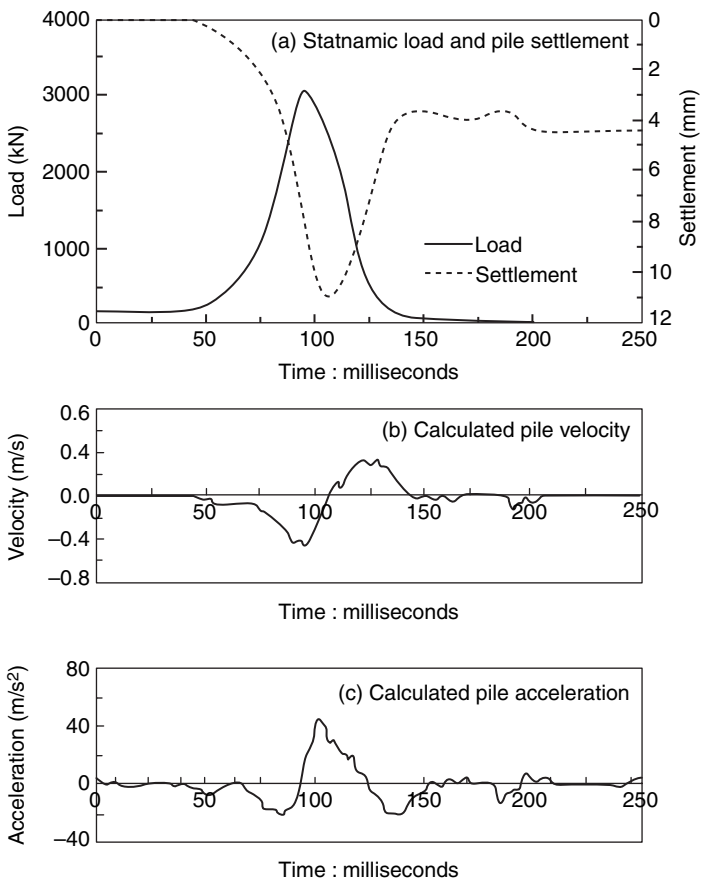


Figure 98.17 Measured and calculated results from a 3000 kN Statnamic load cycle
Reproduced from Brown and Hyde (2006)



Figure 98.18 Statnamic pressure chamber with load cell at its base shown mounted on a test pile

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avoid specifying rapid load testing devices that are incapable of adequately mobilising a pile. For instance, the maximum capacity of rigs currently available in the UK is 3 MN with larger rigs available from mainland Europe.

Another major drawback is that certain techniques such as Statnamic may operate over a very discrete loading range; for instance, a 3 MN Statnamic rig may apply loads as high as 3.5 MN but would not be able to apply loads below 700 kN.

98.5.4 How does Statnamic work?

As Statnamic is the most widely deployed method of rapid load testing it is described in more detail. As mentioned previously, load is applied vertically down a pile by burning a rapid-burning fuel which produces significant volumes of gas within a piston which accelerates reaction mass upwards. The duration and shape of the load pulse (**Figure 98.17**) are controlled by venting of the gas. The reaction mass is forced upwards and is caught by either gravel in the case of large load tests 20–60 MN or by hydraulic catch mechanisms for smaller rigs (3–20 MN, **Figure 98.19**). As Statnamic load application is independent of gravity the device can easily be used to test raked piles and has been deployed horizontally to test piles laterally and simulate impacts on structures.



Figure 98.19 3.5 MN Statnamic rig with hydraulic catch mechanism ready for testing

During the test, load is measured directly by a calibrated load cell incorporated in the base of the combustion piston which sits on the pile head (**Figure 98.18**). Displacement is also measured directly using a non-contact system either by a photovoltaic sensor mounted in the piston which is excited by a remote laser beam or the movement of targets mounted on the pile are recorded by means of a remote optical displacement tracking system. As the Statnamic event may induce stress waves in the ground it is important that the remote component of the measuring device is located far enough away from the pile to avoid disturbance during the duration of the loading event (Brown and Hyde, 2006). The recording of displacement–time history can then be differentiated once or twice to determine pile velocity and acceleration, both of which are required for test analysis.

It is also commonplace for the system to incorporate an accelerometer which can be used for direct recording of acceleration. The acceleration–time history can also be integrated to determine velocity which may be less 'noisy' than that found by differentiation and for a second time to verify optical or laser-based displacement measurements.

98.5.5 How to interpret the data

Rapid load tests must be analysed to remove both the effects of inertia associated with the pile/soil and strain rate-dependent response of the ground. The most common approach for this is referred to as the unloading point method (UPM).

Based on inspection of load-displacement curves from Statnamic tests Middendorp (2000) observed that during the unloading, the pile velocity reached zero which in coarse-grained soils corresponded approximately to the ultimate static resistance of the pile. This point (the unloading point) was then used to determine a constant damping coefficient to correct the measured Statnamic data for the velocity-dependent resistance of the soil. Unfortunately, when this technique is applied to piles installed in fine-grained soils there is a tendency for the ultimate pile capacity to be significantly overpredicted. In order to correct for this effect, a series of soil-dependent average correction factors were developed (**Table 98.7**) which the derived

Soil type	Rate effect factor (μ)	FOS without μ	FOS with μ
Rock	0.96	2.0	2.0
Sand	0.91	2.1	2.0
Silt	0.69	2.8	2.0
Clay	0.65	3.0	2.0

Note: FOS = Factor of safety

Table 98.7 Correction factors for UPM analysis
Information taken from Paikowsky (2004)

static load was then multiplied by to obtain a corrected UPM analysis (**Figure 98.20**). More recently, it has been proposed that much greater average correction factors in clay are required resulting in a μ value of 0.47 (Weaver and Rollins, 2010).

In response to the known shortcomings of the UPM technique, a series of alternative methods have been developed which have their basis in the velocity or strain rate-dependent behaviour of soils. One nonlinear velocity-dependent method of analysis was developed by Brown (2004). The main conceptual difference from UPM is that the method is reliant on the user input of soil-specific rate parameters:

$$F_u = \frac{F_{STN} - Ma}{1 + \left(\frac{F_{STN}}{F_{STNpeak}}\right)\alpha\left(\frac{\Delta v}{v_0}\right)^{\beta} - \left(\frac{F_{STN}}{F_{STNpeak}}\right)\alpha\left(\frac{v_{\min}}{v_0}\right)^{\beta}} \qquad (98.10)$$

where F_u is the derived static pile resistance, F_{STN} is the measured Statnamic load, Ma is the pile inertia, Δv is the pile's velocity relative to the soil and v_{min} is the velocity of the static CRP pile test used to define the soil-specific rate parameters α and β (β is normally set to 0.2 in clay). Both Δv and v_{min} are normalised by v_0 which is assumed to be 1 m/s. This form of analysis has evolved from the analysis of dynamic tests where the majority of the pile capacity is developed through skin friction (Randolph, 2003). The value of α may be selected based on the tentative relationship with plasticity index (PI) as proposed by Powell and Brown (2006):

$$\alpha = 0.03PI(\%) + 0.5\,. \qquad (98.11)$$

98.5.6 Guidance and standardisation of rapid load testing

Like bi-directional testing, rapid load pile testing has historically suffered from a lack of guidance to aid specification. This has again been addressed recently in the UK by SPERW (2007) and the FPS pile testing guide (FPS, 2007). In the US, there is an ASTM standard for rapid load testing of deep foundations (ASTM D7383-08) along with guidance produced for the federal and state highways agencies (McVay *et al*., 2003; Paikowsky, 2004). The Japanese Geotechnical Society has also produced a testing specification, an English language draft of which was published in 2000. A European standard within the Eurocode framework is currently under development and will be published along with guidance on test interpretation (BS EN/ISO 22477-4; Hoelscher *et al*., 2010). Rapid load testing is also mentioned briefly in EN1536-2000 with regard to the testing of bored piles and the need to provide evidence of correlation with static maintained load tests in similar ground.

98.5.7 Specifying a rapid load test

Prior to specifying a rapid load test it is necessary to have access to the results of geotechnical investigation and laboratory testing to allow selection of adequate testing equipment and loading levels. The encountered ground will also influence the analysis approach adopted. For analysis of tests in fine-grained soils, regular determination of the plastic and liquid limits should be undertaken down to at least the depth of the toe of the pile.

In granular soils and pile installed in rock, the applied loads will be similar to the anticipated ultimate pile capacity with additional loading capacity required for the inertial forces generated by the pile and soil. In addition to the inertial resistance, load testing devices used in fine-grained soils such as clays will need to be capable of applying additional loads due to strain rate effects. For example, in a low to medium plasticity clay the ultimate capacity measured during rapid loading may be 1.8 times greater than the anticipated static capacity (**Figure 98.21**).

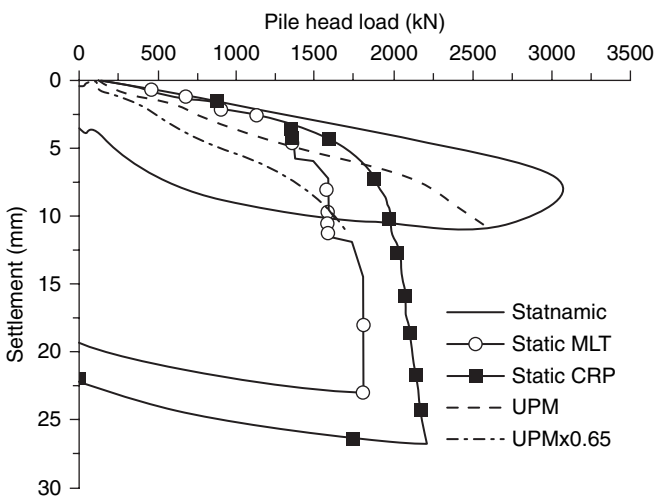


Figure 98.20 Performance of UPM correction of Statnamic data. The pile tests have been reset to zero settlement to allow comparison

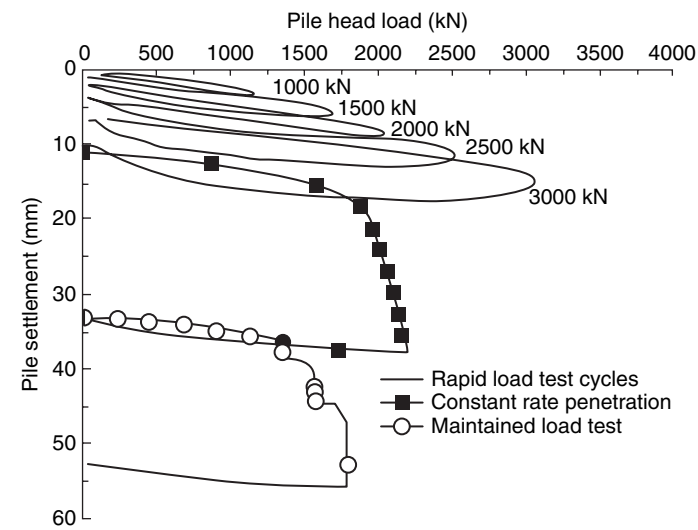


Figure 98.21 Cycles of Statnamic loading compared with CRP and MLT static tests (see Figure 98.1)
Reproduced from Brown *et al*. (2006)

In high plasticity clays the test may need to apply loads of 3.5 times the static capacity or higher. The need to apply such high loads should be considered when designing and specifying test piles.

98.6 Pile testing safety

98.6.1 Why do we need to be careful?

The testing of piles poses a safety hazard for several reasons. The most obvious of these are the high tensile and compressive loads generated in both the pile and reaction system during testing. Failure in any part of the system may lead to rapid release of energy and collapse of the testing arrangement. Other issues may arise due to the specific reaction system: for instance, kentledge relies on placing weights above a supporting frame, this frame needs to be stable throughout erection and testing thus requires appropriate structural design and checks on bearing capacity.

98.6.2 Good practice

As relatively high loads are applied over the relatively small area of the pile head it is important that all attempts are made to reduce eccentricity of loading on the pile and in other areas of the reaction system such as connections to anchor piles. Excessive eccentricity can lead to large moments that the load or reaction system has not been designed for. Eccentricity can be minimised by accurate setting out of reaction piles relative to the test pile, correct alignment of the load column and proper mating of contact surfaces and levelling of reaction beams.

An obvious means of reducing risk to site and testing operatives is to restrict access to the load testing set-up and reduce the need to be close to the heavily loaded elements or reduce the need to work under kentledge. This can be achieved by restricting access only to relevant personnel. The need for direct access to the load column can also be reduced or removed through the use of computer-controlled loading jacks and displacement transducers. Fully automated testing systems have been developed which can be controlled remotely (England, 2002) although it is generally advised that tests should not be left unattended whilst loading.

Good practice can also be incorporated in design of the loading system. For instance, where the testing system uses reaction piles this should ideally incorporate a minimum of three anchor piles as systems based upon two reaction piles are significantly less stable. The use of a two-pile reaction system should be reserved for relatively low loading levels with high tolerance pile and reaction system positioning.

98.6.3 Loading and inspection

It is important that the maximum test load to be applied to the pile is agreed in advance of the test so that the test pile/pile cap and all associated elements of the load testing equipment can be designed and specified to apply the maximum test load safely. This information must be communicated to the relevant operatives and where possible relevant limiting capacities should be marked on individual elements of the testing equipment (see **Figures 98.4**, **98.5**, **98.6**).

It may be the intention during testing to take the pile to its ultimate capacity and cause significant permanent settlement. In some cases a pile's capacity during testing may exceed that anticipated prior to testing and require additional loads to prove ultimate behaviour. Although the goal may be to prove ultimate behaviour, this should never be used to justify exceeding the maximum test load the testing system has been designed for.

During the course of the load test the whole system should be monitored for eccentricities and excessive deflections with appropriate action taken if this occurs. Typical signs of distress may include:

- excessive deflection of the reaction beams;
- upward movement of tensile bars cast into reaction piles;
- horizontal deflection of the reaction system;
- movement of kentledge;
- difficulty in maintaining test loads.

Further key safety issue guidance is given in FPS (2006).

98.7 Simple overview of pile testing methods

Selection of the most appropriate pile testing technique will vary from one situation to the next. Criticism is often levelled at the different techniques for their shortcomings whereas any form of pile test (if correctly undertaken and analysed) should be viewed as a tool to give confidence in the design assumptions made, improve efficiency in construction and verify as-built performance. For instance, where a static load test identifies problems with trial pile performance during working pile construction, rapid or dynamic testing techniques can be rapidly deployed to assess the effect on the constructed working piles. This may then allow modifications to pile design and construction that can be used to mitigate problems during construction without significant effect on the construction programme. **Table 98.8** summarises some of the pros and cons of the methods described in this chapter and offers suggestions for potential deployment considerations. This table only serves as a quick summary and should not be used for definitive selection of a test. Decisions on the most appropriate testing method are influenced by many factors: for example, if there is a pile driving rig on site it will likely be more cost effective to undertake dynamic testing than to mobilise another technique. Definitive guidance should be sought from the specific providers of such techniques who can offer cost-effective and bespoke solutions both for deployment and subsequent analysis and interpretation.

Test Type	Advantages	Limitations	Potential deployment
Static	• Simple test • Simple & quick interpretation • Well understood & accepted	• Slow tests • Significant infrastructure (especially as loads increase) • High space requirements • Safety concerns with increasing loads	• Lower loading (<20 MN) • Low pile numbers • Trial piles/limited number of working piles
Bi-directional	• Capable of very high test loads (higher than other techniques) • Low infrastructure • Low space requirement • Cost effective as load magnitude increases	• Pile for testing needs to be preselected • Less experience • Analysis needs to take account of different surface boundary conditions • Specialised analysis and interpretation	• Medium to very high loading (2–320 MN) • Low pile numbers • Trial piles/limited working piles
Dynamic	• Low infrastructure • Low space requirement • Fast tests • Quick repeat testing • Mature technique	• Pile damage may be a concern • Perceived reliability: Intermediate • Tests may be influenced by pile material and geometry • Specialised analysis and interpretation	• Medium to high loading (1–35 MN) • Low to high pile numbers • Trial and working piles • Where pile driving equipment already on site
Rapid	• Low infrastructure • Low space requirement • Rapid testing • Quick repeat testing	• Limited experience • Analysis techniques under development • Limited case study experience • Availability of high capacity equipment	• Medium to high loading (0.6–40 MN) • Medium to high pile numbers • Trial and working piles • Quality control of working piles • Problems with working pile performance

Table 98.8 Summary of pile test characteristics and potential deployment criteria

98.8 Acknowledgements

The author would like to acknowledge the input of Melvin England (Fugro Loadtest UK) and Mike Kightley (Technical Solutions Ltd) on the Osterberg Cell and dynamic testing sections respectively. The author is grateful to Andrew Bell of Cementation Skanska Foundations, Simon French of Testconsult Ltd and Paul Hoelscher of Deltares for the supply of key photographs.

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98.9.1 Useful websites

Federation of Piling Specialists (FPS); www.fps.org.uk
Spring Hammer Rapid Load Test; www.spring-hammer.com
Pile Dynamics Inc. and GRL Engineers Inc.; www.pile.com
Fugro Load Test; www.loadtest.co.uk

It is recommended this chapter is read in conjunction with

- Chapter 22 *Behaviour of single piles under vertical loads*
- Chapter 54 *Single piles*
- Chapter 81 *Types of bearing piles*
- Chapter 82 *Piling problems*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 99

Materials and material testing for foundations

Stuart Pennington Ove Arup & Partners, London, UK

An introduction to construction materials and their verification for the geotechnical engineer is presented. Verification is discussed in terms of its forms, levels of complexity, documentation and appropriate tests, with an emphasis on UK practice. A selection of materials are introduced and discussed in terms of verification, and an introduction to the verification of existing foundations for re-use is presented.

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99.1 Introduction

The geotechnical engineer is faced with a variety of construction materials as most geotechnical structures involve more than just soil and rock. There is often an overlap with other disciplines, such as structural engineering or materials technology. Verification of these materials during construction requires an understanding of the designer's intent, the characteristics of the material being verified and practical experience of the verification process.

This chapter introduces verification and presents a selection of common materials encountered in the construction of geotechnical structures, with an emphasis on UK practice and standards to illustrate the verification process. Verification by testing is addressed in terms of the general requirements for each material rather than the requirements for a material in a specific structure (e.g. grout rather than grout for anchors). The chapter concludes with an introduction to foundation re-use. Discussion of pavements and contamination is not included in this chapter.

At the end of this chapter, a selected list of documents and websites provide more specific information on the topics addressed throughout the chapter.

As there are numerous overlaps for the issues discussed in this chapter with other parts of this manual, the reader should peruse the contents list to identify potential sources for more detail. This is particularly relevant to topics such as earthworks, integrity testing, ground investigation, ground improvement, site supervision, fill, pavements and reinforced soil.

99.2 Eurocodes

The introduction of European Standards has made construction verification a requirement for designs under their jurisdiction. Documents relevant to material verification include:

- EN eurocodes, e.g. BS EN 1997:1–2004 (BSI, 2004);
- national annexes, e.g. NA to BS EN 1997:1–2004 (BSI, 2007);
- investigation and testing standards, e.g. BS EN 12620–2:2002 (BSI, 2002);
- execution standards, e.g. BS EN 12715:2000 (BSI, 2000);
- non-contradictory complementary information, e.g. BS 8004:1986 (BSI, 1986).

This array of documents shows that there are many scattered references to construction verification of materials that an engineer is required to consider.

99.3 Materials

Geotechnical structures often involve a variety of construction materials in addition to soil and rock such as polymers, timber, steel, concrete and grout. The processes of production, transport, placement and use will influence the method of verification. For example, materials can be:

- relatively unprocessed single materials (e.g. timber);
- processed composite materials (e.g. concrete);
- imported to a particular site (e.g. fresh concrete);
- transported within a particular site (e.g. excavated soil);
- used as is (e.g. *in situ* soil);
- used with other materials (e.g. geosynthetics);
- in a ready-to-use format (e.g. reinforcement cages);
- placed underwater (e.g. fresh concrete);
- used underwater (e.g. piles);
- in need of specific on-site management (e.g. fresh concrete);
- re-used (e.g. old piles).

While specialist understanding of a material is necessary to determine appropriate methods of verification, the practical application of verification of a material can be undertaken once a more general understanding of its composition, potential problems and suitable methods of testing is gained.

99.4 Verification

Verification of materials during construction is the process of checking (by an appropriate or specified method) that the materials brought to site and used in a structure are what the designer intended. While the designer's intent and requirements should be clear in the construction specifications and drawings, a face-to-face meeting with the designer will be invaluable. It should be recognised that verification other than that specified by the designer may be necessary or prudent.

When one thinks of verification of materials, testing in a laboratory may spring to mind. However, this is only one part of the process and should be complemented by other verification procedures such as:

- visual inspection;
- collection and review of records and certificates (e.g. concrete lorry tickets);
- review of quality control procedures at the point of manufacture (e.g. geosynthetics);
- engineering judgement;
- appropriate sampling;
- field testing (e.g. flow table for concrete);
- comparable experience;
- source quality control and certification (e.g. Conformité Européenne (CE) marking);
- understanding the material being verified;
- knowing when to request additional testing;
- being prepared to say that a material is not satisfactory.

The level, quantity and complexity of verification, and responsibility for verification, will depend upon factors such as:

- the category of the structure in which it is used;
- the level of quality control provided at source;
- the type of material;
- the parameters used in design and the extent to which the designer is relying on the testing of materials during construction;
- the importance or size of the structure;
- material origin;
- the construction contract.

Guidance is included in standards such as BS EN 206 (BSI, 2000a) and BS EN 1990 (BSI, 2002).

In considering what verification is required, the following questions should provide the engineer with a starting point for further thought:

- What are the verification requirements in the designer's specification?
- What are the referenced standards in the specification and in the construction contract?
- Who is responsible for the various aspects of verification?
- Is the material covered by a manufacturing standard (e.g. CE marking)?
- What can go wrong with the material once it arrives on site?
- What level of responsibility do you have as a geotechnical engineer on site?
- Is on-site testing required to progress or verify the design?
- Does the specification adequately address verification requirements?
- What level of verification control is required?
- Are the records that are being provided adequate?
- Are there materials that are not covered by standards?
- What has happened to the material since it arrived on site?
- Is the material being stored and handled correctly?
- What are the limitations of any controlling standards?
- What is the process for rejecting a material?
- What are the requirements for personnel or companies undertaking material testing?
- Has a plan for verification been created?
- Is quality control at the material source satisfactory?
- How are the results of verification to be communicated to the designer?

99.5 Concrete

99.5.1 General

The use of concrete in geotechnical applications is extensive; for example, piles, shallow foundations, retaining walls, facing units, tunnel linings and masonry blocks.

In simple form, concrete is comprised of cement, fine and coarse aggregate and water. Typically by volume, cement is 6–16%, fine aggregate is 20–30%, coarse aggregate is 40–55% and water is 12–20%. The water–cement ratio can range from 0.3 to 1.0 (Illston and Domone, 2001). A more complex mix can include admixtures and cement replacement materials which enhance its performance. Admixtures can include plasticisers, superplasticisers, accelerators, retarders, air-entraining agents and foaming agents. Cement replacement materials can include pulverised fuel ash/fly ash, ground granulated blast furnace slag, silica fume, metakaolin and natural pozzolans.

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Concrete is batched using hopper dispensers and mixing drums which combine the constituents (**Figure 99.1** shows an example of a batching plant). Concrete is either poured or pumped into the ground (e.g. pile) or formwork (e.g. retaining wall or pre-cast unit). The use of a tremie may be required, particularly if concreting is undertaken below water, to prevent segregation (see **Figure 99.2** for an example of tremie use on a bored pile). Concrete placed in formwork is typically vibrated when fresh to expel air and compact it. Concrete hydrates and typically achieves an initial set after two to four hours, a final set after many hours, and then hardens over weeks to months.

Concrete is typically delivered to site as ready-mix or as pre-cast units, although on-site batching may be undertaken in special circumstances.



Figure 99.1 Concrete batching plant
Courtesy of Texnokat



Figure 99.2 Tremie being used to pour concrete into a bored pile

99.5.2 Potential problems

Problems with concrete can occur when it is fresh, during setting, or when it has hardened, as a result of the mix proportions, the mix constituents, the environment in which it is used or how it is handled during placement.

While concrete is fresh it can suffer with respect to its consistency, compactability and cohesiveness. For example, if workability is reduced it may be difficult to plunge a reinforcement cage for a continuous flight auger (CFA) pile (**Figure 99.3** shows an example of a CFA reinforcement cage becoming stuck and bent during plunging). Alternatively, if concrete is dropped through water that has collected at the base of a pile, it may segregate.

When concrete is setting it can suffer from bleed, plastic settlement and plastic shrinkage. Once concrete has hardened it can suffer from durability issues caused by internal or external attack, and by physical or chemical attack (e.g. sulfate, sea water, acid, alkali–aggregate reactions, abrasion and freeze/thaw).

During the time that concrete is fresh or setting, the problems it may encounter are mainly associated with mix design

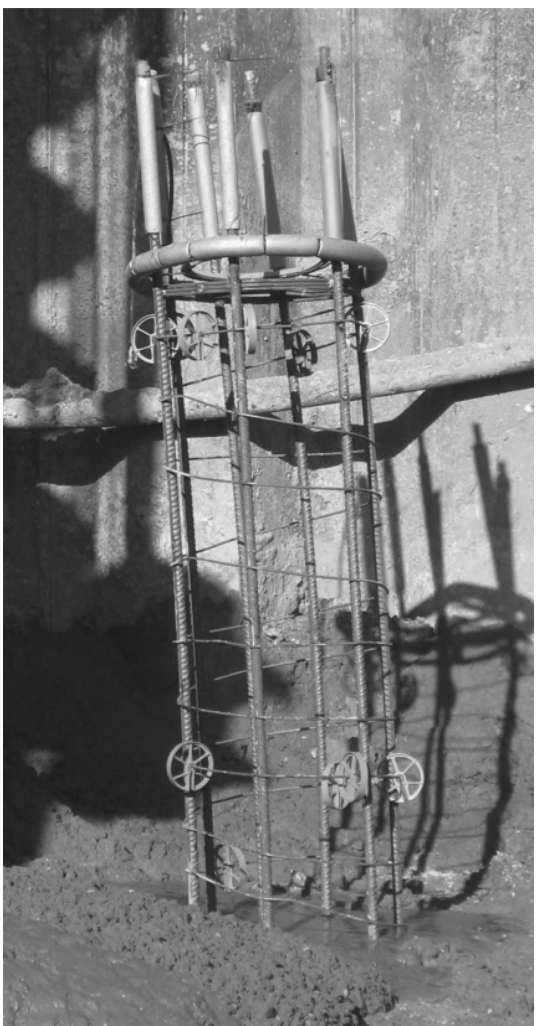


Figure 99.3 Partially plunged and bent CFA cage

and handling, whereas problems during its design life are more typically associated with mix constituents and quality of construction.

99.5.3 Verification

Verification of concrete starts with the mix design which can be specified in various ways. For example, a designed mix is chosen by the concrete supplier; a prescribed mix is chosen by the designer. Mix constituents and proportions are covered by standards and are typically controlled by requirements for workability (short-term performance), durability (long-term performance) and strength. If the mix design is not typical then trial mixes in a laboratory and subsequently in the field may be required.

For concrete batching plants supplying ready-mix concrete or factories manufacturing pre-cast concrete, there should be quality controls in place that monitor performance as detailed in the relevant standards. Ready-mix suppliers may subscribe to QSRMC (UK quality scheme for ready-mixed concrete) and pre-cast concrete may be CE marked.

The first check for fresh concrete should be a review of the delivery ticket; the contents of which are specified by standard. The ticket includes information such as the time of batching, volume, strength class, mix description and consistency. No materials (including water) should be added to ready-mix concrete unless required by the producer and this should be recorded on the delivery ticket. On-site verification tests prior to use (which are covered by standards) address issues of consistency, compactability and cohesiveness. Consistency can be assessed by the slump test, vebe test or flow table test; compactability can be assessed by the compacting factor test; and cohesiveness can be assessed visually. Further verification typically involves the preparation of cube or cylinder samples which are then tested for strength (typically after 7, 28 and 56 days of curing). When referring to concrete strength grade under Eurocode terminology, for example C32/40, note that the '32' refers to the cylinder strength and the '40' to the cube strength.

Box 99.1 Case study – flow table test

The flow table test is similar to the slump test in that it measures the consistence of fresh concrete. The flow table test is suited to high consistency mixes, such as those used in CFA piles, which would be difficult to measure using the slump test. The test procedure is detailed in Part 5 of BS EN 12350–5:2000 (BSI, 2000), however, in simple terms it involves:

- compaction of concrete into a cone (see **Figure 99.4(a)**);
- removal of the cone to release the concrete (see **Figure 99.4(b)**);
- jolting of the concrete on a flow table;
- measurement of the spread of the concrete (see **Figure 99.4(c)**).

Subsequent to the checks, testing and sampling undertaken at delivery time, the quality control of ready-mix concrete, until it has hardened, relates primarily to execution rather



(a)



(b)



(c)

Figure 99.4 Flow table test (a) compaction of concrete into cone; (b) removal of cone; (c) measurement of flow width

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than material verification and generally involves inspection of placement techniques, compaction and curing.

Verification of hardened concrete can be undertaken indirectly, destructively or non-destructively. Indirect verification may involve inspection of surface quality or dimensional tolerance in the case of concrete cast using formwork. Destructive verification may involve full scale load testing of a specially prepared member or small-scale testing of a core sample (see **Figure 99.5** for an example of concrete core from a pile with defects). Non-destructive verification can include rebound hammer testing, resonant frequency testing and ultrasonic pulse testing (Illston and Domone, 2001); however, these tests need to be correlated with core sample testing. Non-destructive verification for piles is generally termed 'integrity testing'.

99.6 Steel and cast iron

99.6.1 General

The use of steel and cast iron in geotechnical applications is extensive. For example, structural sections can be used as piles (e.g. H-piles or sheet piles) or as temporary props. **Figure 99.6** illustrates the use of sheet piles to form a cofferdam. Reinforcing bars (generally 8 to 40 mm in diameter) are used in reinforced concrete, soil nailing and anchors. Steel tendons are also used in anchors. Cast iron is used in tunnels in the form of spheroidal graphite iron lining segments. Stainless steel can be used in the same manner as non-stainless steel, where additional protection against corrosion is required. Steel fibres are used in segmental concrete tunnel linings and shotcrete.

Steel and cast iron primarily consist of iron alloyed with carbon and to a lesser extent with other elements such as silicon (for cast iron), chromium and manganese. Steel is typically formed by hot rolling or cold working and then bent/welded to form more complex sections (reinforcement cages, girders, hollow tubes, etc.); the length of any section generally being limited to the length of a delivery truck (12 m). Cast iron, such as that used in tunnel lining segments, is moulded and then bolted together.

In the UK, structural steel is typically grade S275 or S355 (where 'S' denotes structural steel and the number represents the yield strength in MPa). Reinforcing bars are typically grade B500A, B500B or B500C (where the initial 'B' denotes reinforcing steel, the number represents the yield strength in MPa, and the final letter represents the ductility rating; A being the lowest).

99.6.2 Potential problems

The primary issue with steel and cast iron is their durability, as their strength and stiffness properties are closely controlled by factory production. They can be degraded by dry oxidation, but more so by wet corrosion which may be chemically or biologically based. However, it should also be noted that steel can be damaged mechanically once delivered to site if it is subjected to excessive bending.

The extent to which steel in reinforced concrete corrodes is very much dependent upon the density, quality and thickness of concrete cover (generally 75 mm or more for concrete cast against the ground), and if there any chemicals in the concrete that may attack the steel. In applications where steel or cast iron is exposed to the environment (e.g. sheet piles), alternative measures such as good design/detailing, coatings, cathodic protection or sacrificial thickness are necessary. **Figure 99.7** illustrates how corrosion can vary over relatively short distances. In this case, the steel in the splash zone (central part of photograph) has corroded significantly more than that above and below.



Figure 99.5 Concrete core showing defects



Figure 99.6 Steel sheet pile cofferdam

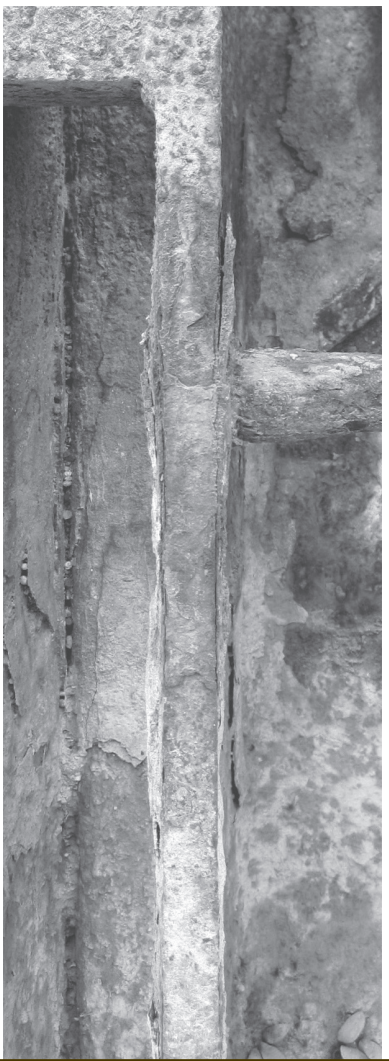


Figure 99.7 Variability in steel corrosion near the waterline

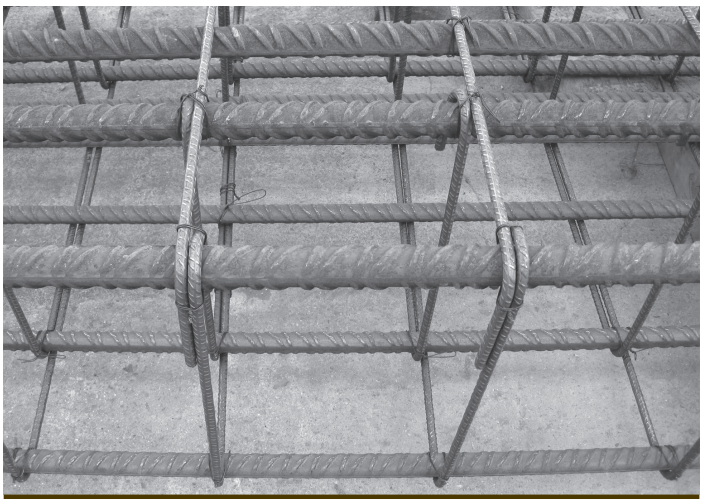


Figure 99.8 Reinforcement cage for barrette

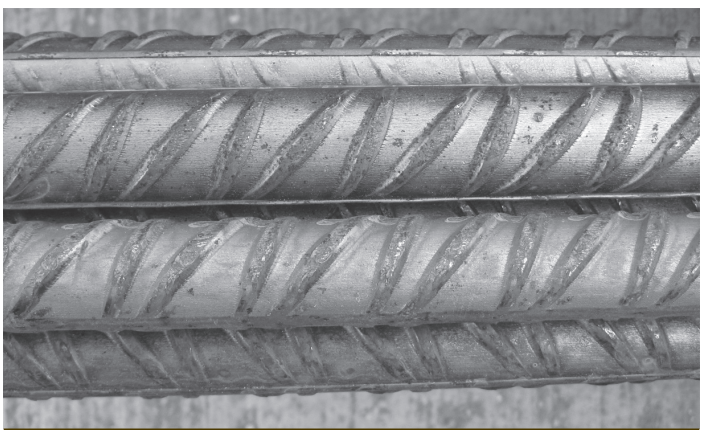


Figure 99.9 Reinforcing bars showing a rib pattern

99.6.3 Verification

For mills supplying steel and cast iron, there should be quality controls in place that monitor performance, as detailed in the standards. CE marking can be used to guarantee certain characteristics and producers often subscribe to CARES (Certification Authority for Reinforcing Steels).

Verification of steel and cast iron typically starts with their delivery to site where inspection of delivery records and the material should take place. Pertinent material characteristics should be checked on identification labels. Reinforcing bars produced by a CARES-certified body may have identifying rib patterns. The material should be free of scale rust, although a dusting of iron oxide is acceptable. Dimensions should be as specified, subject to allowable tolerances.

Samples can be taken for laboratory testing as there is not usually scope for on-site testing to verify material properties such as strength or composition. However, an example of where steel can be verified on site (albeit indirectly) is in the case of anchors – where steel tendons are tested non-destructively during the performance of load tests.

Where a protective coating is specified, for example on sheet piles, this should ideally be applied under factory conditions, and once delivered to site should be inspected for composition, integrity, quality and thickness.

Box 99.2 Case study – reinforcement cage check

Steel reinforcement used in piles or barrettes typically arrives on site as a cage (see **Figure 99.8** for an example of a barrette cage). Verification against construction drawings should comprise the checking of:

- overall dimensions (width, length, breadth, diameter);
- number of bars, bar spacing, bar diameter;
- integrity testing installations (e.g. sonic logging ducts);
- cleanliness (absence of rust and soil);
- steel grade via identification tags and rib patterns (see **Figure 99.9**);
- robustness for lifting purposes;
- interface/couplers for multiple cages.

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Verification of welding is not addressed here. However, it is worth noting that high yield strength bars should not be heated (e.g. by welding) as this can lead to them becoming brittle.

99.7 Timber

99.7.1 General

Timber can be used for driven piles, retaining walls, shoring/falsework and formwork. It can be described as a low-density, cellular, polymeric composite (Illston and Domone, 2001). At the microscopic level, timber is classed as:

- softwood (cells are primarily vertical, density generally lower, e.g. pine); or
- hardwood (cells are more anisotropic, e.g. oak).

At a smaller scale, timber can be subdivided into:

- sapwood (outer growing later);
- heartwood (dead centre section);
- latewood (formed late in growing season); and
- earlywood (formed early in growing season).

Timber should be obtained from sustainable sources and is typically dried before use. Natural timber is available as whole logs or as a sawn product. Timber can also be processed to produce products such as plywood for formwork applications, although these applications are not discussed here.

99.7.2 Potential problems

Natural timber cannot be controlled in the same way that, for example, steel production is. Its properties vary according to, for example, density, grain structure, temperature, knots, ring width, cell structure, moisture content, time and splits. There is a need to consider the small-scale structure as well as gross features. **Figure 99.10** illustrates the effect of a knot on timber structure.

Timber needs to be seasoned before use, otherwise it can be attacked by fungi and its strength and stiffness will be lower. Timber (particularly heartwood) has a natural resistance to deterioration provided its moisture content is kept low; ideally below 20% (Illston and Domone, 2001). However, it should be remembered that timber is hygroscopic (it attracts moisture from the atmosphere).

Timber is mainly susceptible to attack by biological organisms and fire. Other forms of attack, such as chemical and mechanical, are of lesser concern. When timber is placed below groundwater level it is essentially immune from deterioration as there is very little air. However, where water can be oxygenated and in marine environments where borers (animals that eat wood) are present, decay can occur.

While timber has a natural resistance to deterioration due to natural chemicals within it, these chemicals vary between species and depend on the part of the tree used. Chemical processing typically provides more resistance. However, its success depends on the part of the tree (e.g. sapwood or heartwood), the tree species, and the moisture content of the timber. Preservatives can be tar oil based, water based or solvent based. Tar oil and water based products are typically used for in-ground applications. It is noted that EU regulations restrict/forbid the use of many traditional preservatives such as copper–chromium–arsenic (CCA) and creosote (Illston and Domone, 2001).



Figure 99.10 Timber is not a uniform material

99.7.3 Verification

Timber for structural use should be graded into strength classes according to its anticipated performance. Grading is either by visual or machine inspection, and a strength class is assigned to each piece of timber on this basis. In terms of durability, timber is classified according to its life when in contact with the ground. CE marking can be used to guarantee certain characteristics.

Verification of timber on site should start with an inspection of the delivery records and the grade stamps/marks. Samples can be taken for testing to verify various material properties. However, it should be remembered that the size of the sample will affect the measured properties due to the presence of gross features, and that the testing of timber samples usually only informs the values presented in standards.

Grade stamps/marks (an example of which is shown in **Figure 99.11**) should record information such as:

- strength class (C14 to C50 for softwood and D30 to D70 for hardwood – the higher the number the higher the strength);
- species;
- condition (e.g. dry);
- origin (e.g. forest stewardship council (FSC) or programme for the endorsement of forest certification (PEFC) scheme for sustainable sources);
- supplier name;

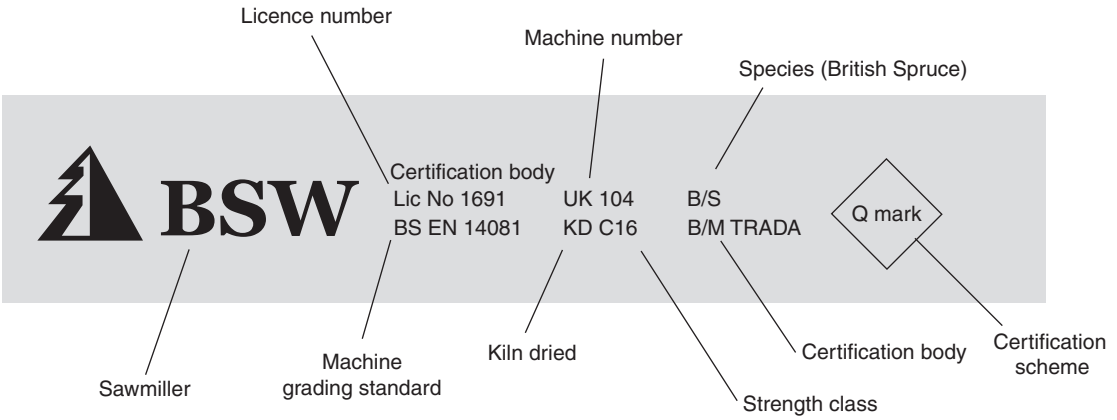


Figure 99.11 Example of a grading stamp
Courtesy of BSW Timber Ltd

- certifying body;
- standard use;
- grading method (i.e. visual or machine).

Dimensions should be verified against construction drawings subject to allowable tolerances, and checks should be made for straightness and imperfections. Timber should be stored in a sheltered location off the ground so as to avoid changes in moisture content, and handled appropriately to avoid damage (see **Figure 99.12**).

Where preservatives are specified, these should be applied under factory conditions and once delivered to site, the integrity and quality of the preservative application should be inspected. If timber is subsequently cut, drilled or poorly handled on site, the integrity of the preservative should be reassessed.

Box 99.3 Case study – timber decay at wharf

The importance of verification with respect to characteristics such as species, heartwood versus sapwood, type and application of preservative is illustrated in **Figure 99.13**. The timber in **Figure 99.13(a)** has not been immersed in water and, apart from some surface discolouration, is essentially fresh. However, just below in the splash zone (**Figure 99.13(b)**) the timber has decayed and crumbles in the hand.

99.8 Geosynthetics

99.8.1 General

Geosynthetics in geotechnical applications can be used to separate materials, reinforce soil (see Chapter 73 *Design of soil reinforced slopes and structures* for further information), permit drainage, provide filtration or act as an impermeable barrier. Geosynthetics can be subdivided into various forms such as geotextiles, geogrids, geomembranes, geocomposites and geonets. **Figure 99.14** illustrates a geogrid and a geocomposite.

Geosynthetics are typically composed of polymers such as polyester, polyethylene, polypropylene and polyamide, and can be formed by processes such as extrusion, spinning, stretching, weaving and bonding. Geosynthetics may be reinforced with fibres or they can be combined with other materials such as bentonite.



Figure 99.12 Storage of timber

99.8.2 Potential problems

During construction, issues for geosynthetics include durability and interface friction. The durability of geosynthetics can be affected on site by acids, alkalis, heat, abrasion, fungus, oxidising agents, UV light and construction plant. While a designer should take account of interface friction and stress–strain behaviour, poor construction practice can adversely affect assumptions made during design.

99.8.3 Verification

For factories supplying geosynthetics, there should be quality controls in place that monitor performance, as detailed in the relevant standards. CE marking can be used to guarantee certain characteristics.

On-site verification of geosynthetics should start with inspection of delivery records. They should be clearly labelled,

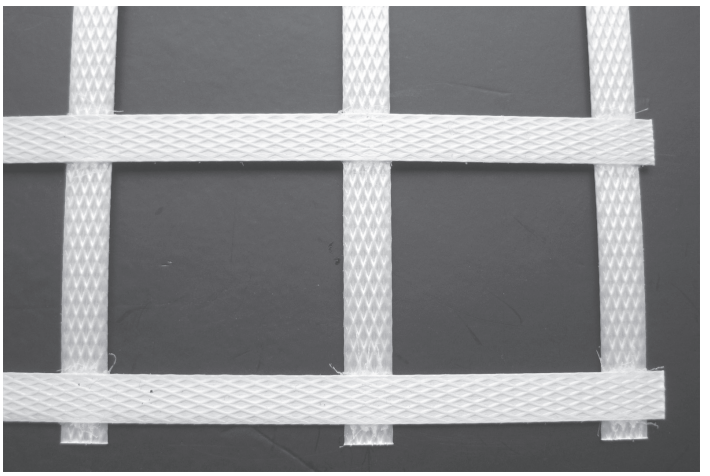


(a)

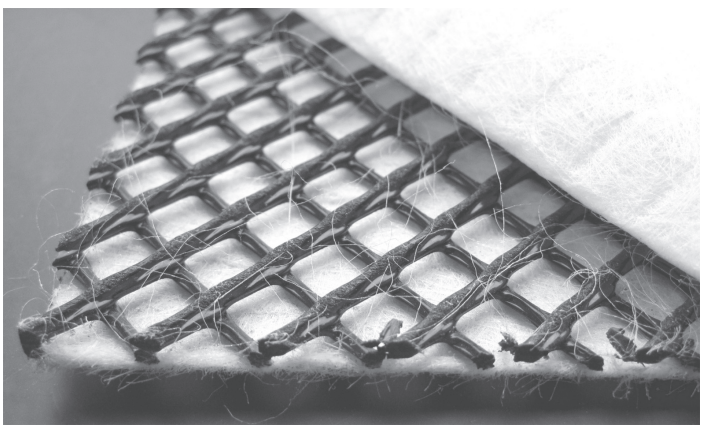


(b)

Figure 99.13 Variability in timber decay (a) above waterline; (b) below waterline



(a)



(b)

Figure 99.14 Example of (a) a geogrid; (b) a geocomposite

stored in the shade and handled with care. Geosynthetic products should be inspected for quality, consistency and integrity. Samples can be taken for off-site testing to verify various material properties, such as tensile strength. However, if tests are undertaken they should be consistent with service conditions.

The surface onto which geosynthetics are to be placed should be appropriately prepared (e.g. level, free of protruding objects) and then inspected to verify that the potential for damage to the geosynthetic to be laid is minimised. While most testing is laboratory based, it may be specified that on-site testing be undertaken to assess the impact of damage from construction plant on a particular geosynthetic being used.

Once a geosynthetic is laid, inspection of lapping or welded joints and flatness (i.e. lack of ripples) should be undertaken, and fill promptly placed to protect it. **Figure 99.15** illustrates the verification of welded joints by pressure testing.

99.9 The ground

99.9.1 General

The ground (soil/rock) is primarily used to support structures in its natural state. However, when excavated, processed, transported or emplaced (i.e. fill/made-ground) it can be used, for example, to raise surface levels, form part of concrete, act as a drainage filter or provide protection against erosion. Its characteristics generally vary according to its origin and composition.



Figure 99.15 Pressure testing of welded joints
Courtesy of Escambia



Figure 99.16 Soil/rock exposure in an excavation

99.9.2 Potential problems

Natural ground can hide many potentially problematic issues and, whilst undisturbed, these issues are generally not of concern. However, once people interact with the ground, these issues often become problems. Soft soils can settle, erosion features may collapse, excess water pressures can cause collapse, aggressive chemicals may attack concrete, sensitive soils are susceptible to landslide, hard rock can cause excavation problems and clay can expand. Man-made ground can hide additional issues such as poor compaction and contaminants.

99.9.3 Verification

The ground can be divided into four groups for the purposes of verification:

- *in situ* ground (existing natural or man-made);
- engineered ground (engineered fill);
- treated ground (improved existing ground);
- excavated material.

Verification of *in situ* ground during construction should be an extension to earlier desk study work and intrusive ground investigation (see Section 4 of this manual for further information on site investigation). It should be used to validate design assumptions, but may be used to fill gaps in knowledge or investigate features previously inaccessible. The simplest form of verification could involve recording strata and groundwater in cuttings, excavations or pile bores (see **Figure 99.16**). Simple portable assessment tools may be used (e.g. a handheld shear vane – see **Figure 99.17**). Existing groundwater installations may be used to monitor water levels. More complex verification could involve intrusive sampling and testing through boreholes, or field experiments such as load testing, pumping trials or plate load tests.

Verification of engineered ground that is emplaced during construction requires additional verification measures – such as density testing, and classification of materials with respect to their ultimate use and compaction characteristics.

Verification of ground that is treated during construction (to improve its geotechnical properties) requires additional verification specific to the properties being improved. The Mackintosh probe (see **Figure 99.18**) is a simple hand tool that can be used relatively quickly to assess improvement in strength over the top few metres of ground.

Verification of excavated material relates to ground that is going to be moved from its source location for use elsewhere. Verification of this type of ground will be specific to its end use and generally involves laboratory testing and visual inspection, for example, maximum dry density testing for clay fill or



Figure 99.17 Shear vane test



Figure 99.18 Mackintosh probe test

durability testing of rock for armourstone would take place in the laboratory; a visual assessment of excavated ground could show it was free of contaminants such as wood or plastic.

99.10 Aggregates

99.10.1 General

Aggregates have a number of geotechnical uses. For example, they can be used in concrete, as drainage filters, as protection against scour, as fill for embankments, in road pavements or in ground improvement (e.g. stone columns) (see Section 7 of this manual for further information on earthworks and pavements). Aggregates range from sand to boulder in size, they can be angular to smooth in shape, and their strength and durability will vary according to their origin. They can be heavyweight such as those sourced from igneous rocks or lightweight such as pumice. Natural aggregates can be collected from locations such as buried channels, riverbeds, river terraces and glacial outwash. However, they are also often produced by excavation and crushing of rock (**Figure 99.19** illustrates the use of a rock crusher). Aggregates can also be man-made, in that they are created by recycling materials such as concrete, asphalt, blast furnace slag or plastic.



Figure 99.19 Crushing plant (Komatsu BR380 JG-1)
All rights reserved

99.10.2 Potential problems

Aggregate problems typically relate to their formation process and constituents. Strength is usually only important in high strength concrete. Of more importance are durability, participation in chemical reactions and physical characteristics.

When some rocks are excavated, exposed to air, mixed with other materials or loaded, they may react to produce undesirable side effects. Potential issues to be addressed by selection and testing include fines content, shell content, resistance to fragmentation, resistance to polishing, resistance to wear, freeze/thaw, drying shrinkage, chloride content, alkali-silica reactivity, carbonate content, shape, size, density, porosity and impurities.

99.10.3 Verification

The production or collection of natural aggregates should be controlled at source. While verification typically starts when aggregates are delivered to site, visits to the proposed source quarry will be beneficial, particularly if suitability has not been addressed during ground investigations.

Once on site, delivery records should be inspected and features such as origin, mass distribution, shape and integrity verified as satisfactory. Samples can be taken for testing to verify material properties such as particle size grading, abrasion value (resistance to wear), magnesium sulphate soundness (resistance to weathering) and 10% fines value (resistance to crushing). An example of aggregate testing is illustrated in **Figure 99.20** which shows the test apparatus used to undertake the Los Angeles abrasion test (steel balls and aggregate are placed in the central drum which is then rotated).

Man-made materials such as recycled concrete may be produced on site. Depending upon their intended use, they should be subjected to similar suites of tests as undertaken for natural aggregates. Recycled material may be required to undergo contaminant testing.



Figure 99.20 Los Angeles abrasion test apparatus
Source: www.pavementinteractive.org

99.11 Grout

99.11.1 General

Grout can be used in geotechnical applications to reduce permeability (e.g. rock grouting), stiffen soils (e.g. compaction grouting), stabilise ground (e.g. permeation grouting), provide support (e.g. jet grouting), compensate for ground movement (e.g. compensation grouting) and fill cavities.

Grouts are commonly cement based, silicate based or resin based and are classified in terms of their particle size as mortars, pastes, suspensions, colloids or solutions (Rawlings *et al.*, 2000). Depending upon the application, they can contain other materials such as cement replacements, clay, sand, fillers, chemicals, admixtures and water. They can be thick and viscous, thin and fluid, weak or very strong.

99.11.2 Potential problems

Problems occur with grouting for a variety of reasons: when the ground being treated is not fully understood; when the grout or grouting technique being used is not appropriate to the situation at hand; when the mix is poorly designed; when control over the procedure is inadequate; or when the grout behaviour is not understood by those using it.

99.11.3 Verification

As grout is often mixed on site, verification of the ingredients and the mixed product is necessary. As there are many possible ingredients, only the mixed product is considered here.

During construction, testing should be undertaken at regular intervals. Of the tests available, typical on-site tests for fresh grout include rheology (viscosity, gel strength, penetrability, thixotropy, workability, bleed) and setting time. For grouts where strength is important, grout cubes made from fresh grout should be prepared. **Figure 99.21** shows a group of freshly prepared grout cubes and a cylinder of grout being used to measure bleed.

Field trials (appropriate to the scale of the project) should be undertaken to demonstrate that the grout is achieving the aims of the designer, especially where there is no prior experience. This may involve intrusive testing or exhumation for verification.

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Figure 99.21 Fresh grout cubes and cylinder bleed test

Box 99.4 Case study – cone test

The cone test is used to provide a viscosity measurement for grout. A common cone used for this test is the marsh cone; however, there are others such as the one presented in BS EN 445:2007. It is important to understand which cone is being specified as different cones may have different volumes and discharge orifice sizes. However, the principles of the test are generally the same and comprise:

- filling the cone with a set volume of grout whilst blocking the discharge orifice (see **Figure 99.22(a)**);
- timing how long it takes a set volume (typically less than was input) of grout to drain from the cone (see **Figure 99.22(b)**);
- using the same procedure, calibrating the cone with water instead of grout.

Post-grouting verification can include laboratory testing of hardened grout samples or field techniques, similar to those used during trials.

99.12 Drilling muds

99.12.1 General

Bentonite and polymer drilling muds are used in the construction of bored piles and diaphragm walls to provide stability to the excavation and assist in the removal of cuttings. Bentonite typically comprises natural or activated sodium bentonite (a type of clay) while polymer comprises long chain molecules.



(a)



(b)

Figure 99.22 Cone test (a) filling of the cone; (b) timing of the discharge duration

99.12.2 Potential problems

Without adequate site control by a specialist contractor, these muds can easily affect the performance of piles or walls in an adverse manner. Problems can include reduced concrete/soil interface friction, reduced reinforcement bond and excavation collapse.

99.12.3 Verification

As drilling muds are often mixed on site, verification of the ingredients and the mixed product is necessary; however, only the mixed product is considered here.

Verification should start with checking that mixing of the constituents with water is thorough and in the correct proportions. The chemical composition of the drilling mud should be noted.

Once in use, testing should take place at various stages throughout excavation and until concrete is placed. Testing should be undertaken on fresh mud and mud that has been cleaned. It should comprise the measurement of density, viscosity, pH and sand content. For bentonite, additional tests for setting time, gel strength and fluid loss/filter cake thickness are necessary. Samples of the drilling mud should be taken from an appropriate point so as to obtain a representative sample – for example, from near the base of a pile bore for sand content prior to concreting. **Figure 99.23** illustrates parts of the sand content test – final washing of the sieve and measuring the sand content. **Figure 99.24** illustrates the apparatus used to determine drilling mud density.

99.13 Miscellaneous materials

99.13.1 Vegetation

Vegetation is often used as a cover for slopes or retaining walls but is often unsuccessful due to lack of control. Use on soil/rock slopes can help to reduce erosion and provide an increased level of stability. However, on steep slopes vegetation is likely to require temporary support until it is established, and in some cases it may need permanent support (see **Figure 99.25**).

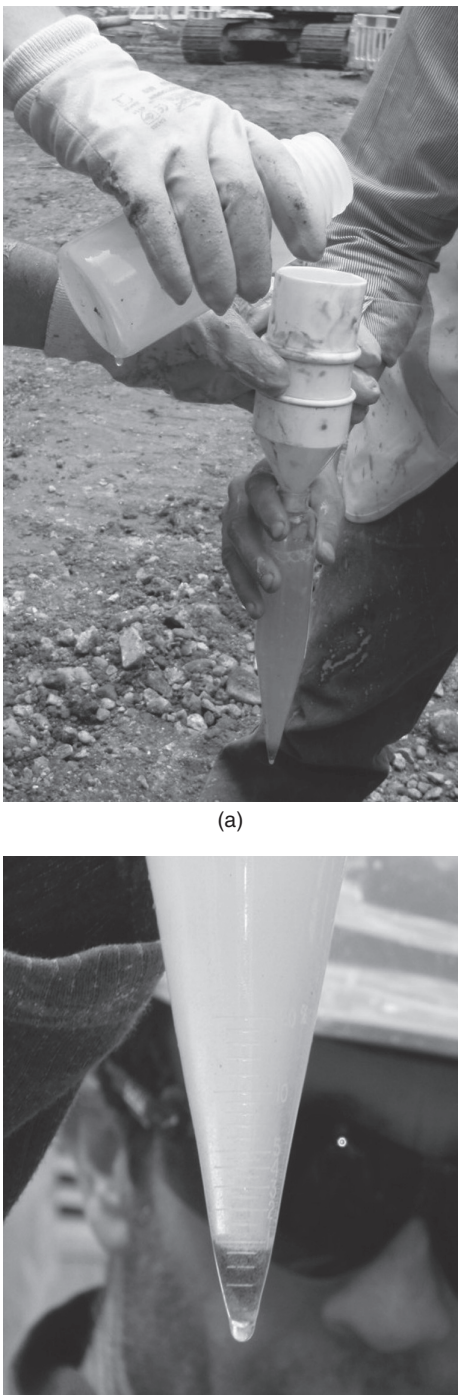
If seeds are being used they should be sampled for testing. If germinated seeds or matured plants are being used, test beds can be set up on site to provide a control. If plants are delivered to site they should be covered while in transport and acclimatised on site. Roots should not be exposed for longer than necessary.

99.13.2 Water

Water is used in many applications on site. Generally, potable water is fine to use without further testing whereas sewage water is not. Non-potable water should be subjected to a preliminary inspection for visual signs of contamination and simple hand-held testing such as pH. Subsequent testing may involve laboratory testing for chemicals. However, the extent of testing should be appropriate to the end use of the water.

99.13.3 Polystyrene blocks

In a similar manner to lightweight aggregates, polystyrene blocks can be used to reduce the impact of embankments on soft soils or earth pressures on retaining walls. Problems that may be encountered during construction include hydrocarbon (e.g. petroleum) attack, heat and UV light damage, although these issues are generally easy to manage with appropriate on-site storage/protection. **Figure 99.26** illustrates the use of polystyrene blocks behind a retaining wall.



(a)

(b)

Figure 99.23 Sand content test (a) washing sand from the sieve into a vial; (b) measurement of the sand content

Testing is generally undertaken prior to delivery to site; however, samples may be collected to determine properties such as density, and compressive and shear strengths. Delivery records should be reviewed, dimensions checked (subject to allowable tolerances) and checks made for imperfections.



Figure 99.24 Density testing of drilling mud



Figure 99.25 Vegetation planted in geosynthetic cells on a steep slope

Photograph courtesy of David B. Andrews, P.E., Atlanta, GA, USA; www.cabeceo.net (personal collection)



Figure 99.26 Polystyrene blocks used as filling behind a retaining wall

Drew Foam Companies, Inc.; all rights reserved

99.14 Re-use of foundations

The re-use of foundations has drawn much attention in recent years due to concerns over future use of sites and conservation of resources.

Assessment of existing foundations for re-use essentially involves three investigation steps:

- desk study (records of construction);
- physical (i.e. material testing); and
- performance (i.e. load testing and historical behaviour).

However, it is important that in this process there is continual appraisal of viability between the design and the assessment components (Butcher *et al*., 2006).

In terms of verification of materials, the physical investigation is most relevant to this chapter. Its aims are to determine the geometry, integrity, strength and durability of the ground and foundation elements.

Investigation of the elements can be intrusive or non-intrusive. Intrusive testing comprises coring to obtaining samples for laboratory testing. Non-intrusive testing comprises external field testing. It is important to understand the limitations of the various non-intrusive tests and their needs in terms of calibration and correlation. Non-intrusive tests include ultrasonic echo testing (to derive the thickness and location of objects – see **Figure 99.27**), radar testing (to locate reinforcement), low strain testing (to derive pile length and integrity), parallel seismic testing (to derive pile length), mise à la masse (to locate reinforcement and the length of steel piles), rebound hammer (to derive concrete strength – see **Figure 99.28**) and cover meters (to derive the thickness of concrete cover).

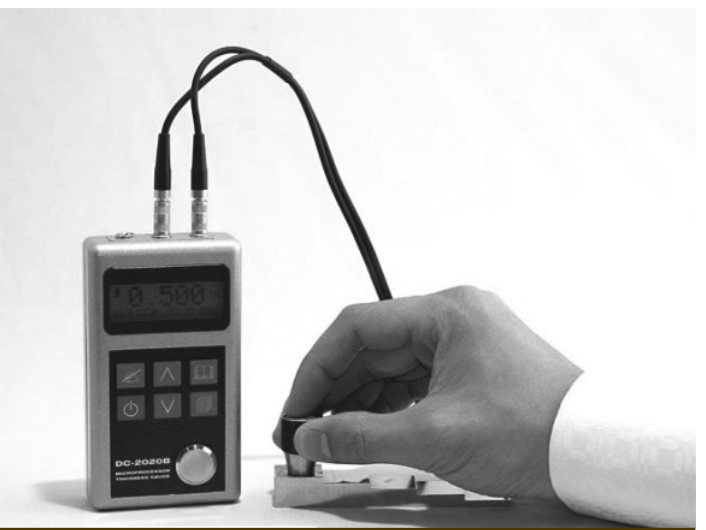


Figure 99.27 Ultrasonic echo tester – used to measure thickness

Ultrasonic Thickness Gauge DC-2020B, DC-2000B series; all rights reserved



Figure 99.28 Rebound hammer – used to measure concrete strength
Ultrasonic Thickness Gauge DC-2020B, DC-2000B series; all rights reserved

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99.15.1 Further reading and useful websites

There are numerous sources of information relating to materials in books, CIRIA guides, British Standards, websites and the like. Some of these documents and websites are listed below. Readers should refer to the bibliographies of these documents and document databases to find further information.

General

BSI – British Standards Institution www.bsigroup.co.uk

CIRIA – Construction Industry Research and Information Association www.ciria.org

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QSRMC – Quality Scheme for Ready-Mixed Concrete www.qsrmc.co.uk

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CARES – Certification Authority for Reinforcing Steels www.ukcares.com

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British Standards Institution (2002). *Mixing Water for Concrete – Specification for Sampling, Testing and Assessing the Suitability of Water, Including Water Recovered from Processes in the Concrete Industry, as Mixing Water for Concrete*. London: BSI, BS EN 1008:2002.

Polystyrene

Sanders, R. L. and Seedhouse, R. L. (1994). *Use of Polystyrene for Embankment Construction*. Wokingham, Berkshire, UK: Transport Research Laboratory, Contractor Report 356.

Re-use

Chapman, T., Anderson, S. and Windle, J. (2007). *Reuse of Foundations*. London: CIRIA, Report C653.

Coventry, S., Woolveridge, C. and Hillier, S. (eds) (1999). *The Reclaimed and Recycled Construction Materials Handbook*. London: CIRIA, Report C513.

It is recommended this chapter is read in conjunction with

- Chapter 93 *Quality assurance*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 100
Observational method

Dinesh Patel Arup, London, UK

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This chapter describes the use of the observational method (OM) on engineering projects. Traditional ground engineering projects are usually based on a single, fully developed, robust design. This can lead to conservative but costly designs. The objective of OM is to achieve greater overall economy by having less conservative designs without compromising safety. Successful implementation of OM relies on a structured approach to both design and construction, an assessment of the most realistic design parameters, comparing these with the cautious design parameters used in traditional designs, having a rigorous monitoring and observation strategy, having a design/construction which in the light of the monitoring data can be modified in a timely way, and having a strong management and reporting structure with well predefined contingency plans in case problems occur. These key ingredients to the successful implementation of OM are described with the help of some case examples. OM can also be used when a project, designed by traditional methods, is in difficulty and the 'best way out' approach is required; this is also discussed. Finally, Eurocode 7 (EN1997-1:2004) (EC7) allows the use of OM in design, but has shortcomings which are also highlighted.

100.1 Introduction

This chapter aims to provide guidance to engineers on the use of the observational method (OM) on engineering projects. It describes the important differences between carrying out designs based on the traditional approach, using 'characteristic values' of parameters (defined in *Eurocode 7: Geotechnical Design, Part 1*: EN1997-1:2004 (EC7)) and those developed using realistic parameters ('most probable') when using OM. It also explains that traditional designs are by their very nature conservative and not flexible to change, but that OM allows the project team to produce an integrated approach to both the design and construction, which can yield significant cost and programme savings. It also fosters good working relationships between all the team members and can be very satisfying, but only if it is implemented properly.

The use of OM is permitted in EC7 but engineers should be aware that there are some shortcomings, which need to be understood before applying EC7. The use of OM reduces the safety margins on design compared with traditional designs, but this can be managed within a rigorous monitoring and observation strategy, implemented within a coordinated team, to still give safe designs. Before applying OM, the management of ground risks and pre-agreed contingency plans (should things not go according to plan) need to be carefully evaluated and explained to the client/stakeholders. Approval for implementation of OM should be agreed with the client in the knowledge that it provides benefits and that there may be some drawbacks (e.g. implementing pre-agreed contingency plans), even if the risks are low. Approval from other third-party checking engineers may also be required before implementing OM.

Whilst the OM approach is intended to provide overall economy, obviously there are higher associated costs for increased instrumentation and monitoring, for implementing a stronger management team to implement the OM, for increased design services and reviewing of the monitoring data. However, on complex projects much of this cost may already be accounted for as instruments and monitoring are required for other reasons, and usually there is already a strong contractor's management team established on site.

The OM approach is not intended to be used where there is a risk-averse client, where there is likely to be a lack of total commitment from any member of the project team, where there is no thorough investigation of the ground and water conditions, and where there is a likelihood of a rapid or brittle mode of failure of the ground (including temporary structural elements).

This chapter guides the engineer through the principles of the use of OM on projects from inception (referred to as the 'ab initio' approach) and also, on projects under construction which for unexpected reasons are running into difficulty (referred to as 'best way out' approach). It relies heavily on the work of some key authors, named below, amongst others:

- *The Observational Method in Ground Engineering*, CIRIA C185 (Nicholson *et al*., 1999);
- Peck (1969);
- Powderman and Nicholson (1996).

These authors provide plenty of examples on the structured implementation of OM on engineering projects. Additional examples from Europe are also given in a recent study carried out by a working party on the use of OM in Europe (GeoTechNet, 2005); this includes seven case examples of the 'structured' use of OM on both building and civil engineering projects and also highlights the overall cost and programme savings made, which the reader might find useful.

100.1.1 Differences between Peck's and CIRIA's approach to OM

The observational method has developed over the last 60 years and was initially applied on a 'trial and error' basis to improve designs. It was not until Peck's Rankine lecture (1969) that an integrated process for predicting, monitoring, reviewing and modifying designs was advocated operating within a framework of OM, and without compromising the safety of the structure.

To successfully implement OM, Peck (1969) identified that it was necessary to have two designs compared with the traditional single design approach in geotechnics. A range of foreseeable conditions needed to be considered, which Peck associated with the most likely condition to happen in practice ('most probable') and the least likely condition to happen ('most unfavourable') (refer to section 100.3.2 for definitions, also illustrated in **Figures 100.4** and **100.5**). He suggested a design starting with the most probable (best estimate) condition and varying the design and/or construction to the planned most unfavourable condition, should observed behaviour be worse than that predicted based on best estimate parameters. Peck's (1969) eight key ingredients for successful use of OM are given in **Table 100.1**.

This approach is fundamentally different to the recent work on OM, published by the Construction Industry Research and Information Association (Nicholson *et al*., 1999). The approach advocated in Nicholson *et al*. (1999) starts with initial moderately conservative parameters (the same as 'characteristic' parameters in EC7), to be relaxed to a likely real situation (i.e. most probable condition) during construction, should the observed behaviour warrant it. This approach is also known as *progressive modification* to the design and was first suggested by Powderham (1994). The setting up of a rigorous traffic-light trigger system (red, amber and green) to deal with uncertainties in the ground during construction was also established after Peck's work. Both these modifications result in improvement to the use of OM on projects, leading to safer designs.

100.1.2 Definition of OM

The best definition of the OM approach is described in CIRIA 185 (Nicholson *et al*., 1999) (see **Box 100.1**). Often engineers mistake monitoring instruments on a project as following an OM approach, and **Box 100.1** shows that there is a proper operational framework for carrying out OM and monitoring is just part of this process.

1	There must be sufficient site investigation
2	Design is developed on most probable (best estimates) to predict behaviour
3	Develop monitoring strategy on calculated values for best case
4	Perform calculations on most unfavourable conditions
5	Identify contingency plans for most unfavourable
6	Monitor and evaluate actual conditions
7	Modify design to suit actual conditions if triggers are exceeded
8	OM can only be done if there is adequate time to make decisions and implement

Table 100.1 Peck's (1969) eight key ingredients for OM

Box 100.1 Definition of OM (CIRIA 185 (1999))

'The Observational Method in ground engineering is a continuous, managed, integrated, process of design, construction control, monitoring and review that enables previously defined modifications to be incorporated during or after construction as appropriate. All these aspects have to be demonstrably robust. The objective is to achieve greater overall economy without compromising safety.'

100.1.3 Comparison between traditional designs and OM design

Traditional ground engineering projects are usually based on a single, fully developed, robust design and there is no intention to vary the design during construction. Instrumentation and monitoring may also be carried out but it plays a very *passive* role, to check original predictions are still valid and provide confidence to third-party checkers, e.g. designers for adjacent building owners affected by a development. In CIRIA (1999) this traditional design is termed 'predefined design'. In comparison, in OM the monitoring plays a very *active* role in both the design and construction, allowing planned modifications to be carried out within an agreed contractual framework that involves all the main parties (client, designer and contractor). The differences in the two design approaches are illustrated in **Table 100.2**.

Peck (1969) defined two OM approaches:

- the 'ab initio' approach, adopted from inception of the project;
- the 'best way out' approach, adopted after the project has commenced and some unexpected event has occurred that is different to the predefined design or failure occurs, and where OM is required to establish a way of getting out of a difficulty.

Predefined design process (traditional design)	The OM process
■ Permanent works	■ Temporary works
■ One set of parameters	■ Two sets of parameters
■ One design/predictions	■ Two designs and predictions
■ Outline of construction method	■ Integrated design and construction methods
■ Contractor's temporary works design/method statement	■ Methods relate to triggers
■ Monitoring checks predictions not exceeded	■ Comprehensive and robust monitoring system
■ If checks are exceeded, consider: (a) best way out approach to design; or (b) redefine the predefined design approach reassessing the geotechnical uncertainties in the ground (see **Table 100.5**)	■ Review and modify process – Contingency plan – Improvement plan
■ Emergency plan	■ Emergency plan

Table 100.2 Comparison of the predefined design process and the observational method

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Case examples presented on the GeoTechNet site give examples of both the 'ab initio' and 'best way out' OM approaches. A paper by Nicholson *et al*. (2006) suggests a structural framework for operating the 'best way out' approach for recovery of deep, multi-stage excavation projects when problems occur during construction, like that which occurred at Nicoll Highway, Singapore (COI, 2005). This operational framework for carrying out the 'best way out' is also described in section 100.5 of this chapter.

100.2 Fundamentals of OM implementation and pros and cons of its use

100.2.1 General

Eurocode 7 does not clearly define the framework to be followed when adopting OM and there are other drawbacks which can be strengthened if use of CIRIA (1999) is also made on OM projects, as described below.

100.2.2 OM drawbacks in Eurocode 7 (2004)

The OM method described in Clause 2.7 of EC7 has been reproduced in **Box 100.2** below to illustrate the main drawbacks, which are as follows:

- It is primarily aimed at the ab initio approach to OM, although it does not exclude the 'best way out' application of OM.
- Whilst it refers to 'acceptable limits of behaviour' it does not define how these may be derived, since EC7's premise for design is based on use of 'characteristic values' which present a lower cautious limit in design, but not the upper limit to represent the most likely behaviour (most probable case), needed to implement the OM approach.
- No trigger limits are defined to establish planned contingency actions to check behaviour.
- There is no operational framework described for management of the OM within a contract, either within national policy or in a project organisation.

Box 100.2 Eurocode 7, clause 2.7 (2004)

Observational method

(1) When prediction of geotechnical behaviour is difficult, it can be appropriate to apply the approach known as 'the observational method', in which the design is reviewed during construction.

(2) The following requirements shall be met before construction is started:

- acceptable limits of behaviour shall be established;
- the range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits;
- a plan of monitoring shall be devised, which will reveal whether the actual behaviour lies within the acceptable limits (the monitoring shall make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully);
- the response time of the instruments and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system;
- a plan of contingency actions shall be devised, which may be adopted if the monitoring reveals behaviour outside acceptable limits.

(3) During construction, the monitoring shall be carried out as planned.

(4) The results of the monitoring shall be assessed at appropriate stages and the planned contingency actions shall be put into operation if the limits of behaviour are exceeded.

(5) Monitoring equipment shall either be replaced or extended if it fails to supply reliable data of appropriate type or in sufficient quantity.

100.2.3 Operational framework for following OM

The operational framework for implementing OM is illustrated in **Figure 100.1**. The OM has to be carried out within the framework of any national and corporate policies governing design codes, specifications, quality management systems and health and safety regulations (e.g. in the UK this is Health and Safety (HSE) Regulations, and Construction Design and Management (CDM) Regulations, 1994). This

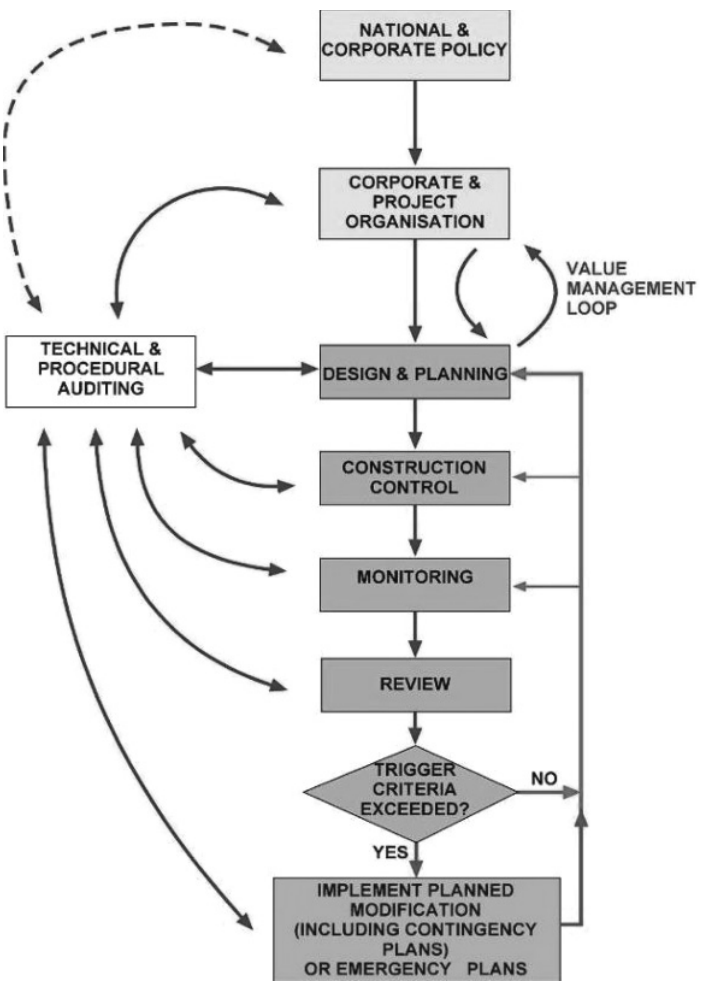


Figure 100.1 The observational method
Modified with permission from CIRIA R185 (Nicholson, Tse and Penny, 1999). www.ciria.org

is represented by the upper box in **Figure 100.1** (see also section 100.3.5 for details).

The second box defines the structure of the key players in the stakeholder's organisation (i.e. the client, designer, contractors, third-party checkers and other inspectors), their roles and responsibilities, and the relationship between organisations and the individuals. This needs to address the culture of each organisation, the level of staff training, experience, openness to communication and management commitment to implementing the OM approach. The stakeholders also need to 'buy into' the technical and commercial risks should any planned contingency or emergency measures need to be implemented, even if this risk is considered low.

Once the OM is agreed at 'project organisation' level, the remaining boxes describe the robust management structure required to implement OM at both design and construction stage and to control the monitoring and reviewing aspects of the observational method when the works are on site. The works have to progress to an agreed plan, with risks being recognised at each construction phase. Daily construction progress has to be under the control of a management structure that ensures any deviation from the method is fully thought through by all members of the project team and covered by an amendment to the plan. A monitoring regime has to be set in place, with competent staff made available to check, review and respond to all monitoring results within a given timescale from when they become available. There then needs to be clear instructions to all involved for all foreseeable situations. Finally, contingency plans need to be in place that can be rapidly implemented should preset 'trigger' limits be breached or any other unforeseen situation develops.

'Auditing', preferably by an independent geotechnical firm, is essential as it checks that the OM designer and project team are following established procedures and reaching the correct technical interpretations. Ideally this should be carried out by a designer who is unconnected with the OM process.

100.2.4 OM management process on site

At site level, there are usually many layers of contracting organisations involved in a project and all the main players need to (a) buy into the OM process and (b) have clearly defined responsibility levels. An example of the interaction between these organisations and the *managed* reporting of the construction and monitoring process is given in a paper by Chapman and Green (2004) for a deep basement project in central London; this structure chart is illustrated in **Figure 100.2**. On this project successful implementation of the OM relied on a clear understanding of the process, roles and responsibilities between the main contractor (HBG), the groundworks contractor (McGee), the concreting contractor (Byrne Bros), the instrumentation contractor (Soil Instruments) and the designer/reviewer (Arup). The management and reporting structure was defined under a 'traffic-light' system of green, amber and red trigger levels, so that all parties were clear of their responsibilities, should pre-agreed contingency or emergency plans need to be implemented (see also section 100.4 of this chapter for details). These plans consisted of increased monitoring, stopping work and implementing additional propping or berms and/or reverting to the predefined design. In the event, the measured movements of the walls during construction were within acceptable limits, thus allowing considerable savings to be made as the alternative design allowed the excavation of double height basements to be sequenced for faster top-down construction.

100.2.5 Pros and cons of OM

OM offers potential savings of time and money and the monitoring provides the needed assurance concerning safety. Some potential benefits of OM are illustrated in **Figure 100.3** and seven detailed case examples of the benefits provided to clients are described in GeoTechNet (2005) (see also CIRIA 185).

However, whilst there are significant advantages associated with the OM application, there are extra costs associated with prescribing a higher level of management and control on site, more instrumentation, preparing for contingency plans and readiness to use back-up plans, e.g. extra propping, and reporting compared with a conventional design situation.

100.3 OM concepts and design

100.3.1 Uncertainty and serviceability

OM is most effective where there is a wide range of uncertainty. **Table 100.3** summarises the types of uncertainty that are often encountered in geotechnical projects.

The OM approach is not suitable where there is a possibility of 'brittle' behaviour in the structure or rapid deterioration in the materials which does not allow sufficient warning to implement any planned modifications (e.g. 'discovery–recovery' contingency plans to be used). Examples of such are rapid deterioration of soils caused by groundwater or non-ductile failures of structural members (struts/walling connections) in multi-propped basements.

100.3.2 Selection of design soil parameters

Where there is a wide range of uncertainty in the soil parameters the OM process in CIRIA 185 uses the terms 'most probable' and 'most unfavourable' to describe the range of soil conditions as illustrated in **Figure 100.4**.

The 'most probable' is a set of parameters that represent the probabilistic mean of all the data, although a degree of engineering judgement must be used in assessing this to take account of the quality of the data. The 'most unfavourable' parameter represents the 0.1% fractile of the data as shown in **Figure 100.4**, and this represents the worst value that the designer believes might occur in practice. The moderately conservative parameter (CIRIA 185) or 'characteristic value' of geotechnical parameters (defined in EC7, clause 2.4.5.2) represents a 'cautious estimate of the value affecting the occurrence of the limit state', and should ideally result in prediction of the upper 5%

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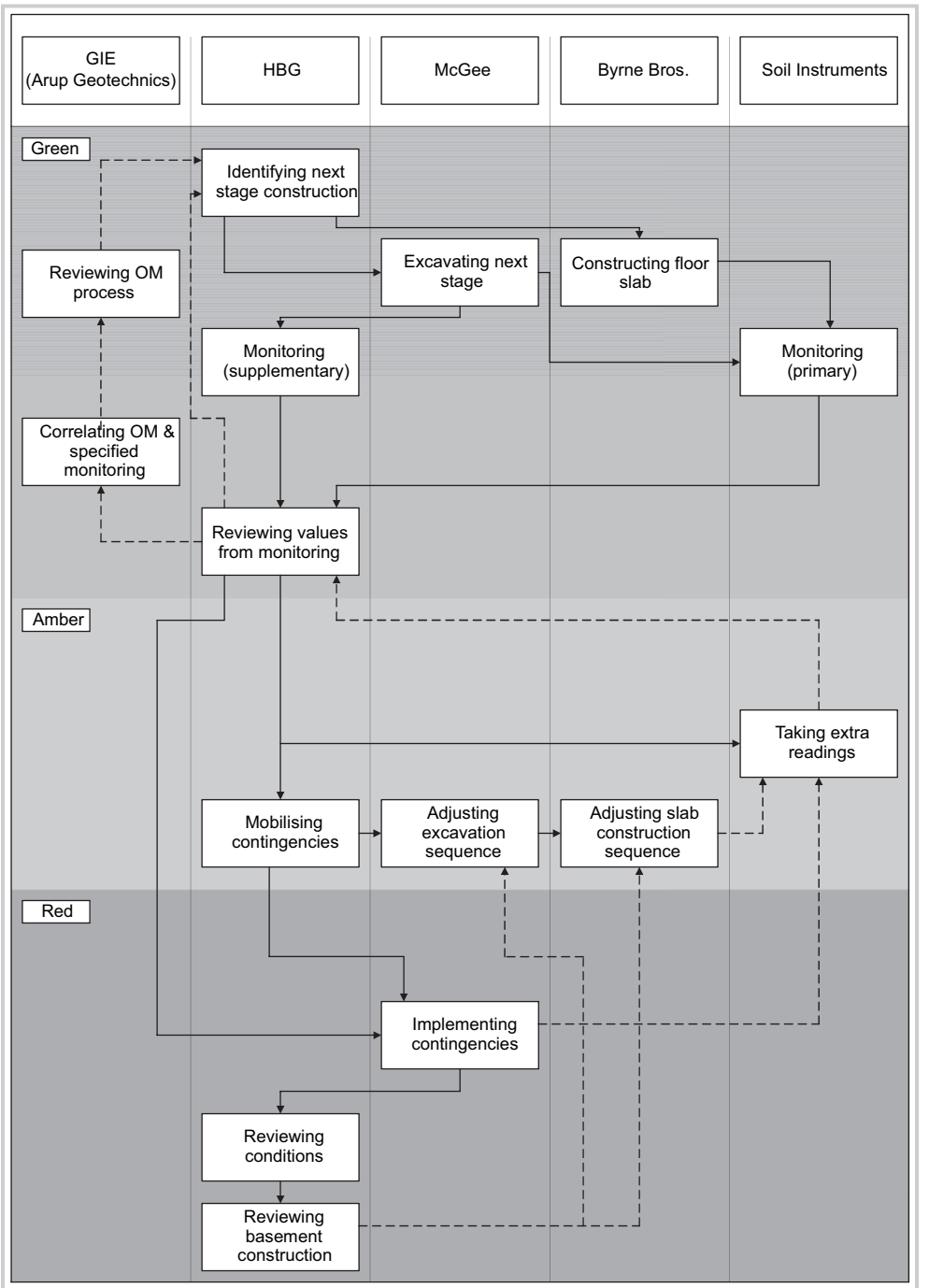


Figure 100.2 OM management and reporting structure on site
Data taken from Chapman and Green (2004)

fractile of the *measured deflections* as shown in **Figure 100.5**. The moderately conservative parameter is therefore not a precisely defined value. It is a *cautious estimate* of a parameter, worse than the probabilistic mean but not as severe as the most unfavourable as shown in **Figure 100.4**. In assessing these parameters the designer should carefully consider the quality of the site investigation data and assess their appropriateness for use in the OM approach. Often the original data may be appropriate for a more robust 'predefined design' approach but may not be of a higher quality for purposes of implementing OM. In this case it may be necessary to carry out further investigations, for instance if there is a lack of groundwater table information.

A typical example of the two sets of OM parameters (most probable and moderately conservative) which were used for a

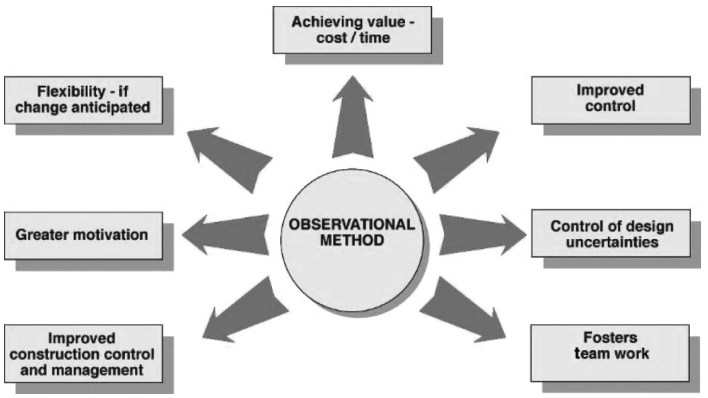


Figure 100.3 Some potential benefits of OM
Modified with permission from CIRIA R185 (Nicholson, Tse and Penny, 1999). www.ciria.org

Geotechnical uncertainty	Example
Geological	Complex geology and hydrogeology
Parameter and modelling	Undrained soil vs drained behaviour
Ground treatment	Grouting, dewatering
Construction	Complex temporary work

Table 100.3 Examples of uncertainty in the ground

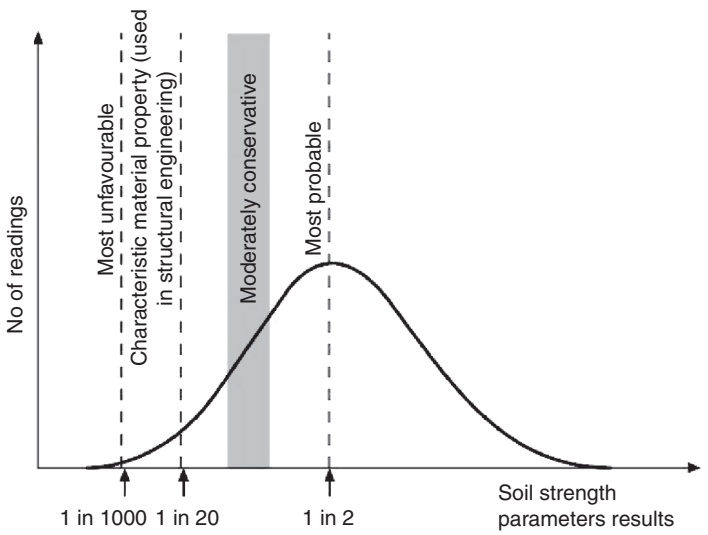


Figure 100.4 Types of soil strength parameters
Modified with permission from CIRIA R185 (Nicholson, Tse and Penny, 1999). www.ciria.org

20-m-deep basement wall design into London Clay is illustrated in **Table 100.4** (Chapman and Green, 2004). When selecting the most probable parameter for use in OM it is important that the designer can justify the choice; for instance this may be proven experience from other projects or back analysis of case studies.

100.3.3 Serviceability and ultimate limit state prediction

When designing to EC7, checks are required to ensure that the following ultimate limit states (ULS) are not exceeded:

- loss of equilibrium of the structure or the ground;
- internal failure or excessive deformations of the structure or structural elements;
- failure or excessive deformations of the ground due to loss of strength;
- loss of equilibrium of the structure or ground from uplift water pressures;
- hydraulic heave, internal erosion and piping of ground caused by hydraulic gradients.

EC7 also require checks on 'serviceability' limit states (SLS); states which are less serious than ULS but which are nevertheless undesirable and would need intervention or repair. In (traditional) predefined designs, calculations are used to check these states use 'characteristic values'.

In OM, the acceptable limit of behaviour is a 'serviceability' calculation, made using both the 'most probable' and 'characteristic' parameters and conditions. These provide the predictions against which the field performance can be monitored and reviewed. Trigger values can be established and contingency plans introduced as illustrated in **Figure 100.5** and **Table 100.5**.

The OM approach illustrated in **Figure 100.5** is for a cantilever wall design but the principle applies to other examples. The green, amber and red zones represent the trigger limits or traffic-light control system used in OM. The precise deflections set for the trigger values will depend on the 'discovery–recovery' contingency plans being used and not simply on the calculated values of predictions made. For instance, if the contingency plans involve a berm in front of the cantilever wall or introduction of a raking prop, then the time taken to implement these measures will influence the setting of these trigger limits. The triggers may then be based on two criteria: the first being wall deflection and the second based on the rate of wall movement to ensure that sufficient time remains to implement the contingency measure. The SLS wall deflection limit is sometimes used as an easily measured proxy for a range of undesirable outcomes, including bending moment failure and buckling of props. It should be noted that OM should not result in excessive movements and that, if limits were breached, the risk of a collapse would still be remote.

In respect to ULS predictions, EC7 identifies three sets of partial factors to apply when assessing the ultimate limit case. These partial factors are applied to 'characteristic values' of the ground but in essence the ULS design values are then similar to the most unfavourable conditions (see **Figure 100.5**). Although, for example in retaining walls, the ULS predictions are used for assessing structural forces, moments and shear,

Parameter	Moderately conservative Conventional design	Most probable (OM)	Reference
Made Ground	$\phi' = 25°$	$\phi' = 38°$	OAP – Broadgate
Taplow Gravel	$\phi' = 36°$	$\phi' = 40°$	Lehane *et al.*
London Clay	$\phi' = 23°$ $c' = 0$	$\phi' = 24°$ $c' = 10$ kPa	OAP, Cross Rail
Lambeth Clay	$\phi' = 23°$ $c' = 0$	$\phi' = 28°$ $c' = 25$ kPa	OAP, Cross Rail
Water Pressure in Taplow Gravel	+7.5 mOD	None	Geotechnical Report
MEFP	applied from GL	None	Observations
Softening	Passive soil	None	Observations – OAP Horseferry Road
Surcharge	20 kPa	20 kPa (Perm) 10 kPa (Temp)	–
Overdig	0.5 m	None	–
Undrained Shear Strength	70 + 7.5z* kPa	+5.5 ≤ d ≥ –11.0 mOD	Results – see **Figure 100.4**
London Clay		112 + 5.19z + kPa –11.0 ≤ d ≥ –17.5 mOD 200 kPa –17.5 ≤ d ≥ –29.2 mOD 400 kPa	
Undrained Shear Strength	300 kPa	400 kPa	Results – see **Figure 100.4**
Lambeth Clay			
Stiffness of Clay (E_q)	1000 c_u	1500 c_u	Back analysis

Table 100.4 Two sets of design parameters used in OM
Reproduced from Chapman and Green (2004)

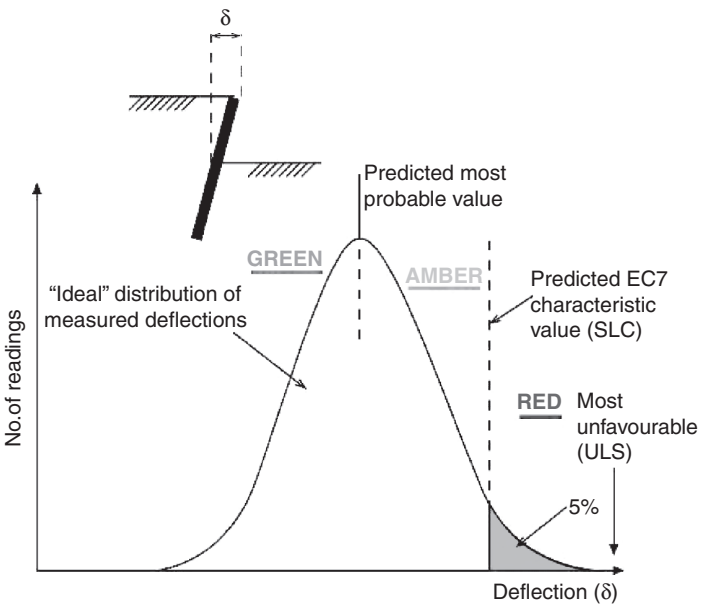


Figure 100.5 Ideal EC7 predicted versus measured performance
Modified with permission from CIRIA R185 (Nicholson, Tse and Penny, 1999). www.ciria.org

Most probable	50% likelihood of movement predictions being exceeded
Characteristic values (EC7) or moderately conservative (CIRIA185)	5% likelihood of movement predictions being exceeded
Most unfavourable (CIRIA 185)	0.1% likelihood of movement predictions being exceeded

Table 100.5 Definitions of most probable, characteristic values and most unfavourable
Data taken from CIRIA R185 (Nicholson, Tse and Penny, 1999)

the ULS deformations of the wall can also be a useful guide to determining the maximum predicted movements in the red zone (**Figure 100.5**). This 'upper limit' of wall deformation (or curvature) can provide a useful input when developing emergency plans of unexpected behaviour in OM and also with the 'best way out' approach in OM, provided the problem has been identified in time to implement a disaster and recovery plan before a ULS condition occurs, and with due regard to safety.

100.3.4 Factors of safety

The design values for ULS (stability) calculations are chosen so that the probability of failure will be acceptably small. It should

be noted that the intent of OM is to take out uncertainties in the ground (see **Box 100.2**), *not reduce factors of safety*, when assessing the ultimate limit state condition in design.

UK standards and those of some other European countries have traditionally applied 'factors of safety' on the design soil parameter, which can vary depending on the type of foundation and type of redistribution of load in the ground (e.g. piled rafts operate differently from single piles per column). EC7, on the other hand, applies partial factors on the characteristic value of the ground parameters. **Figure 100.6** illustrates how the two approaches can produce different factors of safety and how they can vary for different assumed design soil parameters.

100.3.5 Setting OM within the context of a contractual model and safe design

In the UK, there are essentially two forms of main design contracts:

- The client appoints a consulting practice to carry out the permanent design ('engineer design') and the contractor is responsible for carrying out the specified works. In this form of contract the contractor is only responsible for any temporary works design required to complete the permanent works.
- The client appoints a 'design and build' contractor to complete the design based generally on an outline or scheme design performed by a consulting engineering firm.

Other variations to this also occur when a construction manager or project manager is appointed by a client to manage the overall contract. It should be noted that in some European countries the contractual model is very different, responsibilities between parties are less clear, the legal obligations on parties may be severe, and so the application of OM also needs to consider these factors.

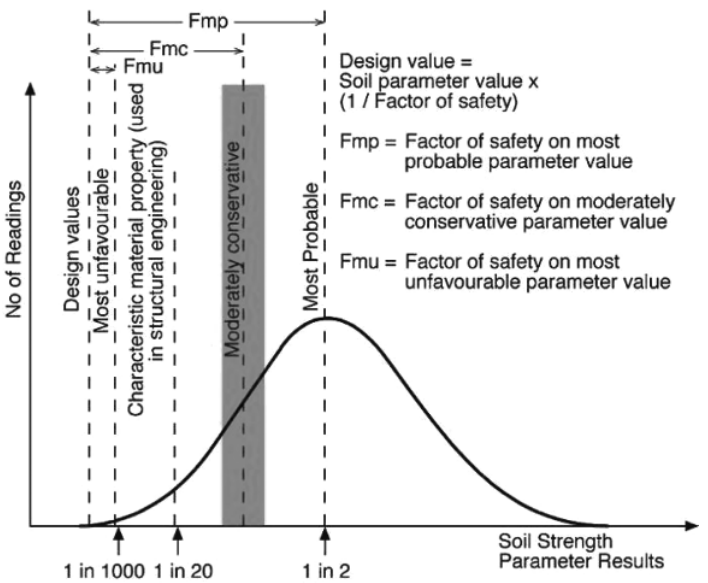


Figure 100.6 Application of factors of safety to different types of design soil parameters
Modified with permission from CIRIA R185 (Nicholson, Tse and Penny, 1999). www.ciria.org

The OM process can be applied to both forms of contract in the UK model described above. In both cases the design product comprises:

- drawings;
- work specifications and bills of quantities;
- calculations.

In addition to this, the UK Construction (Design and Management) (CDM) Regulations 2007 place new duties on the client, designers and contractors to take health and safety into account in both the design and construction of a project. For the designer this means that the design is no longer a set of calculations but must also:

- address buildability issues;
- identify hazards and risks in respect to safety;
- eliminate hazards through good design or, where it is not possible, to reduce the risk to a low level;
- show how this process has evolved in the design by producing a 'risk register';
- address impact on adjacent structures (above or below ground).

These regulations are intended to produce stronger links between the designer and the contractor and minimise risk of failures, via:

- production of 'heath and safety plans' by both the engineer and contractor;
- appointment of a planning supervisor by the client, who vets these plans before and during construction;
- seeking approval from third-party checkers.

Therefore, the CDM Regulations are in line with the OM objective of integrating the design and construction process to produce safe designs and construction practices. From work carried under Work Package 3 of GeoTechNet funded by the European Commission the UK contractual model with appropriate CDM Regulations and risk assessments do not appear to exist in all countries in Europe. Again, when carrying out OM in these countries, the national polices and regulations need to be fully understood and incorporated within the OM process.

On NATM tunnelling projects, HSC (1996) also describes the process of managing risk through the use of a 'discovery–recovery' model before an unacceptable failure scenario is reached, and this approach is entirely aligned with the OM approach.

100.3.6 Rapid deterioration

In certain engineering situations, rapid deterioration can be controlled by modifying the construction sequence as follows:

(1) **Using the multi-stage construction process** – for instance, an example may be an embankment construction over soft

clays. In this situation a rapid deterioration in the factor of safety can be controlled by 'staged' filling, between rest periods, using monitoring to control when the next lift is done (an example of this is given in **Figure 100.7**).

(2) **Using the incremental construction process** – for instance, an example may be in NATM tunnelling work, where the rate of advancing the tunnel face and controlling face loss is a critical component in determining how the ground movements are controlled in the discovery–recovery programme (see **Figure 100.8**) using the traffic-light system described above. This figure shows that the later the problem is discovered, the higher the risk and the longer the structure remains in a state of reduced stability (red zone). Late instigation of decision-making and recovery would also have the same effect. In such instances, trigger limits can be set as both absolute values and/or rate of movements.

The importance of early decision-making to instigate actions for recovery is an important feature of the UK Health and Safety Executive (HSC, 1996) discovery–recovery model, and is a legal requirement for use on all UK construction sites. The use of trigger values described below, an essential feature of OM, can also be used in the 'predefined designs' (see section 100.1.3) to allow sufficient time for implementation of emergency measures when monitoring is being used.

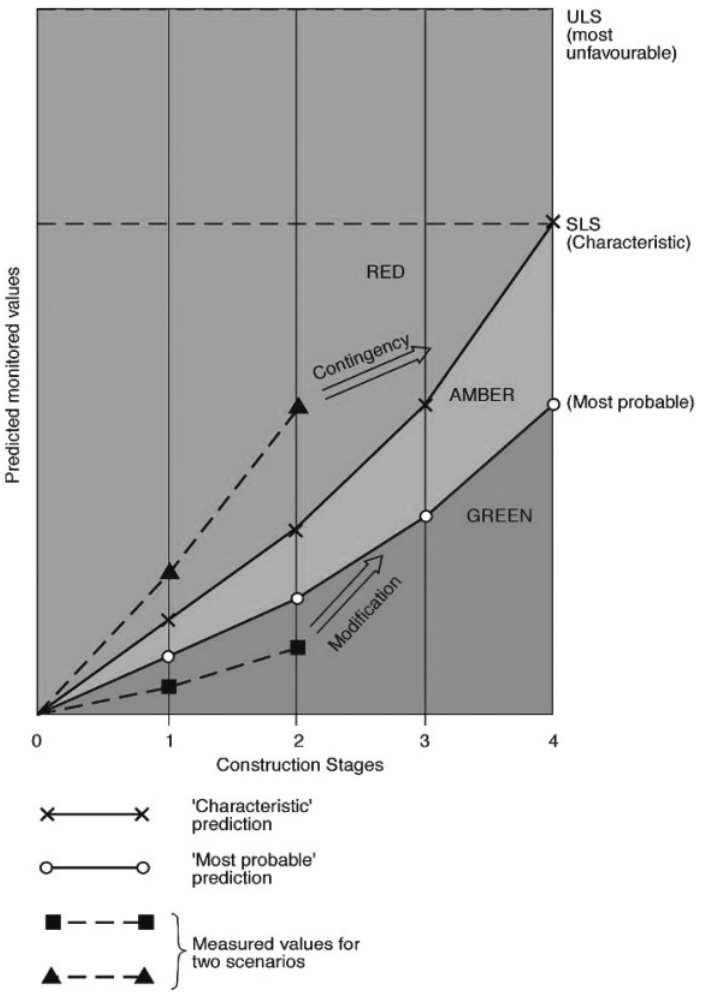


Figure 100.7 Multi-stage construction trigger values
Modified with permission from CIRIA R185 (Nicholson, Tse and Penny, 1999). www.ciria.org

100.4 Implementation of planned modifications during construction

100.4.1 Trigger values

As previously mentioned OM uses a 'traffic-light' system with green, amber and red response zones which allow construction to be controlled, should there be a risk of exceeding the safe green limit, as follows:

- green – continue construction;
- amber – continue with caution and prepare to implement contingency measures, increase rate of monitoring;
- red – stop progress, do everything possible to slow movements, implement contingency measures.

In setting the trigger values the following should be noted:

- The values set may be absolute values or rates of movements or both.
- The trigger limits set should also consider the accuracy of the instrument and whether it is practically measurable.
- The choice of instruments to measure movements should therefore be appropriate for the project and not based simply on lowest cost.

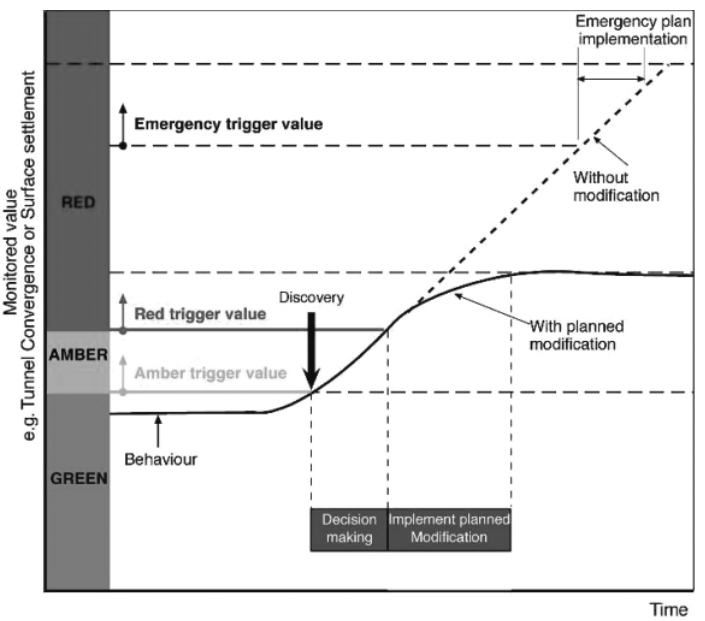


Figure 100.8 Traffic-light system for an incremental excavation (tunnels) process
Modified with permission from CIRIA R185 (Nicholson, Tse and Penny, 1999). www.ciria.org

- The amber/green limits should be sensibly set based on the design so that the likelihood of breaching this limit is small (e.g. values may occasionally stray outside the green zone) but also treated seriously if it is breached.
- The contingency/implementation plans should be taken seriously if the trigger limits are being breached.
- The choice of the instrumentation should be treated seriously and the monitoring contractor should be experienced in this type of work.

The trigger limits should be linked to the 'most probable' (green/amber limit) and the 'characteristic values' (amber/red limit) for implementation of the planned modifications in OM, as illustrated in **Figures 100.5**, **100.7** and **100.8**. A further example, for use with cantilever retaining wall movements is illustrated in **Figure 100.9**. It can be seen that in this example the measured movements were also well below the most probable conditions.

100.4.2 Monitoring systems

Monitoring systems will vary depending on the type of construction project in which OM is implemented. It is also very important to define both 'primary' and 'secondary' monitoring systems in OM. For example in multi-prop deep basements, the primary system (e.g. inclinometers and wall-mounted settlement gauges on neighbouring party wall structures) may be the main instruments relied upon to allow implementation of any contingency measures in OM, whilst the secondary system (e.g. 3D targets at top of walls or levelling surrounding ground) might be a more frequent and fast monitoring system to quickly assess the progress of the excavation works and aid a broader understanding of the pattern of ground movements on a site. When considering the amount of instrumentation for use on a project, considerations should be made in respect to whether to monitor using remote methods (can be expensive) or by manual methods (labour-intensive) and if the latter, how long it will take to carry out a round of monitoring in a single day. This will then allow assessments of the most important instruments to be monitored to be made as part of the overall OM strategy.

An essential part of OM is that the primary system needs to be immediately repaired if damaged on site, to ensure that OM can be continued. Chapman and Green (2004) explain the use of primary and secondary instrumentation for a deep basement in London, in the context of the trigger limits and the process owner (see **Figure 100.2**).

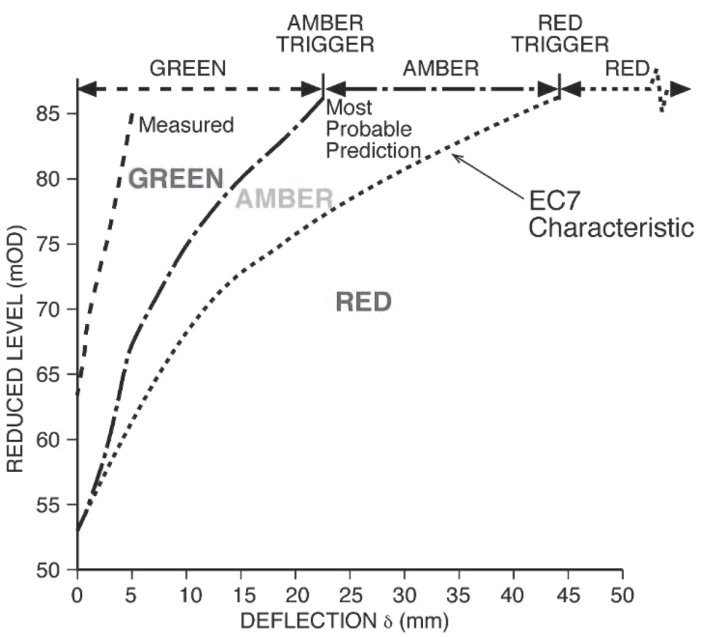


Figure 100.9 Example of trigger limits in retaining walls
Modified with permission from CIRIA R185 (Nicholson, Tse and Penny, 1999). www.ciria.org

100.4.3 OM quality plans

It is essential to have a quality plan before implementing OM on a site. This plan would present the designer's movement predictions based on the defined construction sequence. Each stage of construction sequence should also show the acceptable limits of predicted behaviour using the traffic-light system described above.

Simple graphical outputs that the whole project team can understand are essential and an example of this is illustrated in **Figure 100.10** for a 20-m-deep basement, excavated in a top-down manner, in London. This quality plan shows the predicted green and red limits for wall deflection at each dig stage, for a planned double height excavation technique, to allow floors to be built quickly. The actual movements recorded from inclinometers can then be plotted for comparison with predicted behaviour. These graphs allow the OM reviewer to make informed judgements and the whole project team are brought into the decision-making process: whether to continue to the next dig level, or to implement contingency measures before continuing. For the example illustrated, contingency measures involved additional propping, use of natural berms in front of the wall before proceeding to the next level, and changing the construction sequence to the original 'predefined design' which involved progressing excavation/floors one after another.

100.4.4 Construction control

Successful construction control is a vital part of OM; the main process is as follows.

- A construction control proforma is used to record all details of construction operations, strengths of materials exposed during staged excavations, the fabric and structure of exposed materials and the deterioration of surfaces exposed to water (see example of cutting **Figure 100.11**).
- This control has to be fully integrated within the project team, simple to use and the data easy to read; graphical outputs are essential for informed decisions to be made (see **Figure 100.10**).

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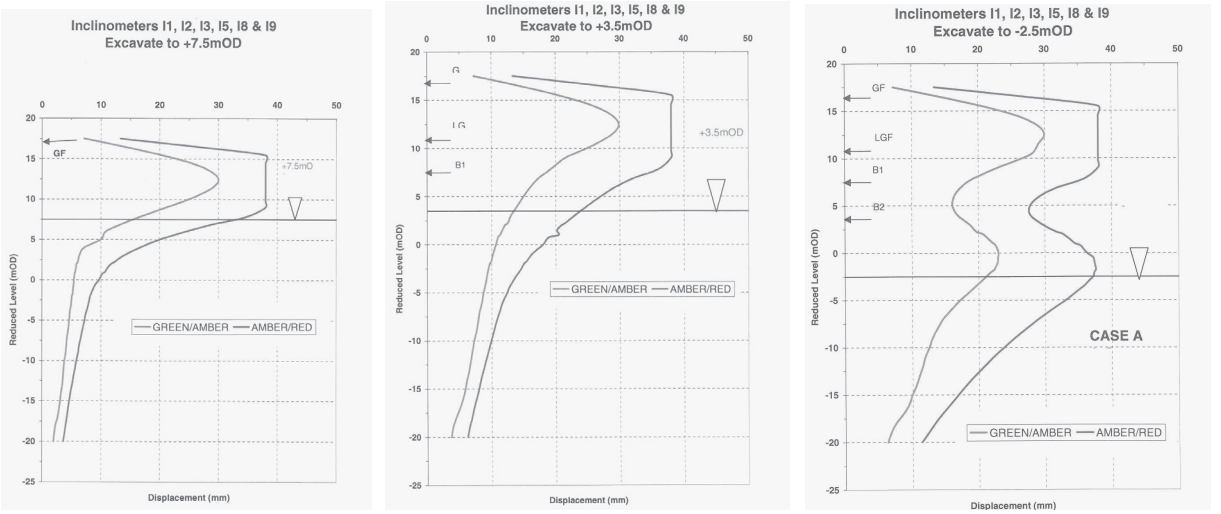


Figure 100.10 Typical quality plan developed for a phased top-down basement excavation
Reproduced from Nicholson, Dew and Grose (2006)

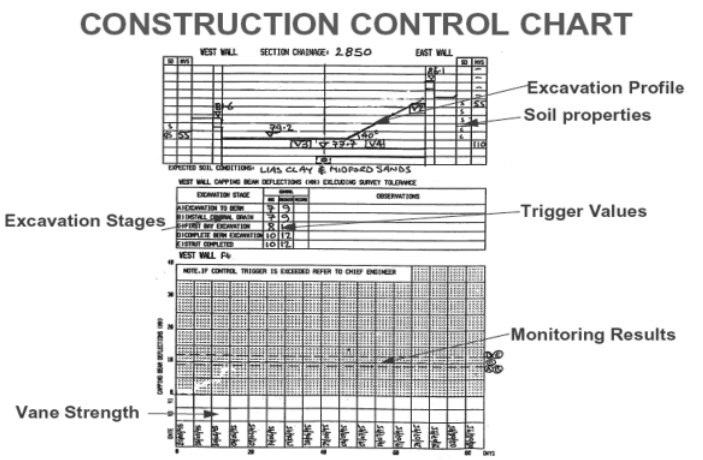


Figure 100.11 Example of construction control proforma sheets on site prepared by a contractor
Reproduced from Nicholson, Dew and Grose (2006)

- Each process has to have a process owner, with certain levels of responsibilities and implementing of actions, as illustrated in **Figure 100.2** and described in Chapman and Green (2004). Where there are many trade sub-contractors involved in a project, the key organisation for making the decision for reviewing the values from the site monitoring and implementing planned contingencies is often the main contractor, who has greater control and powers to immediately stop works on site.

100.5 'Best way out' approach in OM

The 'best way out' approach is used when monitoring checks in a predefined design (**Table 100.1**) exceeds, for unexpected reasons, predicted values but before an emergency condition is reached. Peck (1969) gives some good examples of the 'best way out' approach in which OM is adopted in response to some unexpected events or failure to establish a way out of a difficulty. Nicholson *et al*. (2006) present a more structured approach for recovery of deep, multi-stage excavation projects when problems occur during excavation. These authors studied the post-inquiry reports following the collapse of Nicoll Highway (COI, 2005), and to avoid such failures occurring again suggested a 'best way out' solution which is described below.

In the event of a 'discovery', the 'best way out' approach would trigger an '*initial recovery decision-making*' stage as shown in **Figure 100.12**.

In all cases this will result in stopping work and/or implementing emergency planned measures to secure the safety of site staff and the general public while the unexpected event is fully investigated. This assessment will inevitably be somewhat qualitative rather than quantitative as decisions need to be made rapidly at this initial stage.

Once the safety of the site has been secured, the project team can then turn their attention to recovery of the project back to a fully stable condition which means first carrying out a 'design review process' of the unexpected event, by back analysis of the actual conditions and comparison with the original design. This process can be broken down into four processes (termed 'RADO') as shown in **Figure 100.12**. These processes are briefly described in section 100.5.1 below.

Following this design review, the project team can then consider the following two stages:

- whether to initiate OM 'best way out', in which case this approach would follow the OM framework illustrated in **Figure 100.1** and described in this chapter; or
- whether to make a complete re-design based on a traditional, but revised, predefined design.

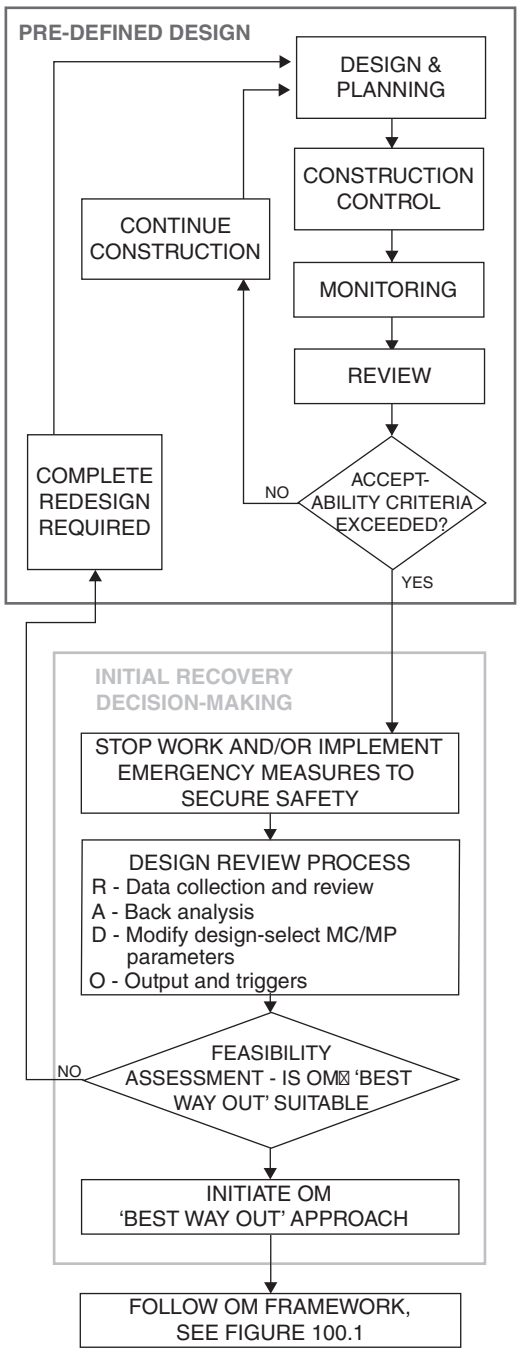


Figure 100.12 'Best way out' operational framework
Reproduced from Nicholson, Dew and Grose (2006)

100.5.1 Four processes of design review (RADO)

100.5.1.1 Process R: data collection and review

This process involves collecting all available data to define the behaviour of the structure for use in the back analysis. Particular emphasis should be placed on understanding the actual conditions and behaviour operating in the field, rather than justifying the original design assumptions. Sources of data should include: soil data and stratigraphy; construction records; actual sequence of events to inform back analysis process; and observations and physical measurements leading up to the unexpected event.

100.5.1.2 Process A: back analysis

The purpose of this process is to refine the designer's understanding of the actual behaviour of the structure and reduce uncertainty in the design. The process involves: establishing most probable parameters; developing a satisfactory model using most probable parameters; comparing results with monitoring data and field observations; reviewing/revising parameters if good agreement is not achieved; and once a reliable model has been produced, proceeding to design.

100.5.1.3 Process D: verify modified design

This process involves predicting the future behaviour using the realistic model and set of parameters developed from back analysis, for the remaining construction stages, but adopting a level of conservatism into the model. The structure behaviour should adopt moderately conservative (characteristic) parameters for the serviceability design and 'worst credible' or factored parameters for stability checks (see sections 100.3.3 and 100.3.4)

100.5.1.4 Process O: Output plans and triggers

If the OM 'best way out' is to be used then the process of OM as described in this chapter has to be agreed with all stakeholders in the project, with appropriate contingency and monitoring plans and setting up of trigger values and management teams.

100.6 Concluding remarks

This chapter provides an overview of the use of OM in engineering projects. It provides a structural framework for carrying out OM for both the 'ab initio' (projects from initial conception) and 'best way out' approaches (traditionally designed projects running into difficulty). An extensive guide to OM is the UK CIRIA 185 (1999), and this was consulted widely in preparing this chapter. The reader is also asked to consult this document should OM be considered, as it sets out a proper framework for operating the observational method and is more rigorous than currently described in Eurocode 7. Other case examples quoted in this chapter also provide an excellent history of the use of OM on more recent projects since CIRIA 185.

100.7 References

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100.7.1 Useful websites

GeoTechNet (European Geotechnical Thematic Network): www.geotechnet.org

Health and Safety Executive (HSE): www.hse.gov.uk

It is recommended this chapter is read in conjunction with

- Chapter 78 *Procurement and specification*
- Chapter 79 *Sequencing of geotechnical works*
- Chapter 94 *Principles of geotechnical monitoring*
- Chapter 96 *Technical supervision of site works*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

Chapter 101

Close-out reports

Roger Lindsay Atkins, Epsom, UK
Martin Kemp Atkins, Epsom, UK

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The geotechnical close-out report is intended to describe, collate and summarise the geotechnical works that occurred during the construction phase of the works together with any post-construction monitoring. The report forms an essential part of the construction verification process and it provides information relevant for maintenance, demolition or re-use purposes by future owners of the asset. The close-out report should represent the final step in the geotechnical reporting cycle.

101.1 Introduction

This chapter describes the relevance and importance of producing a close-out report. Although it is often a contractual requirement of major civil infrastructure projects the close-out report rarely attracts the right level of attention to do it justice. Typically this report is produced in a rush, after the event, without the continuity of staff and during a period when the emphasis is on staff leaving site and demobilisation of the design team.

The importance of the close-out report during its production is not always immediately apparent to the author, as it is not until some future event (usually involving others) that the recorded information and specific details are required. In recognition of this, the report should be prepared on the basis that it forms an essential and valuable part of the design and construction process rather than a contractual compliance.

The geotechnical close-out report is the culmination of the geotechnical reporting cycle which includes preliminary studies and geotechnical reporting of the factual, interpretative and design data (see Chapter 50 *Geotechnical reporting* and Eurocode 7). The report may also be known as a feedback, completion, verification or validation report.

101.2 Reasons for writing close-out reports

It is good practice to record and document the geotechnical elements of the construction phase of a project as part of the construction verification in a close-out report. In some instances, as described below, it is a requirement to produce one. The report is an essential component of assessing re-use of the geotechnical elements in future projects (see BRE EP73, BRE EP75 and CIRIA C653). For example, faced with 100 30-year-old piles, there can be much greater confidence if there is a final as-built plan and knowledge that only piles numbered 23 and 76 had non-conformance reports (NCR) and that they were satisfactorily resolved, together with full details of installation and testing.

The close-out report provides a means of demonstrating the verification and implementation of the geotechnical design (from the geotechnical reporting phase) and records any changes to the design during the construction phase. As-built works frequently differ from those shown on the 'design' or 'for construction' drawings. **Figure 101.1** shows a chalk slope cut to the 'for construction' requirement, while **Figure 101.2** shows the additional remedial works added during site works to maintain stability of the infill deposits prior to topsoiling the slope.

A key element of the report is feedback on constructability issues which can then lead to more cost-effective buildable designs. This is an important feature for infrastructure owners, such as the UK Highways Agency, where feedback can be incorporated into future improvements to their standard specifications. An example would be reporting on how earthworks material behaved when placed and compacted which was just outside the specified grading envelope and whether the standard compaction criterion had to be altered to achieve a compliant compaction.



Figure 101.1 Cut slope in chalk formed to the 'for construction' slope angle



Figure 101.2 Cut slope to be recorded on the 'as built' drawing after remedial works



Figure 101.3 Historical foundations and well encountered during site clearance

The close-out report also provides information on the geotechnical works for ongoing maintenance, demolition, re-use and on rare occasions for forensic investigations when problems had occurred, such as excessive foundation settlement to the works.

Projects designed under the Eurocodes require the production of a Geotechnical Design Report (see Chapter 50 *Geotechnical reporting*). This design report, which is detailed in Eurocode 7 (EN1997-1:2004) as a Principle, has a requirement to provide a 'plan of supervision and monitoring' for the works. This plan identifies items to be appropriately checked and monitored both during and after construction of the works. When the checks and monitoring are completed Eurocode 7 requires the checks and monitoring results to be recorded as an 'addendum to the [Geotechnical Design] Report'. The close-out report can be used as a means of recording and providing the 'addendum' to the design report.

Box 101.1 Illustration of the benefit of a close-out report

The designed foundation solution for a 10-storey development in central London was for pile groups supporting the column loadings. During construction the following were noted by the supervision team: (a) some of the installed continuous flight auger (cfa) pile depths were shortened due to the reassessment of pile capacities using the results from site pile tests; (b) the installed pile reinforcement was of varying depth (10 m design length, 8 to 10 m installed lengths) due to difficulties in pushing the cage into the fresh concrete; and (c) the presence of historical mass concrete footings and a well (see **Figure 101.3**), which meant that a number of pile cap arrangements were re-designed to avoid the footing. Recording all these variations in the design in a close-out report means that in the future these piles could be assessed for re-use in a future redevelopment. Additionally, the report can provide information on the piles as an obstruction to tunnelling projects, or the foundation information can be used to assess the effects that ground movement from a tunnelling project would have on the piles.

An important justification for producing a close-out report is in satisfying the requirements of the UK CDM 2007 regulations. When these regulations apply to a project, there is a requirement for a 'Health and Safety File' (see Chapter 42 *Roles and responsibilities*) to be produced on completion of the project. This file is intended to provide information to future users, designers and contractors. A close-out report can be used as the mechanism for recording the relevant information for inclusion into the file. It should be noted that other parties to the construction works also input to the 'Health and Safety File' and that agreement should be reached between these parties as to the scope of their individual input. This agreement reduces the potential of either overlap and conflicting information or gaps in the information supplied.

The Highways Agency, one of the main UK infrastructure owners, has recognised the importance of recording the construction activities on its projects and as part of its design standards has a requirement for a feedback report which is described in HD 22/08 *Managing Geotechnical Risk* (2008). The piling industry has also understood the need for recording its on-site construction activities and has included the requirement for a 'Completion Report' in the widely used industry standard piling specification 'ICE specification for piling and embedded retaining walls' (2007).

The 'Reuse of Foundations for Urban Sites' (RuFUS) international conference held at the Building Research Establishment in 2006, highlighted the problem of verification of existing foundations from available information for re-use in new works. The proceedings (BRE EP73) and its associated best practice handbook (BRE EP75) provide useful background information of what information is required for assessing the re-use of foundations. This was further augmented in 2007 by CIRIA C653 *Reuse of Foundations* (Chapman *et al*., 2007) which provided greater detail of the information required for assessing the re-use of piled foundations, but the principles described could be applied to other foundation solutions. Important issues relating to insurance and liability of foundation re-use

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are described in the paper by Chapman *et al*. (2001) and in CIRIA C653.

The AGS (2005) report *AGS Guidelines for the Preparation of the Ground Report* includes reasons for producing a close-out report of which the main points have been summarised in this chapter.

101.2.1 Key learning points

- A close-out report can demonstrate the verification and implementation of the geotechnical design in accordance with the design specification.
- The report brings together and records design changes and how they were implemented during the construction phase.
- The report provides useful information for the future re-use of the works.

101.3 Contents of close-out reports

The close-out report should be a stand-alone report that may cross-reference the design reports or the design specifications (including drawings). It does not completely replace them but should add to the geotechnical knowledge bank of the works. Therefore, the report should describe the site, the geotechnical works, verification and monitoring data in succinct text and be supported by material such as drawings, figures and photographs (see **Figure 101.4**). The source material should accompany the report, either as appendices to the report or as stand-alone reports, to which cross-reference is made. 'As-built' drawings are also an essential component of a close-out report detailing the actual constructed works. Photographs, properly referenced (e.g. with date, orientation, subject) also provide a good record of the works on site. A key element of the report is the inclusion of the site documentation, construction and supervision records (see Chapter 96 *Technical supervision of site works*).

The close-out report is not intended to be verification/validation of any geoenvironmental works carried out as part of the construction works. Geoenvironmental works include works such as testing of samples for contamination and remediation of contaminated ground. However, salient factors from the geoenvironmental works, such as contamination barriers, should be included in the close-out report. A separate report dealing with the verification/validation of any geoenvironmental works is generally required for issuing to the relevant authorities.

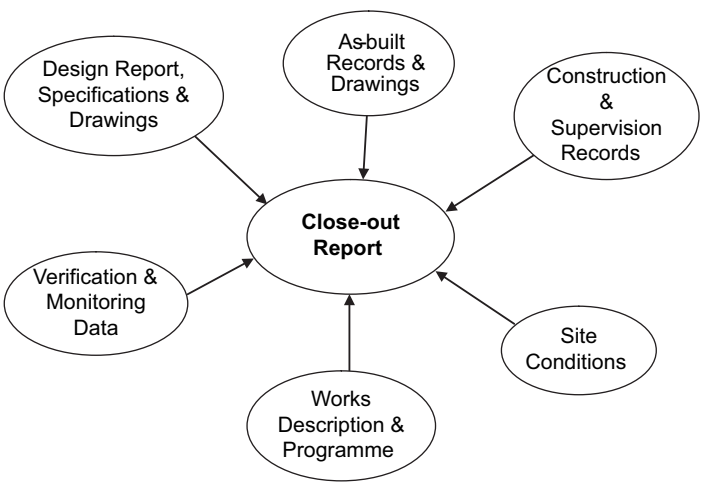


Figure 101.4 Major inputs to close-out reports

The majority of projects will generate large quantities of data that can be difficult to present or understand in the context of the overall project. The close-out report should present this information clearly within a logical framework. Summary data can often be presented more effectively in graphical format such as time series graphs or as spatial plots (**Figure 101.5**). In common with all reporting the presentation and clarity is important. Guidance on good practice in writing ground reports is given by the AGS (2007) *Guide to Good Practice in Writing Ground Reports*.

As a minimum the contents of a close-out report should be tailored to the size and complexity of the works and for the geotechnical works include details of the following:

- project organisational structure involved (e.g. client, consultant, main works contractor, groundworks contractor);
- description of the works (main and geotechnical) and other relevant works (e.g. geo-environmental remediation);
- works programme;
- works location related to a nationally recognised survey grid;
- description of the site background and original site conditions;
- summary of the relevant site investigation reports (including design reports);
- detail of additional site investigation works;
- site documentation, construction and supervision records;
- description of the ground and groundwater conditions encountered;
- geotechnical and compliance testing (including calibration certificates);
- monitoring of the works and its effects on adjacent buildings, structures and utilities;
- temporary works (including as-built details if significant);
- as-built records including drawings, schedules and registers (e.g. piles, soil nails, etc.);
- non-conformance reports or similar together with remedial actions;
- site and progress photographs;
- environmental monitoring (e.g. vibration from piling or plant movements);
- details of communications with regulators (e.g. the UK Environment Agency);
- post-works monitoring arrangements and maintenance requirements;
- critical requirements for decommissioning of the works.

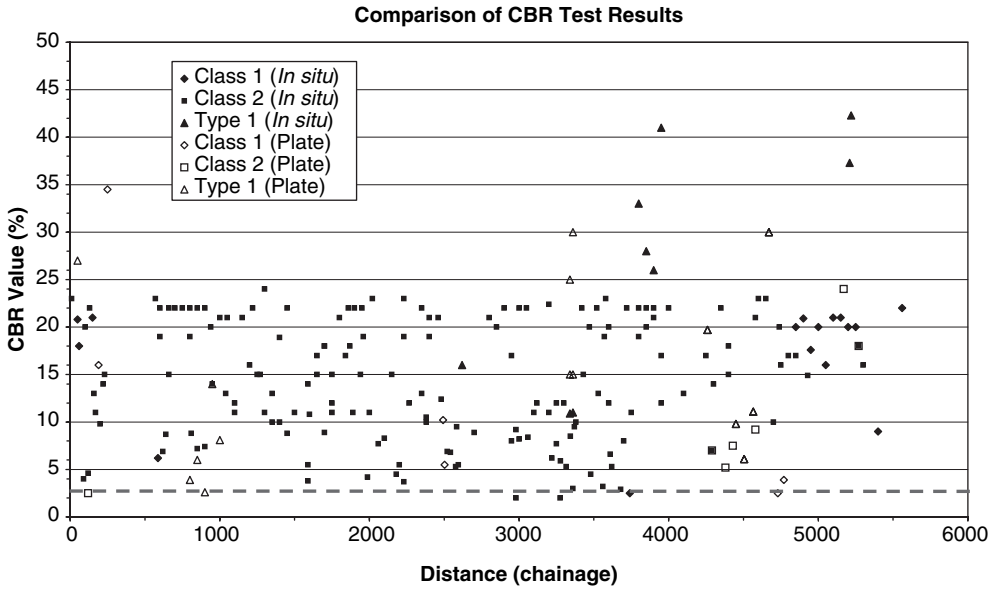


Figure 101.5 Example of graphical presentation of data

The UK Highways Agency, in their design standard HD 22/08, provides a detailed format for their feedback report. This format and content listing could form the starting point for close-out reports for other infrastructure-based projects. Further advice on the preparation and content of a close-out report has been published by the AGS (2005).

101.3.1 Key learning points

- Close-out report is to succinctly describe the site, the geotechnical works, verification and monitoring data and be supported by drawings, figures and photographs.
- The report should clearly record any changes made on site from that detailed in the geotechnical design report, specifications and construction drawings.

101.4 Reporting on quality issues

The quality system in place for the construction of the main works and geotechnical works should be described in the close-out report. The quality assurance strategy for the supervision of the works should also be described in the report. Relevant records relating to quality issues for the geotechnical works should be included in the close-out report together with a summary of the key quality issues and their resolution.

NCRs or similar are to be expected on geotechnical works due mainly to the variability of the ground (conditions and response) and the limitation on the extent that the ground can be assessed by the ground investigations (see Chapter 44 *Planning, procurement and management*). The resolution of NCRs should be described in the close-out report together with the remedial action taken and its effects on the design, construction and use of the geotechnical works. The fact that an NCR has been raised often means that there has been a variation in the construction of the geotechnical works and this would require additional design considerations. Therefore, the close-out report needs to show that the NCR has been rationally considered and the appropriate action taken to close out the NCR.

101.4.1 Key learning points

- Inclusion of quality-related records aids the verification process of the works.
- Reporting on the resolution of construction-related non-conformances is a key part in proving the quality of the works.

101.5 Reporting on health and safety issues

The designers of the geotechnical works will have identified the key health and safety issues and risks relating to their design. The preferable method of communicating these issues and risk is for them to be shown on the 'for construction' drawings typically as notes. These issues/risks should relate to the designer's significant assumptions on construction, maintenance and decommissioning. On completion of the construction activities the designer's significant assumptions on maintenance and decommissioning should be included on the 'as-built' drawings and summarised in the close-out report.

At the construction stage where a significant issue has been detailed by the designer, such as a suggested construction sequence, the contractor may adopt this sequence or develop their own alternative approach. This may significantly alter the original designer's assumptions relating to the maintenance and decommissioning of the works. Therefore these issues or risks would need reassessment by the contractor and/or designer before being included on the 'as-built' drawings.

The majority of geotechnical works once constructed are buried and not readily accessible for inspection/survey and

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therefore accurate and detailed recording of exposed features and their treatment during the construction works are necessary. The following are examples of features that could have health and safety issues relating to maintenance, decommissioning or re-use for inclusion in the close-out report:

- observations on founding stratum, e.g. replacement areas of poor founding material;
- groundwater in excavations, e.g. amount, quality and from which stratum;
- areas where contaminated ground was encountered;
- historic man-made obstructions, e.g. foundations, archaeological remains, wells;
- newly created man-made obstructions, e.g. test piles, sheet piles left in place;
- utility services, especially on private land;
- temporary works, e.g. sheet piles, soil nails, extent of excavations/slopes, dewatering;
- areas of slope/excavation failures;
- details of adjacent structure foundations;
- areas of in-ground testing, e.g. piles, ground treatment, soil nails;
- locations of basements and voids.

Further information is described in the CIRIA SP136 *Site Guide to Foundation Construction* (1996), which contains useful information relating to observations to make during the construction works.

101.5.1 Key learning points

- Significant health and safety issues where appropriate should be included on the as-built drawings.
- Observations made during construction works are important and need to be recorded even if they do not impact on the current design.

101.6 Documentation systems and preserving data

Consideration needs to be given to the maintenance and accessibility of close-out reports. To facilitate the handling and storage of reports it is increasingly common for such reports and supporting data to be stored in electronic form. Various international standards are available that relate to the indexing, handling and transferring of electronic information. Electronic storage is subject to fast-moving technological changes in storage media type, availability of software (e.g. backward compatibility) to read the electronic version of the report and hardware able to support this software. It is recommended that hard copies are maintained as a safeguard, for example against software compatibility issues or corruption of electronic data.

Where a paper print-out of an electronic file (typically a spreadsheet) is provided then the format of the print-out should visibly include all the computational steps that were undertaken within the spreadsheet to produce the end result. For example, the steps in conversion of a frequency reading from a vibrating wire piezometer into a groundwater level should be shown together with cross-reference to the appropriate calibration data sheets.

In addition to providing the close-out report to their client, the consultant or contractor should hold such reports, information and data for the period required by their contract (typically six or 12 years). Note Eurocode 7 recommends 10 years with the more important documents stored for the lifetime of the relevant works. There should be mechanisms in place to have the ability to read the electronic data even if this means opening and re-saving the data using current software/hardware to safeguard against software/hardware becoming obsolete. However, the drawback is that data might be lost or damaged during this process and holding a hardcopy would allow the electronic copy to be verified against the hard-copy version. Finally, consideration should also be given to the safeguarding of the documentation storage site with off-site back-up of electronic data being a benefit against total loss of the main storage site. With the rising cost of storage there will be continuing pressure on destruction of the original design information. Consequently, the close-out report will become an invaluable reference for future works, be they ongoing maintenance, demolition or re-use.

101.6.1 Key learning points

- Maintain a hardcopy of the close-out report.
- Review software/hardware backward compatibility at regular intervals.

101.7 Summary

The close-out report, the final part of the geotechnical cycle, is intended to describe, collate and summarise all relevant geotechnical works that occur during the construction phase of the works together with any post-construction monitoring. It forms an essential part of the construction verification process. The report also provides valuable information on the geotechnical works allowing the future re-use of these works and hence increasing the sustainability profile of the initial works.

101.8 References

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Health and Safety Executive (HSE) (2007). *Managing Health and Safety in Construction*. Construction (design and management) Regulations 2007. HSE Health and Safety: Legal Series L144. Approved Code of Practice (ACOP). London: HSE.

101.8.2 Web references

Association of Geotechnical and Geoenvironmental Specialists (AGS); www.ags.org.uk

It is recommended this chapter is read in conjunction with

- Chapter 78 *Procurement and specification*
- Chapter 93 *Quality assurance*
- Chapter 96 *Technical supervision of site works*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles*. A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation*.

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