
Earthworks: a guide

Second edition

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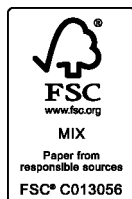
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Preface to the first edition

Earthworks are the most common product of civil engineering operations. Nothing can be built without some excavation and some transfer of soil (or rock) from one part of a site to another. Although normally seen as major structures such as earth dams, or cut and fill for airports, highways or railways, many earthworks are connected with minor civil works and building construction. Typically they comprise small-scale cuts and embankments for diversions and widening schemes, together with site filling, levelling and estate road construction. In many cases, earthworks involve stripping sites of contaminated land or backfilling former quarries or open-cast mines. Whatever the type of work, the principles are the same.

It was not until relatively recently that earthworks were designed. The practical engineers driving roads, railways and canals across difficult terrain during the industrial revolution would build cuts and embankments at the slopes they believed appropriate on the basis of experience, cutting them back and slackening the slopes if failure occurred and using the same slopes for similar soils on the next occasion, if it did not. Thus maintenance engineering became accepted as part of earthworks in a way quite inappropriate for foundation engineering. Here, given the dangerous consequences of failure to a structure's occupants, the designer had to get things right first time. Even if a building did not collapse with life threatening consequences, an underpinning operation was hardly regarded as maintenance; on the contrary, it was (and still is) seen as an operation on a structure which had failed in its function in a way that most failed slopes had not.

As the volume of traffic using civil engineering infrastructure increased, this philosophy had to change. Today, failed earthworks are no longer a minor inconvenience but are seen as potentially dangerous by a public which expects higher standards of infrastructure facilities, yet is becoming more and more risk averse. There is every expectation that in the early decades of the twenty-first century this situation will continue.

This book attempts to bring together topics usually considered as being essential to earthworks engineering. As a guide, it does not attempt to treat each exhaustively but to highlight what may be regarded as salient features. In particular, no attempt is made to establish technical proofs: this is amply dealt with in textbooks elsewhere. Instead, attention is paid to the practical consequences of

the choices made by the engineer and to parameter selection, particularly in the matter of cut and embankment slope design. After a brief historical review (Chapter 1), the essential background to the compaction process is established (Chapter 2) and a description given of some fills commonly available, including some industrial materials (Chapters 3 to 6). There is a note on the nature of some current specifications (Chapter 7) and a description of design and construction control procedures (Chapter 8), noting that design and control in earthworks are intimately linked. The essential processes behind excavation, fill placement, cuttings and embankments are presented (Chapters 9 to 12), together with a brief note on site safety (Chapter 14).

A separate chapter (Chapter 13) is devoted to factor of safety of cuts and embankments. Developments in risk analysis and risk management suggest that the traditional approach, a lumped factor for all circumstances, may be inadequate and that more emphasis should be placed on the degree of risk to which the structure is likely to be exposed. Thus factor of safety may be regarded as a matrix in which risk to life on the one hand, and environmental and economic risk on the other all play a determining role. In this respect, the work of the Hong Kong Geotechnical Engineering Office in its *Geotechnical Manual of Slopes* (1984) is freely acknowledged. The interested reader will doubtless wish to consider the BS Eurocode 7 (1995) approach to factor of safety selection, as an alternative.

It is hoped that the guide will be a useful accompaniment to BS 6031: 1981 Code of practice for earthworks, the *Specification for Highway Works* (1998) and the various Advice Notes issued by the Highways Agency, particularly Advice Note HA 44/91. The readership is expected to include the chartered or near chartered civil engineer, who has a reasonable understanding of the essential soil mechanics vocabulary. It is also hoped that the geotechnical or engineering geologist specialist will find the material of interest, as will the university student. The intervention of suitably qualified and experienced specialists is considered essential in some matters of design and construction, especially in those topics dealt with in Chapters 11 to 13. The need for such intervention is occasionally stated, but in any doubt, specialist advice should always be sought. Likewise, there will be a need for a drainage engineer (for cuttings and embankment design and construction) and a rock mechanics engineer (for cuttings in rock). The involvement of the site safety

engineer to augment the information given on safety is essential.

Developments not covered include the engineering of contaminated land. Another highly promising development, also not covered, is the use of vegetation to reinforce earthworks. As the public becomes more and more sophisticated in its expectations of the civil engineer in the environmental context, it is likely that there will be welcome progress in this field in the years ahead. Nevertheless, the requirement for information on the design and construction of conventional earthworks is expected to remain for a long time.

N.A. Trenter, 2001

Preface to the second edition

The subject of earthworks is special because it provides an added dimension to the normal complexity of soil mechanics, which is that the engineer has to determine how best to mould the soil to create an appropriate fill and the properties of that fill will depend on how well the compaction has been managed. Both of us have found the subject fascinating and enjoyed using our knowledge and experience to deliver successful earthworks projects. However, all too often we find that the earthworks elements of a project are dealt with by engineers who clearly have little knowledge of the subject; which commonly leads to major problems, and contractual claims that could have been avoided. Whilst being called to provide advice to resolve such problems is very interesting, our hope is that by updating this guide to earthworks we can help many professionals to understand the subject and avoid future problems.

Around the world clients commonly require earthworks to meet three criteria: a defined stiffness, an earthwork platform that will remain stable, and achieve 95% of maximum dry density during construction. The project engineers then generally focus on the density criteria assuming it will ensure that the other two requirements will be achieved, but this often proves not to be the case. Over the past 20 years the Highway Agency, Building Research Establishment and the first edition of this book advised against this overly simplistic approach, but engineers continue to make this mistake. One of our priorities with this second edition has been to clarify and develop the guidance on the selection of appropriate material properties for earthworks fills (which can be thought of as ‘fill material design’).

Within this second edition we have sought to bring the book in line with the requirements of BS 6031 and Eurocode 7 that are the current standards that govern the field of earthworks, and have also drawn upon certain aspects of a draft European earthwork standard. However, since earthworks remains as much art as it is science we have avoided attempting to make the second edition of this book a servant to those standards. Our main aspiration has been to prepare a book that will be useful to those involved with the field of earthworks.

Paul Nowak and Peter Gilbert, 2015

Acknowledgements for the first edition

This book began as a Design and Practice Guide, one of a series produced by the Institution of Civil Engineers, and the early drafts on compaction were reviewed by a peer group comprising Mr F.R.D. Chartres, Professor B.G. Clarke, Mr L. Threadgold and Mr I.L. Whyte. Their patient and constructive assistance and the many suggestions made at that stage are greatly appreciated. The author would also like to express his thanks to his former colleagues at the Geotechnics Department at Halcrow Group Ltd for their advice, particularly to Dr D.H. Beasley, Mr A. Andreou, Dr N.J. Burt and Dr R.A. Nicholls who made several valuable comments on the text, and to Mr J. Thrift who produced much useful material for the chapter on site safety. The Halcrow Group Librarian, Ms Julie Cordrey, always provided courteous and efficient help, which is gladly acknowledged.

Acknowledgements for the second edition

Within this second edition, we have endeavoured to follow the structure that Neil Trenter used for the first edition as closely as possible. Hopefully this will help the many readers who have used the book for reference over the past 14 years. We felt some sections had nothing that needed to be added, whilst others have been modified significantly, either to reflect changes in current practice and standards that govern the topic of earthworks, or to add detail where subjects were overlooked or only covered in limited detail in the first edition. Therefore our first acknowledgement must go to Neil Trenter for writing the book that has become the key reference in the UK on the subject of earthworks. Our second acknowledgement must go to ICE Publishing for giving us the opportunity to update the book to ensure it remains relevant for the next decade.

We would like to thank Atkins (our employer) who has supported our work, and in particular Shaun Smith whose thoroughness in reviewing all the chapters, and drafting various figures, tables and paragraphs has been enormously helpful. We would also like to acknowledge the input of the following on particular sections of text: Bob Stork (various), Steve Dunn (soil stabilisation) and Peter Neville-Jones (hydraulic fills).

Over the past eight years Peter has been heavily involved with preparing earthworks standards for the UK and for Europe; this has been a great way to broaden and focus knowledge which has benefited the development of sections of this book, and I'd like to acknowledge the wisdom and friendship of all of that team, in particular: Lee Parry, Alan Phear, David Norbury, Richard Hocking, Alex Kidd, Niall Fraser, Brian Simpson, Phil Dumelow, Mike Cambridge, as well as those named above.

Finally we would like to thank the many geotechnical engineers and earthworks contractors with whom we have been lucky enough to work, helping us to develop our interest in the subject of earthworks; and to our families for putting up with that interest and their patience over the many hours of compiling this book.

Paul Nowak and Peter Gilbert

Chapter 1

Earthworks: an historical perspective

This chapter provides a discussion of the history of earthworks in the UK, covering various highway, canal, railway and reservoir applications. A timeline, summarising the key developments in earthworks in the UK is presented as Tables 1.1 and 1.2. Sections 1.1 to 1.4 below describe the historical development of earthworks methods up to and including the Industrial Revolution (summarised in Table 1.1), and Section 1.5 describes the development of modern earthworks methods based on scientific research and theory (summarised as Table 1.2).

Tables 1.1 and 1.2 illustrate the interrelationship of these various developments through time. Table 1.1 recognises the historical discoveries by the Romans that were then largely forgotten until the major growth in requirements for earthworks projects as a result of the transport and Industrial Revolutions, when experience was gained primarily through trial and error. Table 1.2 charts the development of modern earthworks based on scientific research and soil mechanics theory. Various sources have been drawn on in the preparation of these tables, including the references mentioned throughout this chapter.

1.1. Highways

The origins of modern British earthworks lie in the Roman period, some 2000 years ago. The Romans constructed about 80 000 km of main roads, centred on Rome and extending outwards into the then Roman empire. Roads were graded into roughly five classes from the *via*, constructed to allow the passage of two carriages and about 4.25 m wide, to the *callais*, a mountain track. In their approach to road-building, the Romans demonstrated their understanding of one of the most important factors influencing the construction and maintenance of roads, namely drainage. This they did in two ways for their principal roads: first by drainage ditches either side of the carriageway; and second by a layered construction, with a cambered surface, standing proud of the surrounding land. Water was not only shed off the road surface but the wearing course was protected from inundation by rising groundwater both by the adjoining open ditch drains and also by the elevated construction.

Roman roads radiated from London and probably formed the basis for Britain's main roads until the eighteenth century, although in the Middle Ages the Church took a keen interest in road and bridge maintenance for communications with churches and monastic institutions. With the dissolution of the monasteries in the sixteenth century, even this interest waned and roads in Britain deteriorated to a level probably not known before

Table 1.1 UK earthworks timeline, part 1: historical development of working methods based on experience (separated by periods of inactivity)

~Date	Earthworks activity in the UK	Related points of interest								
2500–800BC	Bronze and Iron Age hill forts: earthwork fortifications followed contours of a hill (e.g. Maiden Castle in Dorset, British Camp at Malvern).	First use of hydraulic lime in foundation concretes and pozzolanic additives (e.g. volcanic ash) to produce hard cement.								
AD 43–410	Roman roads constructed and maintained, consisted of a metallised surface (i.e. gravel or pebbles) on a placed engineered fill of earth or stone appropriate to the setting. The Romans also introduced the use of hand compaction of earth to construct walls, now known as ‘rammed earth’.									
500–1730	Period of minimal earthworks construction. Roman roads formed the basis of Britain’s network until the eighteenth century. Church involved in road maintenance until dissolution of monasteries in 1536. Few new roads constructed.									
1730–1840	Transport and Industrial Revolutions: Large scale civil engineering projects, using empirical methods for earthworks design, with most embankments constructed of uncompacted fill material. Individual engineers (e.g. Metcalfe, Telford and McAdam) developed their own approaches to successful construction of transport infrastructure, with some aiming for longevity and others focussing on low initial cost (see Figure 1.1).	Earthworks techniques developed through increased practice during the Industrial Revolution, largely by trial and error. Some empirical methods were formalised.								
<table><tr><th>Canals</th><th>Dams</th><th>Highways</th><th>Railways</th></tr><tr><td>Pre-1740: ‘contour canals’ – improvements to natural waterways, minimising the need for earthworks.</td><td>1748: Grimsthorpe Dam, Lincolnshire, by John Grundy, the earliest known dam formed with earth fill</td><td>1730–1850 roads: built by rudimentary earthworks methods to link major</td><td>1758: Middleton railway – the first modern railway, transporting coal to Leeds. 1807: Oystermouth</td></tr></table>			Canals	Dams	Highways	Railways	Pre-1740: ‘contour canals’ – improvements to natural waterways, minimising the need for earthworks.	1748: Grimsthorpe Dam, Lincolnshire, by John Grundy, the earliest known dam formed with earth fill	1730–1850 roads: built by rudimentary earthworks methods to link major	1758: Middleton railway – the first modern railway, transporting coal to Leeds. 1807: Oystermouth
Canals	Dams	Highways	Railways							
Pre-1740: ‘contour canals’ – improvements to natural waterways, minimising the need for earthworks.	1748: Grimsthorpe Dam, Lincolnshire, by John Grundy, the earliest known dam formed with earth fill	1730–1850 roads: built by rudimentary earthworks methods to link major	1758: Middleton railway – the first modern railway, transporting coal to Leeds. 1807: Oystermouth							
			John Smeaton (1724–1792), first self-proclaimed civil engineer, designed the third Eddystone Lighthouse							

1741: Newry Canal (NI) and 1761: Bridgewater Canal opened as the first truly 'artificial' canals. Enabled efficient transport of goods between industrial centres and required substantial earthworks. 1770–1830: the 'golden age' of British canal construction with over 3000 km of canal constructed. 1835: Completion of the Birmingham to Liverpool Junction Canal marked the end of the canal boom.	shoulders and a central clay core. Design records show the use of experience to select, place and compact the fill (possibly the first use of bulk fill compaction). Various dams were constructed to feed the growing canal network, e.g. 1797: Rudyard Dam (Leek Canal) constructed by John Rennie.	centres of population. Reliably financed and operated for the benefit of long-distance commercial traffic, rather than to satisfy the limited needs within individual parishes (thus justifying expenditure on earthworks).	railway, Swansea, was the first fare-paying passenger railway in the world. 1812: Middleton railway upgraded to become the first commercial railway to use steam locomotives. 1829: George Stephenson constructed Liverpool and Manchester Railway across Chat Moss by 'floating' the line on a bed of bound heather and branches topped with tar and covered with rubble. 1833–1841: Nine main line railways constructed, 1060 km length and 54 million m ³ of excavation.	(1755–1759) and pioneered the use of 'hydraulic lime', which was critical to the rediscovery/development of cement, and led to the invention of Portland cement. 1807: Geological Society of London founded. 1815: Geological map of Britain published by William Smith, transforming understanding of the nature of natural ground. 1818: Institution of Civil Engineers founded, Thomas Telford appointed as the first president in 1820. 1862: Molesworth's <i>Pocket Book of Engineering Formulae</i> 1st edn, included standard retaining wall sections, earthwork profiles, cut slope angles, and labour for earth moving.
1840– 1890	1887–1893: Manchester Ship Canal construction, largest single earthworks project of the era.			

Figure 1.1 Summary of the different approaches taken of turnpike road construction during the transport revolution

John Metcalfe (1717–1810): built well drained roads with good foundations, and a smooth convex surface to allow rainwater to drain quickly into ditches at the side. Understood the importance of good drainage. Worked out a way to build a road across a bog using a series of rafts made from ling (a type of heather) and furze (gorse) tied in bundles as foundations. He acquired a mastery of his trade with his own method of calculating costs and materials, which he could never successfully explain to others.
Thomas Telford (1757–1834): his system of road building required digging a trench, installing a foundation of heavy rock, and then surfacing with a 6 in (15 cm) layer of gravel. During construction, the centre of the road was raised, producing a crown that allowed water to drain off. Although his system was faster and less expensive than the Romans’ method, it was still costly and required frequent resurfacing with gravel.
John Loudon McAdam (1756–1836): McAdam’s system was based on the principle that a well drained road made of suitable material does not need the stone foundation of Telford’s system, but could be built directly on the subsoil. McAdam placed a closely compacted 10–12 in (25–30 cm) layer of stone, which had been broken to an inch (2.5 cm) in diameter, and which was raised in the centre to facilitate drainage, followed by a carpet of finer grained stone that was cemented by the setting of the powder. The process was completed in stages, allowing the road’s traffic to compact each stage. Advantages to McAdam’s system were its speed and low cost, and it was generally adopted throughout Europe. However, the lack of a firm foundation for the roadbed proved the ultimate undoing of macadam roads with the advent of heavy motor vehicles, especially trucks. On roads that had to support heavy loads, Telford’s system of construction became the standard.

or since. In the seventeenth century, turnpikes were introduced, but their numbers grew slowly. By the time Macaulay wrote his *History of England* in 1848, there were only some 48 000 km of turnpike roads (Pannel, 1964), still over 30 000 km short of the total of main roads engineered by the Romans in their empire 2000 years previously.

With the establishment of turnpikes came the appointment of surveyors: salaried people who were paid to take a professional interest in highways. Standards of road-building began to improve in the late eighteenth and early nineteenth centuries, with Metcalfe, Telford and McAdam incorporating layered construction and drainage into road design. Figure 1.1 compares and contrasts the different approaches developed by these early civil engineers. Although the design principles adopted by these pioneers are known, along with information regarding the construction of the pavement layers, little information exists concerning their approach to earthworks as such. It is not known, for example, whether the fill forming the embankments was systematically compacted or whether there was indeed any concept of suitable or unsuitable fill. However, highways followed topographical contours to reduce cut and fill, and hence expense was kept to a minimum. Telford appears to have been aware of the problem of consolidation when he specified ‘Where the height of the embankment shall exceed three feet, they are to stand from one to three months, in proportion as they increase in depth, as shall be determined by the inspector’ (Pannel, 1964). There are a small number of drawings from the 19th century showing earthworks methods such as the example in Figure 1.2.

Figure 1.2 Earthworks in the Strand, London, 1851. Note the foreman in the foreground beating time (from Binnie, 1987)



1.2. Canals

Early canals simply followed the topographic contours, following natural waterways wherever possible, and with earthworks being constructed largely on side-long ground in order to minimise the earth moving requirements (Reeves *et al.*, 2006). The Industrial Revolution brought a requirement to transport goods efficiently between major centres; this required direct routes and necessitated earthworks and locks to create the first ‘true canals’, which were artificially created waterways.

Modern canal construction in Great Britain began in about 1760 with the Bridgewater Canal and was to continue for well over 100 years. Between 1770 and 1830, over 3000 km of canals were built and by the end of the eighteenth century, the navigable Mersey, Trent, Severn and Thames rivers were all interconnected (Binnie, 1987). Although their construction may not be as celebrated as the later railways, they presented at least as many difficulties because canals had even more stringent vertical alignment criteria and had to be made leak proof. This latter point alone demonstrates the intuitive skills of the eighteenth and early nineteenth century ‘navvies’; with no theory to guide them, they must nevertheless have been intimately acquainted with the required fill properties. There was also more civil engineering, both design and construction, associated with lock building and with the reservoirs from which the canals drew their water.

Probably the greatest earth-moving operation in Britain in the nineteenth century was the Manchester Ship Canal, constructed in the 1880s, which involved excavating and removing over 53 million yd³ (40.5 million m³) of material, of which 12 million yd³

(9.2 million m³) was sandstone rock. The majority of the excavation was performed in the dry by nearly 100 steam excavators, backed up by over 6000 spoil wagons drawn by over 170 steam locomotives. The Ruston and Dunbar Steam Navvy was the most popular piece of excavating equipment at the time and comprised a self-propelled rail-mounted wrought iron frame with the engine and boiler at one end and the crane at the other. It had the same role as a modern face shovel, and the machine had a capacity of 1000 yd³ (765 m³) per day.

1.3. Railways

If large earthworks could be avoided with highway construction, they could not be avoided with railways, because of the shallow gradients required. The London to Birmingham railway constructed by Robert Stephenson in the 1830s was probably one of the first modern examples of disastrous cost over-runs in British civil engineering. The final cost more than doubled from £2.4 million to £5.5 million, or some £50 000 per mile. The main reason appears to be Stephenson's wish to avoid large gradients, in railway terms, which meant that tunnels and deep cuttings had to be introduced. The London to Birmingham railway was one of the first major civil engineering structures to encounter the formidable problems posed by the London Clay, with the high retained cuttings between Camden Town and Euston and the Primrose Hill tunnel. Stephenson was also unfortunate enough to neglect the 'desk-study phase' of the site investigation for the Kilsby tunnel. The investigation failed to detect a 400 m length of 'quicksand', even though there was local knowledge of the feature. Eventually, it took 13 pumps and some 18 months to stabilise.

A completely different approach was taken by Locke with the Grand Junction railway, which connected the Manchester to Liverpool line to Birmingham. Locke accepted steeper gradients (within the capabilities of the locomotives of the time) and made use of the terrain. He was therefore able to dispense with risk-laden tunnels and deep cuttings, at least to some extent, and the cost of the Grand Junction railway was reportedly only £19 000 per mile, roughly one-third of the cost of the London to Birmingham railway. This approach demanded, at the same time, more earthworks including the Penkridge bog crossing.

According to Skempton (1996), some 2–3 million yd³ (1.5–2.3 million m³) of earthworks per year were undertaken on each of the nine main railway lines under construction in England between 1834 and 1841. Thousands of men were employed, hundreds of horses, but only a handful of steam locomotives and stationary engines. This compares with the 7 million yd³ (5.4 million m³) per year during the construction of the early sections of the M1 motorway in the late 1950s, using modern construction plant (and producing well compacted fill). It is a tribute to the organisational and management skills of the early engineers that their productivity rivalled that achieved more than a century later. Although some of the fill placed would have been layered, especially at bridge abutments, this method would have proved too slow for the majority of construction. Most fill was end-tipped using teams of horse-drawn earth wagons. Settlement would have been large but seems to have been accepted as an inevitable consequence of the construction method. Skempton reports that settlement effects would have been mitigated by speed restrictions and by packing ballast beneath sleepers until settlements had stabilised.

McGinnity and Russell (1995) describe some of the London Underground earth embankments (largely London Clay). Built 60 or 70 years after those of Stephenson and Locke, and still in use today, they were constructed with ‘minimal, if any, effort at compaction’. The authors report that the large resulting settlements were made up by the addition of further clay fill and, once construction was complete, still further settlement would be made up using ash from the steam locomotives then in use; in other words, techniques very similar to those of the 1830s and 1840s.

The use of hand labour and horse-drawn wagons for earthworks in the early nineteenth century was replaced by steam-driven equipment in the latter half of the century, under the pressures of shortage of labour, now becoming increasingly organised, and the cost of loans for construction (Pannel, 1964). But even earlier, Robert Stephenson’s Kilsby tunnel (referred to above) was rescued by steam pumps without which, according to some reports, tunnel completion may well have been impossible. Steam-powered winches were increasingly replacing the horse gin, the capstan and the treadmill, and steam-driven piling hammers were coming into use.

For a useful introduction to the history of road and railway construction in the UK, the reader is referred to Pannel (1964) and Upton (1975).

1.4. Reservoirs

The earliest example of an English earth dam for which engineering details survive is Grimsthorpe (Skempton, 1979). Constructed in 1748, the 18 ft (5.5 m) high structure was not built to retain a canal reservoir but an ornamental lake, and contained a central clay core with earth shoulders. It exists to this day. The engineering details (dated 1766 and 1767) refer to a 25 ft (7.6 m) high dam that was to have been built downstream of the original, also with a central core of clay ‘well rammed and watered’ and shoulders of ‘rammed earth’. It is likely that these details also apply to the original structure. Interestingly, according to the records, the designer, Grundy, allowed 2 in (5 cm) of settlement per foot (0.3 m) of dam height and stated that the new dam would have to be built 29 ft (8.8 m) high. The construction of the Grimsthorpe dam is one of the earliest documented applications of formal compaction of bulk earthworks fill material, and is an excellent example of rational design and construction based on a combination of observation, experience and engineering judgement.

Earthworks for dam engineering require the highest degree of skill, both in terms of design, construction and also subsequent maintenance, and, given the crude methods available at the time, it is surprising that so many of the old dams survive to this day. One of the early failures was the Blackbrook dam (1795–1797), which failed twice and was rebuilt as a masonry structure a century later. The engineers for the later structure found that there had been no attempt to construct a puddle clay cut-off down to solid rock; only a narrow trench filled with ‘riddled soil’ had been constructed and founded in the debris covering the valley bottom. The dam shoulders were constructed from ‘small lumps of rock mixed with soil, and were full of cavities’ (Binnie, 1987).

Given the compaction techniques available, some form of puddle clay central core would have kept leakage to manageable proportions. This was not always the case. None of the dams built for the Rochdale Canal (completed in 1804) had clay cores (Binnie, 1987). A reasonably high standard of compaction is therefore implied, with the fill well keyed into the underlying solid rock, especially as steep slopes were often employed. The 12.5 m high Blackstone dam built by Jessop was originally raised with side slopes of 1V to 1.5H, which remained unaltered until 1878 (upstream) and 1925 (downstream), when they were slackened.

Cases of bad practice were, however, recorded. One observer noted ‘such was the inattention of the engineer that he laid the foundation of the bank upon nothing but the moss and ling that covered the soil’. The 21 m high Slaithwaite dam for the Huddersfield Canal was higher than any other built at the time (Binnie, 1987). It was reported to have leaked badly and the failure of the draw-off system points to large settlements of the embankment, no doubt due to poor compaction.

John Rennie’s specification for the 10.5 m high Rudyard dam for the Leek Canal exists, dated 1797. It is of historical interest to the earthworks engineer because it describes in some detail how the embankment should be raised, namely on solid ground in ‘regular stratas throughout its whole length, none of which should exceed 2 ft thick’. The layers were required to dip towards the dam centre where a puddle core was provided to keep the puddle moist. No details of the fill are given except that it should comprise ‘a good mixture of earth with few stones in it except small ones, that the whole may be solid throughout’. Of the puddle clay core, it was required to be placed on ‘good ground both in the bottom and the sides of the vale’. It was to be ‘made of good clay or clay and gravel mixed in the proportion of about 2 of the former to 1 of the latter’, laid in 8 in (20 cm) layers. Rennie specified that they should be covered with water ‘until they (the layers) are dissolved, after which they must be well chopped and mixed, rising gradually with the bank’. The reference to ‘chopping’ means labourers working the puddle with spades. Despite the specification, the bank is reported to have leaked at both ends when the reservoir was raised above a certain level, probably because the cut-off trench was not founded on the ‘good ground’ specified by Rennie at the higher parts of the valley slope (Binnie, 1987).

Historically, large amounts of hand labour have always been employed in major earthworks. Penman (1986) describes how construction of embankment dams in India could involve 20 000 people excavating soil by hand, carrying it on the head, and then spreading it in thin layers to be watered and compacted underfoot. In Spain, a 28 m high dam was built by 1500 convicts, 200 labourers, 400 beasts of burden and (only) four steam engines. Compaction was by herds of animals driven backwards and forwards across the placed fill, a common construction expedient before the advent of mechanised plant.

The collapse of the Dale Dyke dam in 1864 with the loss of 250 lives and involving much damage led Victorian engineers to re-examine the traditional embankment dam designs, particularly the puddle clay cut-off. This feature was liable to intense erosion in fissured and fractured ground such as Carboniferous Limestone. Developments were

introduced, including concrete and brickwork linings and grouting, in order to secure the cut-offs. Another innovation, of major importance to the earthworks engineer, was the introduction of zoning of earth banks with the finer material placed in the shoulders adjacent to the core. This was a consequence of the failure of the Bilberry dam in 1852. This measure was introduced to control differential settlement between the shoulders and the core, by providing a more uniform decrease in stiffness when progressing from the outer slopes to the central core. However, it also introduced a filter effect, which dam engineers later discovered was of great importance to the stability and safety of their structures.

An outstanding account of the history of early dam construction in the UK is provided by Binnie (1987).

1.5. Modern development of a design-based approach

The breakthrough of a number of scientific theories during the nineteenth century provided the mathematical foundations for others to apply to soils, and this was taken forwards with the establishment of the new discipline of soil mechanics during the early part of the twentieth century. These advances led to extensive research in the field of earthworks enabling the development of design methods, improvements in construction techniques and formalisation of codes of good practice and standards. Some of the key events are described below and are summarised in Table 1.2.

The theory of earth pressures was formulated in Great Britain by William Rankine (1820–1872); in France, Collin (1808–1890) investigated the stability of both undisturbed and remoulded soils (such as are found in earthworks) by a study of slope failures in railway cuttings and embankments, and in earth dams. However, the extent to which these studies were taken into account in earthworks engineering later in the nineteenth century is not clear.

Mohr's development of a generalised form of Coulomb's theory at the end of the nineteenth century (Mohr–Coulomb theory) paved the way for others to derive methods of slope stability analysis.

The massive slope failures that accompanied the construction of the Panama Canal prompted the formation of a committee by the American Society of Civil Engineers in the first decade of the twentieth century to investigate the problem. At about the same time, in Sweden, a state commission was established to examine the reason for failures in railway embankments and cuttings, which had claimed over 40 lives. These studies led to the development of the Swedish method of slices for the solution of slope stability problems. Research into stability problems also began in Germany following the construction of the Kiel Canal, where failures had been common. Again it is not clear to what extent the results of these studies were adopted as routine in earthworks engineering at the time.

The large scale mechanisation of earthmoving equipment in the USA in the 1920s and 1930s created the need for rational design methods for earthworks compaction and a rapid means of control. The work of Proctor (1933) and Porter (1938) in California in the 1930s led directly to major improvements in these areas of earthwork engineering.

Table 1.2 UK earthworks timeline, part 2: twentieth century development of earthworks standards based on scientific research and soil mechanics theory

Date	Milestone events that shaped earthworks practice in the UK	Extended research and developments in theory
~1890	Mohr–Coulomb theory formalised as a mathematical form for engineering design, providing the opportunity for slope stability analysis to be developed in the twentieth century.	
1927	Fellenius used limit equilibrium theory to determine the ‘ordinary method of slices’, providing the first model to enable engineers to calculate the stability of slopes based on circular failure planes.	1920s to 1940s: Karl Terzaghi (1883–1963) established the fundamentals of soil mechanics enabling a design-based approach to geotechnical engineering. 1936: Terzaghi and Casagrande organised 1st International Conference on Soil Mechanics and Foundation Engineering. Terzaghi finally published a collation of his work as <i>Theoretical Soil Mechanics</i> 1st edn in 1943.
1933	Proctor developed the concept of optimum water content and the standard compaction test in connection with dam construction. Researchers then studied the compaction process at a laboratory scale. The test soon started to be used on major UK earthworks projects.	
1936–1937	Chingford dam construction; the first use of mechanised plant to form embankments of compacted fill. The dramatic increase in construction rate led to slope failures. Led to Mowlem establishing the first soils laboratory.	
1939–1945	World War II – major advances in earthworks theory and construction practice included: 1942 Casagrande soil classification system; TRL established the relationship between soil type, water content, compactive effort, fill density and air voids; cement stabilised soils pioneered for airfield runways; towed sheepfoot rollers and small vibrating plate compaction plant developed.	
1948	BS 1377 1st edn includes Proctor compaction tests (modified Proctor test developed but not included in BS).	1940s to 1980s: Taylor (1948) produced slope stability charts for design. Bishop (1955) developed a method of analysing circular failure surfaces; then extended by Morgenstern and Price (1965) and Janbu (1973) for analysing non-circular failures. Analysis of individual slopes by computer software became routine during the 1980s.
1950s	Large self-propelled scrapers become common on major earthworks projects. Development of vibratory rollers.	
1951	<i>Specification for Road and Bridge Works</i> 1st edn (title changed to <i>Specification for Highways Works</i> [SHW] in 1986), with compaction control to be achieved by ‘end product compaction’ criteria.	

1952	<i>Soil Mechanics for Road Engineers</i> published: TRL collated World War II advances in earthworks technology for application on civil engineering projects, including a detailed understanding of compaction theory. Compaction plant at this time was limited to deadweight rollers, the most commonly used being 8 ton steam engine rollers.	1940s to 1990s: 1946–1990: Comprehensive programme of research by TRL on the Compaction of Soil and Granular Materials, by laboratory testing and full-scale field trials to understand earthworks and fill materials and improve compaction plant. Collated by A. W. Parsons (1992), which forms the basis of modern earthworks theory in the UK (the Highways Agency's <i>Design Manual for Roads and Bridges</i> [DMRB] design manual HIA 70/94 was prepared based on this work).
1951–1954	Jackfield landslip enabled Skempton to clarify the role of relict slip planes and residual angle of shearing resistance.	
1951–1968	M1 built, followed soon afterwards by the main routes of the present motorway network.	
1959	<i>BS 6031</i> 1st edn published (as CP2003) Code of Practice for Earthworks (largely unchanged in 1981 revision).	
1969 and 1976	<i>SHW</i> 4th and 5th edns replaced end-product compaction with 'method compaction' (by 1976 the methods were similar to those of the present). Vibrating rollers were permitted and became commonly used through 1970s/1980s.	
1975	<i>BS 1377</i> updated, formalising the earthworks soil testing methods that are used today in <i>BS 1377:1990</i> .	
1978	Earthworks conference entirely on 'clay fills' held at the ICE.	
1970–1986	M25 orbital motorway constructed in sections (188 km long).	
1986	<i>SHW</i> 6th edn in modern format with: 600 series clauses; engineer to select appropriate fill acceptability properties at Table 6/1 based on relationship testing; MCV test added to improve control on fill material suitability; re-introduction of 'end-product compaction' for certain fill classes and updated 'method compaction'. This triggered further developments of compaction plant.	
1996–2007	First high-speed railway constructed, 108 km Channel Tunnel Rail Link (HS1).	
2004	<i>Eurocode 7</i> (EC7) system of common European standards introduced into UK practice.	
2009	<i>BS 6031 Code of Practice for Earthworks</i> fully updated to reflect modern methods and EC7.	

Proctor developed the concept of ‘optimum water content’ for soil compaction when he was involved in the construction of earth dams, while Porter developed another (static) compaction test for the California Division of Highways. This was designed for use with an important innovation in pavement design, the California bearing ratio (CBR) test.

The introduction of mechanised plant in the UK brought new challenges to earthworks construction. The construction of the William Girling reservoir (commonly known as ‘Chingford dam’) commenced in 1936, and marked the first occasion when mechanical scrapers and bulldozers were used for major earthworks in the UK. The use of such plant significantly increased the rate at which fill could be placed and compacted, and during 1937 two major slope failures occurred in the partially completed embankment. Investigations found that the cause of the failures was excess pore water pressure development in the underlying soft alluvial clays, brought about by the rapid construction (Goodman, 1999). The research associated with this failure is generally recognised as the birth of modern soil mechanics in the UK, with Terzaghi and Skempton working together to resolve the problem. It led to the contractor, Mowlem, establishing the first commercial soils laboratory in Britain. The construction of the dam was eventually completed in 1951.

The outbreak of World War II in 1939 provided impetus for simple methods of classifying soils for military airfield and road-making purposes. The Casagrande system of soil classification was published in 1942 and drew heavily on the concept of a soil’s plasticity index, as formulated by Atterberg in Sweden in the early years of the century. This classification was used as a basis for British classification schemes to follow. During the war, and in the years immediately following, the Road Research Laboratory (later to become the Transport Research Laboratory [TRL]) undertook extensive trials to develop the relationship between soil type, water content, compactive effort, fill density and air voids. This work was undertaken to improve methods of construction of both roads and airfield runways that were critical for the war effort, and led to various developments in earthworks technology (e.g. the first formal use of cement stabilised bulk fill material). Following the war the research was formally collated for civil engineering purposes in the book *Soil Mechanics for Road Engineers*, which was published in 1952 (Road Research Laboratory, 1952). This comprehensive publication shows that compaction theory was very well understood at this time, and still provides a useful reference to this day.

Research into earthworks at the Transport Research Laboratory continued after the war until 1990. It was during this time that the body of knowledge relating to earthworks that we have now was developed, along with modern compaction plant. This work was comprehensively captured in a single volume by Parsons (1992).

A landmark document that was borne out of the extensive testing undertaken by TRL was the *Specification for Road and Bridge Works*, first published in 1951 by the Ministry of Transport (later to become the Highways Agency (HA)). This formalised compaction to an ‘end-product’ criterion, which simplified the tendering process and made compaction supervision much easier because of the unified approach to compaction. It is worth

noting that the majority of the UK's early motorways were constructed using 8 ton deadweight rollers, compacting to an end-product criterion in accordance with this specification. The document saw several subsequent revisions, with 'method compaction' replacing end-product compaction in 1969 to reduce the problems and delays associated with the huge number of tests that were required on major earthwork projects. The 1976 update included vibratory rollers, which gradually became commonplace. The publication of 'the Brown Book' (6th edn) in 1986 saw the title of the specification changed to the *Specification for Highway Works* (SHW) and the reinstatement of end-product compaction for certain classes of materials, with bulk earthworks continuing by method compaction. The current edition of the SHW (HA, 1998, 2013) has changed relatively little since 1986.

The comprehensive update of the British Standard BS 6031 in 2009 (BSI, 2009) recognised the fact that the SHW had become the form of earthworks specification used across most industries in the UK. This standard captures modern earthworks good practice and provides guidance on the application of the Eurocode design approach on earthworks projects.

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Chapter 2

The compaction process

2.1. Introduction

Compaction is the process by which soil particles are forced together by reduction of the air voids content. It has the following effects on most materials when compared with their loose state:

- increase in density
- increase in shear strength
- reduction in compressibility
- reduction in permeability.

2.2. Phase relationships

Soil or rock fill comprises three components or phases:

- solid (soil or rock particles)
- water (leachate or other fluid)
- air (or other gases).

The classic vehicle for understanding the relationship between these three phases is the phase diagram (Figure 2.1). From this diagram, certain important relationships can be written down and the following formulae derived.

$$\text{Degree of saturation } S_r = \frac{wG_s}{e} \quad (2.1)$$

where w is the water (moisture) content, G_s is the specific gravity of the soil and e is the void ratio.

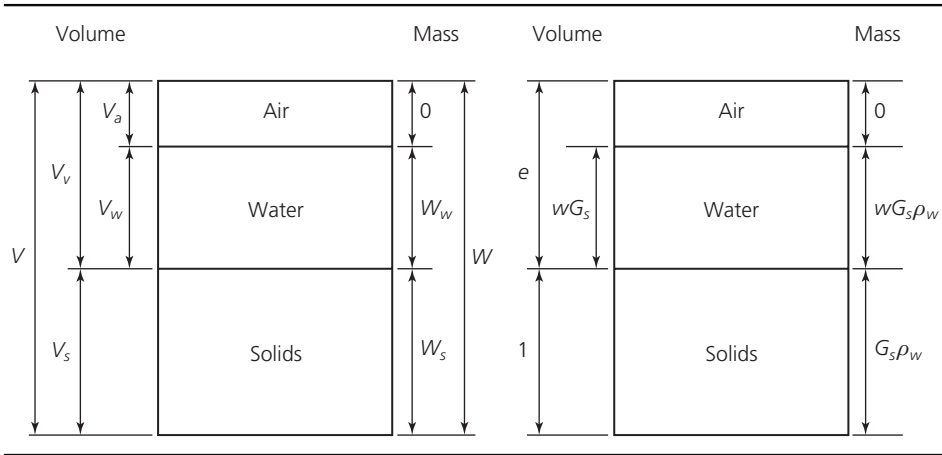
$$\text{Dry density } \rho_d = \frac{G_s \rho_w}{1 + e} \text{ (Mg/m}^3\text{)} \quad (2.2)$$

$$\text{Bulk density } \rho = \frac{(G_s + S_r e)}{1 + e} \rho_w \text{ (Mg/m}^3\text{)} \quad (2.3)$$

where ρ_w is the water density.

In BS 1377 (BSI, 1990), the term particle density (the product of the specific gravity and the water density ρ_w) is used instead of specific gravity but, as pointed out by Craig

Figure 2.1 Phase diagram for establishing properties of compacted fills (from Craig, 2004)



(2004), it is advantageous to use the dimensionless specific gravity when deriving relationships from Figure 2.1.

Bulk density and dry density are related by means of the water content, as follows:

$$\rho = \rho_d(1 + w) \text{ (Mg/m}^3\text{)} \quad (2.4)$$

Air voids content is the ratio of the volume of air to the total volume and is written:

$$A_v = \left(1 - \frac{\rho_d}{G_s \rho_w} [1 + wG_s]\right) \quad (2.5)$$

The degree of saturation S_r and air voids content A_v are expressed in terms of void ratio by:

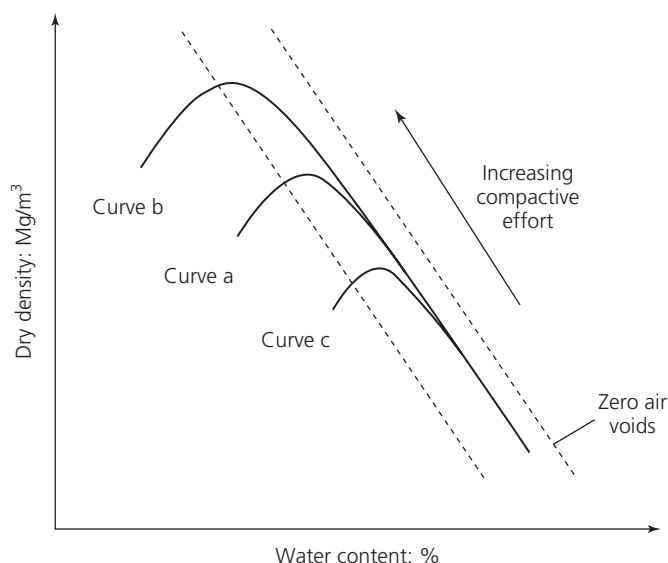
$$\frac{A_v}{1 - S_r} = \frac{e}{1 + e} \quad (2.6)$$

Void ratio may be expressed in terms of specific gravity, water density and dry density from Equation 2.2, as follows:

$$e = \frac{G_s \rho_w}{\rho_d} - 1 \quad (2.7)$$

2.3. Optimum water content and maximum dry density

If a soil is subjected to the same compactive effort over a wide range of water contents, a curve such as the one shown in Figure 2.2 (Curve *a*) results. At low water contents, the dry density is low but, as the water content increases, the soil particles of fine soils may be softened and lubricated, which facilitates air void reduction. Dry density increases as a consequence.

Figure 2.2 Effect of compactive effort on compaction properties

When the water content increases above a certain value, water and air in combination begin to resist further close packing of soil particles. The addition of more water only increases the total voids in the soil and the dry density begins to fall as a result. The water content at which this occurs, for the given compactive effort, is termed the optimum water content (OWC) and the corresponding dry density is termed the maximum dry density (MDD).

According to Vaughan (1982), when the degree of saturation exceeds about 90%, usual for most UK soils wet of optimum, air is occluded and the soils can sustain positive pore pressures; below this percentage saturation, air is continuous and pore water pressures will be sub-atmospheric or negative. Thus the degree of saturation of the excavated soil will control, in part, the (undrained) behaviour of the fill when placed and compacted, unless special measures are taken to condition the soil (see Section 10.5). This 'downwards' leg of the curve, on the wet side of optimum, is typically approximately parallel to the 0% air voids line (sometimes referred to as the 'saturation line') as shown in Figure 2.2.

Typical practice in the UK is to plot air voids lines onto compaction charts, with 0%, 5% and 10% air voids commonly being shown in relation to the compaction curve. As shown in Equation 2.5, air voids are related to the specific gravity of the soil, and so the specific gravity needs to be established for accurate plotting of these lines. Some European countries, particularly those that frequently work with dry fills, plot saturation curves instead of air voids lines. For a given specific gravity, these curves are divergent and,

relative to air voids lines, they are sub-parallel and plot at a steeper angle to the vertical. While the use of saturation curves is the standard approach in several European countries (e.g. Spain), experience of UK soils tends to show that air voids lines define the zone of acceptability more effectively.

2.4. Effect of compactive effort

Curve *a* shown in Figure 2.2 is just one of an infinite number that could be produced for a given soil, depending on the compactive effort. If the compactive effort is increased, the apex of the curve moves upwards and to the left (Curve *b*); if it is decreased, the apex moves downwards and to the right (Curve *c*). Also shown in Figure 2.2 is the zero air voids content. At water contents in excess of OWC, the air voids content does not change significantly, whatever the compactive effort employed.

The orientation and packing of the soil particles change with water content and compactive effort. For a given compactive effort, the soil texture varies from disaggregation at low water content to more and more particle orientation, until at OWC the compacted soil takes on a more homogeneous and orientated texture. At water contents considerably dry of OWC, the fill, particularly clay fill, can be unstable, as will be shown in later chapters.

The OWC varies with compactive effort as illustrated in Figure 2.2; it is not a fundamental property for a given soil, as is sometimes supposed.

In UK practice there are two levels of compactive effort to which soil compaction properties are usually related. Both tests employ rammers to compact the soil; the rammers are of different masses (2.5 and 4.5 kg), fall from different heights and are applied to different numbers of soil layers in the mould. The tests are, respectively, BS 1377:1990: Part 4: Test 3.3 and Part 4: Test 3.5 (BSI, 1990). A third test uses a vibrating hammer of known weight, vibrating at a known frequency for a known time period (Part 4: Test 3.7). The moisture condition value test (Part 4: Tests 5.4 and 5.5), commonly used in earthworks (see Section 8.4.5), can be regarded as a compaction test but one in which the soil sample is compacted to refusal and not to a specified level of compactive effort, as with the tests mentioned above.

It is relevant to note that the original compaction test was developed by R. R. Proctor in the USA (in 1933) and became the ASTM 'standard Proctor compaction' test. The test has been adopted by many countries around the world and is generally recognised as the standard laboratory compaction test, with subtle variations introduced by different countries' standards institutions. The 2.5 kg rammer test is the British Standard version of the standard Proctor compaction test.

In 1958, the modified Proctor compaction test was developed as an ASTM standard. In the UK the 4.5 kg rammer test was established in line with the 'modified Proctor', and is often referred to as the 'BS heavy compaction' test. A number of countries utilise the modified Proctor test for certain situations, but the standard Proctor test has remained the most commonly used.

The choice of test reflects the type of material and type of compaction plant to be utilised. Traditionally, the 2.5 kg rammer has been used for cohesive and fine grained granular soil, the 4.5 kg rammer for granular soils and the vibrating hammer for processed well-graded granular materials.

The level of compaction also attempts to mirror the type of earthworks plant used in construction. This has evolved with technological advances, and heavier plant is now available than that when the testing was developed in the 1930s. Consequently, earthworks will commonly exceed the results anticipated based on the 2.5 kg laboratory compaction test. However, despite these improvements in plant, the 4.5 kg rammer test continues to set an onerous target for earthworks in the field, even with modern heavy compaction plant, and so tends to be reserved for earthworks in which a particularly high standard of compaction is required.

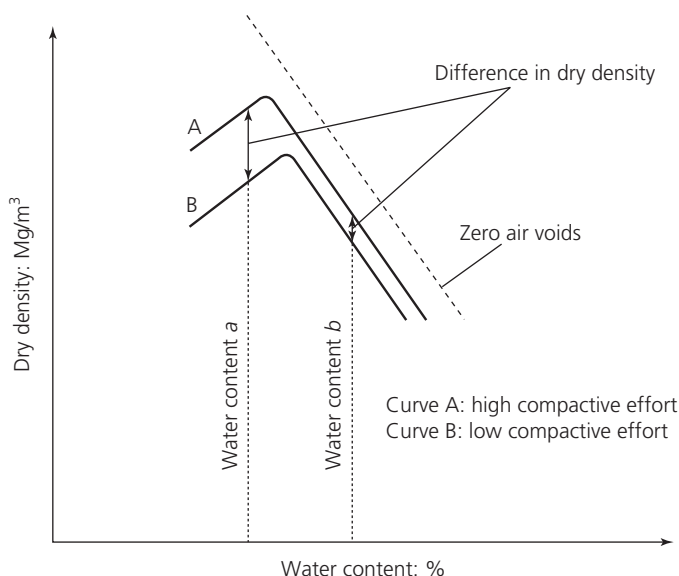
Care should be taken in comparing results across national boundaries. A 100 mm dia. mould is used for 2.5 kg and 4.5 kg rammer compaction in the UK and USA, with a 150 mm dia. mould used for the vibrating hammer test. In France, a 150 mm mould is commonly used for all three tests, which therefore gives rise to a potential for differences in the OWC and MDD for similar materials. Countries working to Russian-based test standards tend to impart a similar total compactive effort but by means of a large number of lower energy blows; the results are supposedly similar but for some soils the compaction curves obtained can be very different.

Earthworks specifications developed in countries with a more arid climate than the UK, for example, Spain, indicate the preferential use of the 4.5 kg rammer test for all materials.

2.5. Effect of initial fill water content

The adoption of higher compactive effort does not necessarily produce a uniform increase in the dry density of the material compacted. The initial water content of the fill (usually the natural water content of the soil at the borrow, allowing for any changes in handling and placing) has a major influence on the maximum dry density as illustrated in Figure 2.3. At water contents at or dry of optimum (water content *a*), there is a large difference between the dry densities achievable for the two compactive efforts. At water contents well wet of optimum (water content *b*), the corresponding difference between the dry densities is small. Therefore, the use of high compactive efforts wet of optimum water content will not be particularly productive in terms of denser fill and, in the case of clay fill, could lead to the construction problems of ‘mattressing’ (see Section 3.4.4).

Note should also be taken of the effect of initial water content on the suction (negative pore pressure) existing in the compacted soil. If a clay fill is compacted considerably dry of optimum, there will be a substantial suction and consequently a marked affinity for water, which could affect fill stability (volume change) during service. Clay fill compacted at or above optimum will show much smaller suctions, with a correspondingly reduced affinity for water. Working with a sand–bentonite mixture, Dineen *et al.* (1999) showed that at optimum water content (2.5 kg rammer), suctions of about 200 kN/m² existed,

Figure 2.3 Effect of initial water content on maximum dry density

whereas at 2% dry of optimum the suctions had more than doubled to 550 kN/m^2 . At 2% wet of optimum, suction had reduced to about 150 kN/m^2 .

Also noteworthy in the Dineen *et al.* (1999) test series was the increase in suctions measured between optimum water content for the 2.5 kg rammer test (about 200 kN/m^2) and the 4.5 kg rammer test (about 1000 kN/m^2). Compacting to a heavier specification does not always produce a more stable fill. This matter is discussed in more detail in Chapter 8.

2.6. Compaction of fill with oversize particles

2.6.1 Effect on dry density

The maximum particle size that can be treated with the BS rammer tests is 20 mm, unless a CBR mould is employed. In this case, a maximum particle size of 37.5 mm may be adopted; however, the proportion between 20 and 37.5 mm should not exceed 30% of the total. Reference should be made to BS 1377: 1990: Part 4: Test 3.4 for the 2.5 kg rammer and to Part 4: Test 3.6 for the 4.5 kg rammer methods (BSI, 1990).

Three common methods are used to correct dry density when particle sizes in excess of 20 mm have been removed:

- the elimination method
- the adjusted maximum dry density method
- the substitution method.

2.6.1.1 Elimination method

The dry weight of the portion larger than 20 mm (the ‘stones’) in the sample is measured and the ratio F is determined, where F is the fraction of the stones to all particles by dry weight. The total dry density ρ_t of the fill is then

$$\rho_t = \frac{\rho_m \rho_w G_s}{\rho_w G_s (1 - F) + F \rho_m} \text{ (Mg/m}^3\text{)} \quad (2.8)$$

where ρ_m is the dry density of the matrix (smaller than 20 mm), ρ_w is the water density and G_s is the specific gravity of the stones. Head (1992) recommends that F should be less than 0.25, but Day (1989) adopts an F value of less than 0.5.

2.6.1.2 Adjusted maximum dry density method

This method (NAVFAC, 1982) is written in terms of maximum dry density of the soil matrix only, and

$$\rho_t = \frac{1 - 0.05F}{\frac{F}{2.6} + \frac{1 - F}{\rho_m}} \text{ (Mg/m}^3\text{)} \quad (2.9)$$

where F is as defined above. A maximum value of F of 0.6 is recommended by the Naval Facilities Engineering Command.

2.6.1.3 Substitution method

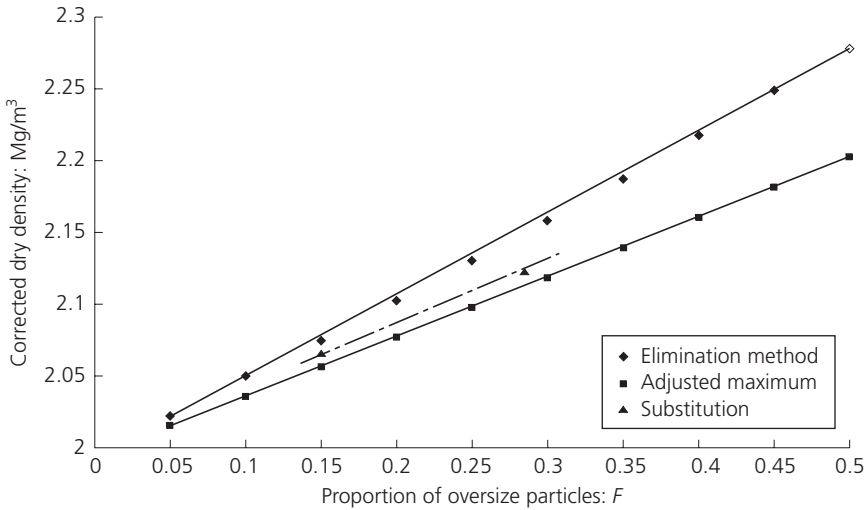
The substitution method involves removing the stones from the sample during preparation and replacing this proportion with particles of a smaller size. The method is no longer included within laboratory test standards because it is not considered directly equivalent, but it is very useful to provide information for designers. The method can be applied for samples with values of F of up to 0.3 (although Winter *et al.* (1998) suggest that the method should be restricted in application to $0.05 < F < 0.1$).

By this procedure, the amount of material passing the 76 mm sieve and retained on the 20 mm sieve is replaced by the same weight of material passing the 20 mm sieve and retained on the 5 mm sieve. The compaction test is then performed on the substituted sample in the normal way.

When the stone content is very small the stones can simply be removed and no correction made, i.e. $\rho_t = \rho_m$. This approach can often provide a reasonable degree of accuracy for samples with F of less than 0.1.

In an attempt to quantify the results of the above three methods, Figure 2.4 was prepared for a soil having a maximum matrix dry density ρ_m of 2.0 Mg/m³ and a specific gravity G_s of 2.65. It will be seen that, for a given value of F , the elimination method gives the largest maximum dry densities and the adjusted maximum dry density method the least: the substitution method produces results below the mean of the two sets of values. The elimination method, often termed the ‘scalping’ method, is frequently used because of its simplicity. However, the fact that its results are not conservative (i.e. it produces higher

Figure 2.4 Correction methods for oversize particles (data from Day, 1989)



values of dry density for a given value of F than the other two methods) should be borne in mind.

2.6.2 Effect on water content

Head (1992) proposed that the total water content of the whole fill w_t may be related to the matrix water content w_m by the relationship

$$w_t = w_m([1 - F] + w_s F) \quad (2.10)$$

where F is the fraction of the stones to all particles by dry weight (expressed as a decimal) and w_s is the water absorption content of the stones. According to Winter *et al.* (1998), w_s is often taken to be zero or a very small value.

Removing the oversize material during testing changes the water content and dry density as well as the particle size distribution. If the performance of the fill is to be measured in any of these terms, care should be taken to see that the specification refers to the relevant property; for example, w_m rather than w_t , or ρ_m rather than ρ_t . Likewise, if corrections are made to the tests at the design stage, the identical correction methods should be employed during control testing. It is essential that like be compared with like.

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Chapter 3

An introduction to fill classification and a commentary on some characteristics of fine (cohesive) fills

3.1. Soil description and fill classification

3.1.1 Introduction

Central to selecting an appropriate soil for fill purposes is an accurate description and classification of the soil along with an understanding of the nature of the engineering required. In the UK this process is generally thought of under the overall process of ‘soil description’.

The description of soils is generally undertaken in accordance with BS EN ISO 14688-1 (BSI, 2002), based on field assessments of the material. In addition, Amendment 2 was incorporated into BS 5930 (BSI, 1999) in 2010 to align this document with current standards for field descriptions. The basis of soil description from the updated BS 5930 is illustrated in Figure 3.1.

Following laboratory testing procedures to determine the particulate grading, plasticity and any strength properties of the material, ‘soil classification’ is undertaken in accordance with BS EN ISO 14688-2 (BSI, 2004a). The description-based approach has the advantage that the proposed fill can be described and evaluated immediately, without awaiting the results of laboratory tests, which can take several days to complete. This makes the descriptive approach attractive when, for example, a borrow pit has to be extended during construction and immediate decisions are called for. In many cases on an earthworks project the soil description will therefore be more important than soil classification when determining acceptability in the early stages of a project.

When a soil is to be used as a fill material then an additional process of ‘fill classification’ is required. This is a specific classification process to determine how to consider the soil as an earthworks fill material. In the UK, fill classification is undertaken based primarily on laboratory test results, but also with a general consideration of the soil description in order to provide an indication of the nature and origin of the soil or rock (e.g. chalk, mudstone, clay, glacial till, etc.).

3.1.2 Soil description and soil classification

Perhaps the most important assessment in any soil description is deciding whether the fill behaves as a ‘coarse’ soil or ‘fine’ soil. A simple field test is proposed in BS EN 14688-1

Figure 3.1 Identification and description of soils (from BS 5930:1999 + A2:2010 (BSI, 1999))

Soil group	Principal soil type	Particle size: mm	Visual identification	Relative density/consistency Term Field test	Discontinuities	Bedding	Colour	Composite soil types (mixtures of basic soil types)	Minor constituent type	Particle shape	Particle soil type	Minor constituent	Stratum name	
Very coarse soils	1	2	3	4	5	6	7	8	9	10	11	12	13	14
	BOULDERS		Large boulder — 630 Boulder	Only seen complete in pits or exposures Often difficult to recover whole from boulders	None defined Qualitative description of inspection and ease of excavation	Describes spacing of features such as fissures, shear partings, isolated nodules, etc.	Describe thickness of beds in accordance with geological definition	LIGHTNESS Light Dark	For mixtures involving very coarse soils see Clause 41.4.4.2		Very angular Angular	BOULDERS		Name in accordance with published geological maps, memoirs or sheet explanation
	GRAVEL		Coarse — 20 Medium — 6.3 Fine — 2	Easily visible to naked eye; particle shape can be described; grading can be ascertained	Fissured	Inter-bedded or laminated	CHROMA Pinkish Yellowish Orange Greenish Greyish	Term before principal soil type Slightly sandy See Note B (Sandy) See Note B SAND AND GRAVEL	Proportion secondary soil type <5% 2-25% See Note C 25% See Note C About 50%	Well rounded Cubic Flat Elongate	GRAVEL SAND	For example RIVER TERRACE DEPOSITS GLACIAL SAND AND GRAVEL MADE GROUND		
SAND														
	SILT		Coarse — 0.02 Medium — 0.0063 Fine — 0.002	Only coarse silt visible with hand lens; exhibits little plasticity and is not sticky to the touch; clay lumps in water; lumps dry quickly, and powder easily between fingers	Scale of spacing of discontinuities Term Mean spacing: mm Very widely >2000 Widely 2000-600 Medium 600-200 Closely 200-60 Very closely 60-20 Extremely closely 20-6	Scale of bedding thickness Term Mean thickness: mm Very thickly >2000 Thickly 2000-600 Medium 600-200 Thinly 200-60 Bedded 60-20 Thinly laminated 20-6 Laminated <6	HUE Red Yellow Orange Brown Black	Terms and to fine constituents sustained difference from HCl Silty CLAY Clayey SILT	Terms can include: with occasional frequent/abundant Proportions defined on a site or material specific basis, or subjectively Proportions defined on a site or material specific basis, or subjectively Calcareous - clear but not sustained difference from HCl Highly calcareous - strong and sustained difference from HCl	SILT CLAY				
CLAY													Fine soils (over about 55% silt and clay size)	Only coarse silt visible with hand lens; exhibits little plasticity and is not sticky to the touch; clay lumps in water; lumps dry quickly, and powder easily between fingers
	Condition	Firm	Fibres compressed together Fibrous peat Non compressible, open structure Pseudo-floous peat Amorphous peat Gytis Humus	Accumulated in situ Predominantly plant remains, usually dark brown or black in colour, distinctive smell, low bulk density, can include disseminated or discrete inorganic particles Plant remains recognisable and retain some strength; water and no solids on squeezing No recognisable plant remains; mushy consistency; peat and >50% solids on squeezing Decomposed plant and animal remains; may contain inorganic particles Remains of plants, organisms and secretions with inorganic particles	Relative density/consistency Term Field test	Discontinuities	Bedding	Colour	Composite soil types (mixtures of basic soil types)	Minor constituent type	Particle shape			
Spongy												Plastic	Organic soils	A Percentage coarse or fine soil assessed excluding cobbles and boulders B Gravelly or sandy and/or silty or clayey C Or described as fine soil depending on mass behaviour D Gravelly and/or sandy E Or described as coarse soil depending on mass behaviour F Gravelly or sandy
	Example descriptions	Loose brown very sandy subangular fine to coarse flint GRAVEL with small pockets (up to 30 mm) of clay (TERRACE GRAVELS) Medium dense light brown gravelly clayey fine SAND. Gravel is fine (GLACIAL DEPOSITS) Silt deeply weathered medium strength orange mottled brown slightly sandy slightly gravelly CLAY Firm thick laminated grey silty CLAY with closely spaced thick laminae of sand (ALLUVIUM) Plastic brown clayey amorphous PEAT	Firm thick laminated grey silty CLAY with closely spaced thick laminae of sand (ALLUVIUM) Plastic brown clayey amorphous PEAT											

(BSI, 2002) whereby a sample of the wet soil is remoulded, following the removal of cobbles and boulders (i.e. particles >63 mm). If the soil sticks together and behaves in a plastic manner during remoulding, then it is described as a 'fine' soil (primary fraction being clay and/or silt). Alternatively, if the material behaves in a coarse, non-plastic manner, then it is described as a 'coarse' soil (sand and/or gravel). The boundary between coarse and fine soils is defined in BS EN 14688-1 (BSI, 2002) and BS 5930 (BSI, 1999) as 0.063 mm (refer to Figure 3.1), which is also the approximate limit of visibility of individual grains to the naked eye.

Another key assessment lies in defining the proportion of fine soil that is required to be present in order for the overall fill body to behave in a plastic manner (i.e. at what proportion of fine soil do the fundamental engineering properties and overall behaviour of the fill change from non-plastic to plastic). BS 5930 (BSI, 1999) reports this boundary as being at 'over about 35% silt and clay sizes' (refer to Figure 3.1), although it recognises that there can be considerable variation in this value. BS EN 14688-1 (BSI, 2002) is also deliberately vague in this respect; Norbury (2010) reports that this is because members of the drafting committee could not agree on a definition at the time of preparation of the standard, with suggested values ranging from between around 35% and 50% fines. BS 5930 (BSI, 1999), however, reports that values as low as 15% have also been known.

It has been described in numerous publications and standards that this boundary value primarily varies as a result of two main factors: the grading of the coarse fraction and the plasticity of the fines. Other properties, however, such as angularity of grains and provenance of the fines (e.g. clay minerals or rock 'flour', etc.), can also be a contributory factor. The theory is perhaps best understood with the assistance of two simple examples.

- A soil comprising 80% coarse gravel and 20% silt and clay is likely to behave as a coarse soil overall as the fines content is insufficient to 'stick the soil together'. The fines will instead only occupy pore space, hence the permeability (and strength) of the soil will be affected, but overall the fill will behave in a non-plastic manner.
- A soil comprising 80% fine sand and 20% silt and clay is likely to behave as a fine soil overall as the fines content will be sufficient to 'stick the soil together' when remoulding in a wet condition.

Fine soils thus need to be described and classified based on how the fill will behave, rather than simply applying a set of predetermined grading criteria. For further detailed discussion on this topic, refer to Norbury (2010).

Furthermore, the use of soils for earthworks requires special consideration and BS 6031 (BSI, 2009) Clause 3.1 states that the engineering properties of the soil will differ between two major cases.

- When a soil is assessed for geotechnical design (e.g. slope stability, settlement or bearing capacity), fine (cohesive) soils are likely to include over 35% fines (this is the soil description and soil classification process).
- When a soil is considered as an engineered fill, all fills with over 15% fines are classified as fine (cohesive), (this is specific to the fill classification process).

A further element of the Eurocode (BS EN 1997-1 (BSI, 2004)) soil description and classification system is the term ‘composite soil’, which applies to soils that are not uniformly graded. BS 6031 (BSI, 2009) Clause 3.1 provides clarification on how to apply this term in the field of earthworks in the way shown in Figure 3.2. Reference should be made to BS 6031 (BSI, 2009) for further details.

Unlike granular soils whose particle sizes and distributions, and to some extent shapes, can be estimated by the naked eye, particle sizes within fine soils cannot readily be identified; consequently, the scope for description, as opposed to classification, with clay fill is much more limited. Many descriptors are applied to the structure or fabric of clays, with such terms as fissured, laminated and slickensided being common. As fabric is partially or totally destroyed during compaction, fabric descriptions are of limited use for clay to be used as engineering fill, although they can be of critical importance for undisturbed clay soils revealed in cuttings or in embankment foundations (see Chapters 11 and 12).

Section 6 of BS 5930 (BSI, 1999) provides a basis for soil description. The way the description is written demonstrates the proportion of particle sizes present: materials written in upper case represent the principal soil constituent (after the removal of any cobbles and boulders) and the lower case qualifiers indicate the nature and amount of any subordinate materials. For example:

slightly sandy CLAY (sand-size particles present up to 35%)

sandy CLAY (sand-size particles present between 35 and 65%)

very sandy CLAY (sand-size particles present in excess of 65%).

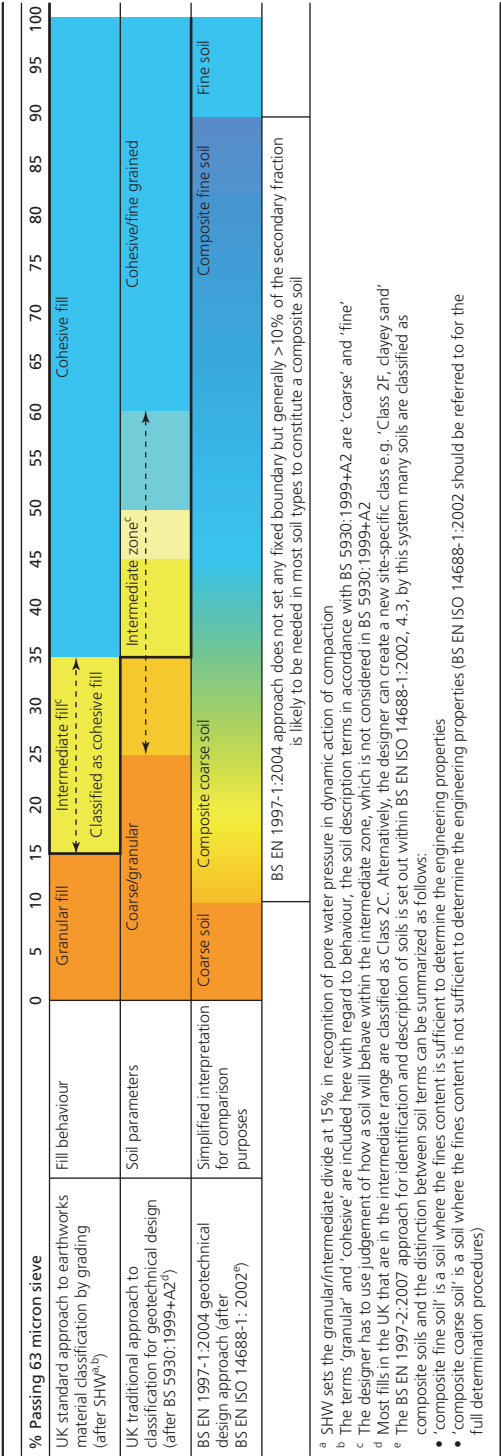
The term ‘gravelly’ is used in the same way when gravel, rather than sand, predominates in the granular fraction. Silts are defined in the same manner: thus a silt containing greater than 65% sand is described as a ‘very sandy SILT’. As already noted, percentage quantities are regarded as a guide and, according to BS 5930 (BSI, 1999), it is the engineering behaviour of the soil mass that determines the description of the soil.

The use of descriptors to indicate the size of the gravel particles, for example ‘fine gravel’ or ‘fine to medium gravel’ is optional. The descriptor ‘coarse’ is helpful because coarse gravel starts at 20 mm, which is the point at which particles are excluded from the BS compaction and moisture condition value (MCV) tests (unless a California bearing ratio (CBR) mould is used for the former). The amount of coarse gravel in any sample of potential fill therefore provides the geotechnical engineer preparing the laboratory test programme with an indication of the nature of the compaction tests that will have to be performed (whether using a standard or a CBR mould or whether the corrections as described in Chapter 2 are necessary).

3.1.3 Fill classification

In the UK, BS 6031 (BSI, 2009) sets the SHW (HA, 1998, 2013) as the default approach for earthworks fill classification. BS 6031 Clause 6.1.5.2 states that:

Figure 3.2 Comparison of soil definitions in different earthworks circumstances (from BS 6031:2009 (BSI, 2009))



‘Earthworks materials should be classified in accordance with Table 6.1 of SHW. The classification of the materials involved during excavation, transportation and deposition can vary, hence soil/fill may be classified at any of the following stages:

- in situ – classification in undisturbed condition prior to excavation;
- on excavation – disturbed material after excavation; and
- on deposition – classification following placing and prior to compaction.

The option for classifying soil/fill should be selected which is most appropriate for the particular project logistics and materials to be worked with, and should maximize the potential to win suitable fill from the site.

Classification should be based on both descriptive determination and standard use of materials, such as SHW Table 6/1.’

3.1.4 Soil description and fill classification terminology for fine soils

At this point it is worth noting that, unfortunately, there is little similarity between BS 5930 (BSI, 1999)/BS EN 14688-1 (BSI, 2002) and the SHW (HA, 1998, 2013) in the way that soils are described and classified. British and European standards do not use the term ‘cohesive’ because it does not accurately describe the properties of fine soils. For example, an unsaturated sand (a coarse soil) can exhibit an apparent cohesion because of the negative pore pressures it contains, while the long-term behaviour of a clay (a fine soil) is controlled by its frictional effective shear strength properties. Therefore, in BS 5930 (BSI, 1999) and BS EN 14688-1 (BSI, 2002), the terms ‘cohesive’ and ‘granular’ are suppressed in favour of the more accurate ‘fine’ and ‘coarse’. Although these objections are scientifically sound, the terms ‘cohesive’ and ‘granular’ are so well established in the earthworks vocabulary, particularly in the SHW, that their removal would appear to offer no particular advantages, and may in fact only increase the confusion that the differences between the descriptive BS 5930 (BSI, 1999) and the classificatory SHW approaches inevitably produce.

BS 6031 (BSI, 2009) refers to the SHW for the approach to earthworks specification and therefore defines fine/cohesive engineered fills (Class 2) as having over 15% fines. Such fills are, however, only rudimentarily subdivided; the names themselves convey only the broadest indication of the nature of the fill (e.g. wet cohesive material, dry cohesive material, stony cohesive material, silty cohesive material). These do not necessarily indicate its likely performance but are used to define the method of compaction to be adopted. Some of the properties involved in classifying fine (cohesive) fill are given in the next section.

3.2. Special characteristics of clay soils

The descriptive term ‘fine soil’ in accordance with the soil description system summarised in Section 3.1 above includes a range of soil types based on the fines content of the soil (percentage finer than 63 μm). However, an additional feature that has a strong influence on the behaviour of a fine soil is the proportion of clay-sized particles (finer than 2 μm) and the mineralogy of those very fine particles. When the soil contains a significant proportion of clay particles (illite, kaolinite, montmorillonite, etc.) then it will tend to show the unique characteristics of clay soils.

There is a wide range of clay soils that vary in particle size distribution and in plasticity characteristics. Text books on soil mechanics (e.g. Knappett and Craig, 2012) demonstrate two types of clay for consideration in British conditions: normally consolidated and over-consolidated clays. Normally consolidated clay exists under effective stresses that have never been exceeded, while over-consolidated clay currently exists under effective stresses lower than those experienced in the past. The reduction of effective stresses could have been caused by erosion, by changes in groundwater level(s) (e.g. due to decommissioning of dewatering systems following cessation of mining activity, or eustatic rise, driven by the melting of ice sheets in the case of glacial materials) or by other geological factors. The 'over-consolidation ratio' may vary, for example lightly over-consolidated or heavily over-consolidated, depending on the change in stress from the original deposition conditions.

Clay materials deposited earlier than the Pleistocene are heavily over-consolidated as a considerable amount of erosion has taken place subsequent to deposition. Common examples in the UK are Eocene London Clay, Cretaceous Gault Clay and Weald Clay and Jurassic Oxford Clay, Kimmeridge Clay and Lias Clay. Glacial clays are over-consolidated as they were deposited below ice sheets some 30 m thick, which subsequently melted.

Lightly over-consolidated clays tend to be those that have been deposited in recent times but have subsequently experienced changes in groundwater level due to eustatic rise, subsequent to the melting of ice sheets, which has resulted in a reduction in applied stress. An example is the Clyde Alluvium in Glasgow, which reaches thicknesses of some 30 m.

Normally consolidated materials are typified by recent river and coastal alluvial deposits, where stress conditions have changed insignificantly since deposition.

Consolidation history has a major impact on the behaviour of the material as fill because it controls both shear strength and water content, with respect to the plastic limit and optimum water content of the soil.

A further problem with over-consolidated clay is that low (sometimes negative) pore pressures may exist, particularly when the clay has been excavated from depth, due to a change of stress state during earthworks. Normal earthworks sequencing will take cut materials from the base of a cutting and place them in the upper layers of an embankment, increasing the likelihood of swelling over the long term. Over-consolidated clay may, therefore, be difficult to work in such circumstances. Further details are provided in Chapter 8.

One issue to be aware of is that not all clay-sized particles comprise clay minerals. For example, aeolian (wind-blown) soils and glacial rock flour can be primarily formed of very fine particles that are actually of quartz mineralogy. The grading of the soil alone can therefore give a misleading indication of the likely behaviour of the material. A further complication is that clay-sized particles may sometimes aggregate to form silt-sized particles, which can exhibit a different overall behaviour, as is the case with Mercia Mudstone. The Atterberg limit tests can help to illustrate these behaviours, but

for some soils it may prove advantageous to consider the mineralogy of the grains as well as the grading and plasticity characteristics of the soil.

3.3. Properties

3.3.1 Plasticity and water content

Plasticity is normally denoted by the plasticity index, which is the difference between the liquid and plastic limits. The plasticity of some British clays is given in the form of a Casagrande chart in Figure 3.3.

The relationship between the water content (w), the plastic limit (PL) and the plasticity index (PI) is defined by the liquidity index (LI), where

$$LI = \frac{w - PL}{PI} \quad (3.1)$$

The liquidity index is thus a measure of the water content of a clay material with respect to its plasticity characteristics. In practical earthworks conditions, the water content can vary from close to the liquid limit to several percentage points below its plastic limit. Depending on the type of clay and the conditions at the borrow, a liquidity index varying from close to unity to zero and below (i.e. a negative value) may be expected.

An alternative method of presenting the information given in Equation 3.1 is use of the consistency index (CI), where

$$CI = \frac{LL - w}{PI} \quad (3.2)$$

The advantage of using this expression is that a negative CI is a clear warning that the water content of the fill is greater than the liquid limit of the soil, and that greater difficulty may be expected in handling and compaction.

Figure 3.3 Plasticity relationships for some British clays (from Vaughan *et al.*, 1978)

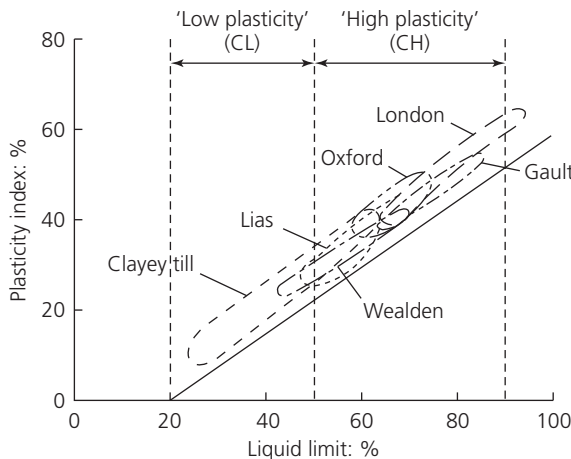


Table 3.1 Atterberg limits of some clay soils (from Lambe and Whitman, 1969)

Clay mineral	Exchangeable ion	Liquid limit: %	Plastic limit: %	Plasticity index: %
Montmorillonite	Na	710	54	656
	K	660	98	562
	Ca	510	81	429
Illite	Na	120	53	67
	K	120	60	60
	Ca	100	45	55
Kaolinite	Na	53	32	21
	K	49	29	20
	Ca	38	27	11

A clay mineral's liquid and plastic limits vary depending on the complexity of its structure, including the nature of the exchangeable ion. Table 3.1 shows data relating to three of the commonest clay minerals, illite, kaolinite and montmorillonite: Illitic clays are probably the most common in the UK, kaolinitic clays less so. The high plasticity montmorillonite is not generally common in British conditions.

From Table 3.1 it could be concluded that where widely varying plasticity results are exhibited by clay soils, the cause is the presence of several different clay minerals, or one clay mineral with several different exchangeable ions. Plasticity results are also significantly influenced by the grain size of the sample tested. The test is performed on a soil sample passing the 425 μm aperture size. The sample can, therefore, contain fine and medium-sized sand grains, and silt- and clay-sized grains. In addition, the nature of the grains may influence the plasticity of the soil. In cohesive soils deposited under water the silt-sized particles are similar to those of clay size in mineral content. In glacial materials the silt-sized particles are commonly 'rock flour', produced by the grinding down of bedrock by the erosive action at the base and sides of an ice sheet.

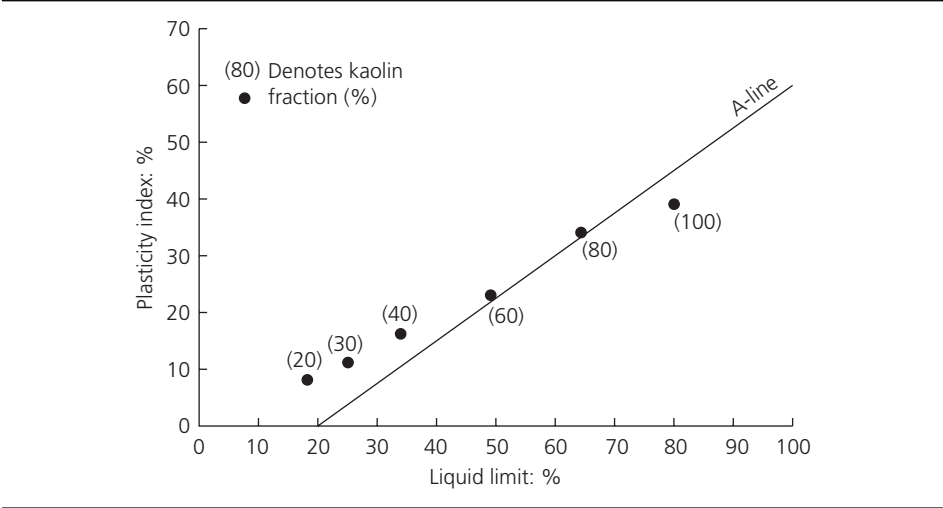
Work by Dumbleton and West (1966) showed that plasticity varies with the granular content of the mixture. Figure 3.4 illustrates this point for various mixtures of a kaolinitic clay and natural quartz sand. They also showed that plasticity varies with the nature of the granular component: whether it comprised well rounded or angular particles, or whether uniform or well graded. Substantial scatter may therefore be expected in the results of any set of plasticity determinations. However, this scatter may be due to the type, nature and amount of the granular material (and silt) present within the otherwise clay soil, rather than to variations in the plasticity characteristics of the clay minerals themselves.

3.3.2 Undrained shear strength

3.3.2.1 Factors affecting the undrained strength of fine (cohesive) fills

Work of major practical importance in understanding the behaviour of clay fill in undrained shear was reported by Vaughan (1978) and Vaughan *et al.* (1978). They

Figure 3.4 Relationship between index properties and clay content for various mixtures of clay and sand (modified after Dumbleton and West, 1966; with adaptations after Trenter, 1999), reproduced by kind permission of CIRIA



distinguished between high plasticity clay (liquid limit in excess of 50%, or CH) and low plasticity clay (liquid limit less than 50%, or CL).

3.3.2.2 High plasticity clay

Vaughan *et al.* (1978) show that at the relatively low rate of loading that controls stability in an embankment, pore pressure redistribution is important and it depends on the compacted structure of the fill. The undrained strength mobilised in the field may be some 20% lower than the corresponding strength measured in the conventional unconsolidated undrained triaxial test (BS 1377:1990: Part 7: Test 8; BSI, 1990). The opposite effect is likely with trafficability (refer to Section 3.4.2). Here the strength mobilised by plastic clay fill on the point of rutting is substantially in excess of the strength measured in the conventional triaxial test, perhaps by a factor of some 50%.

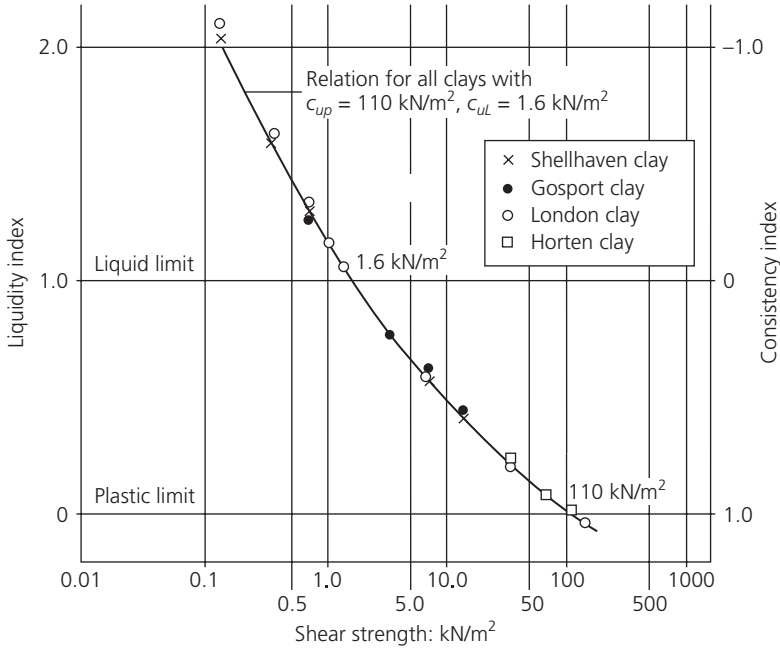
3.3.2.3 Low plasticity clay

The nature of the compacted structure of a sandy clay fill is considered by the authors to have a negligible influence on its undrained strength. Reliable indications of undrained strength in these materials can be obtained by the conventional undrained triaxial test. However, the authors note that if these strengths are employed in embankment stability analysis, unacceptably large deformations may occur at otherwise apparently satisfactory factors of safety.

3.3.2.4 Strength correlations

According to Whyte (1982), there is a simple relationship between the liquidity index and undrained shear strength of remoulded clay soil, as illustrated in Figure 3.5.

Figure 3.5 Variation of remoulded undrained shear strength with consistency index (from Whyte, 1982)



The figure shows that at high water contents (liquidity index close to unity), the undrained shear strength is about 1.6 kN/m^2 ; at low water contents (liquidity index close to zero), the undrained shear strength is about 110 kN/m^2 . From this figure, it is possible to estimate a remoulded undrained shear strength of a clay soil using

$$c_u = 1.6e^{4.23(1 - LI)} \text{ (kN/m}^2\text{)} \quad (3.3)$$

where e is the mathematical constant that is the base of the natural logarithm (≈ 2.71828).

Whyte also proposed the simple relationship between the compression index C_c and plastic index PI , as follows

$$C_c = 1.45PI \quad (3.4)$$

Use of these two expressions permits a relationship between strength, compression index and consistency index (numerically equal to $1 - LI$ and written as a number and not a percentage) as given in Table 3.2.

The plastic limit (necessary to calculate the liquidity and consistency index) is difficult to measure accurately; for this reason, Equations 3.3 and 3.4, together with Table 3.2,

Table 3.2 Suggested relationship between shear strength and consistency

Soil strength classification		Consistency index: $1 - I_f$	Undrained shear strength: kN/m^2
Field description	Strength testing		
Hard	Extremely high	>1.15	>300
Very stiff	Very high	$1.05\text{--}1.15$	$150\text{--}300$
Stiff	High	$0.92\text{--}1.05$	$75\text{--}150$
Firm	Medium	$0.82\text{--}0.92$	$40\text{--}75$
Soft	Low	$0.60\text{--}0.82$	$20\text{--}40$
Very soft	Very low	<0.60	<20

should be used with caution. Note that the above relationships are for remoulded clays, such as clay fill; undisturbed materials that contain structure and/or cementing material are unlikely to behave in the manner described.

It may be undesirable to mix clay and granular soils when forming fill. Diagram (a) in Figure 3.6 illustrates the effect on undrained shear strength of adding sand to a remoulded clay soil while diagram (b) shows the effect of adding gravel. The shear strength curves become steeper (i.e. more sensitive to the addition of water and hence more difficult to employ as a fill) as the granular component in the mixture increases. Vaughan (1978) also concludes that mixing poor fill with a better one is unlikely to yield an effective fill overall. See also Section 10.3.

Dennehy (1978) suggested performing undrained shear strength tests on remoulded samples of fill and plotting the results on a graph of water content against liquid limit

Figure 3.6 Variation of remoulded undrained shear strength with total water content for clay soils with various proportions of granular materials (from Barnes and Staples, 1988): (a) undrained shear strength versus water content, sand added; (b) undrained shear strength versus water content, gravel added

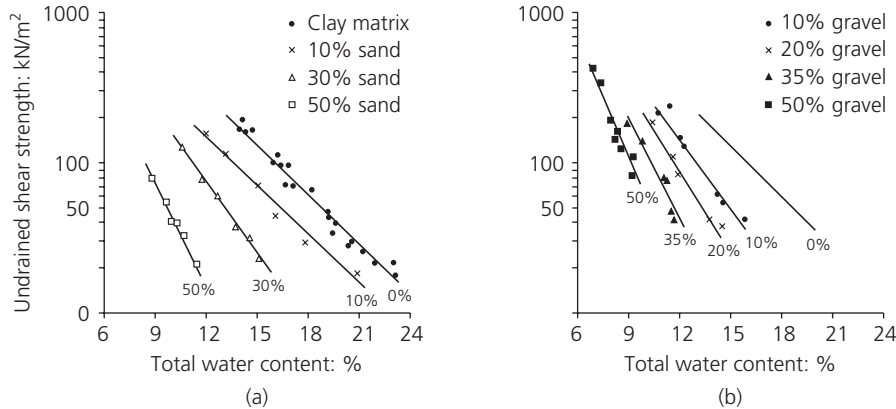
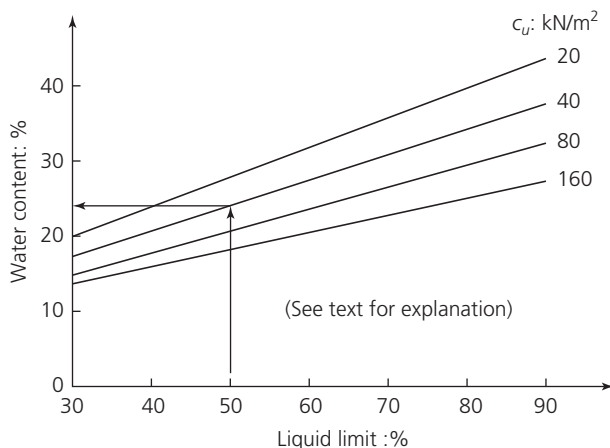


Figure 3.7 Contours of undrained shear strength for various water contents and liquid limits (adapted from Dennehy, 1978)



(Figure 3.7). For fill of a given liquid limit, the shear strength corresponding to the fill water content may be read off the graph. Thus for a fill having a liquid limit of 50%, a water content in excess of about 24% should not be exceeded if a shear strength of 40 kN/m^2 is required for trafficking. For preliminary purposes, Equation 3.3 could conveniently be employed for calculating shear strengths. The above approach neatly demonstrates the effect of plasticity characteristics on trafficability (see Section 3.4.2) and its dependence on weather conditions. Low plasticity clays, including glacial till, can rapidly reduce in remoulded shear strength on mixing with water. Thus, a fill with an appreciably low water content during a dry spell could become untrafficable following a period of heavy rain.

3.4. Behaviour of fine (cohesive) fill: temporary works

3.4.1 General

There are five principal construction problems with fine fills: trafficability, under-compaction, matting, shear surface formation and desiccation. They are influenced by the initial water content of the fill and plasticity characteristics. The likelihood of these issues occurring during the main works should be investigated during compaction trials (see Section 8.5).

3.4.2 Trafficability

A layer (current and previous) may exhibit significant and permanent deformation (rutting) under the weight of compaction or construction plant. The reason is the undrained shear strength of the fill, which is too low for the weight of the plant in operation. Bearing capacity failure occurs, giving rise to the permanent deformation of the fill layer. These features are collectively referred to as 'trafficability', which can be defined as the ability of a specified vehicle to traverse the given terrain.

Methods of improving the trafficability of a fine fill include employing lighter compaction plant, reducing tyre pressures of pneumatic-tyred equipment, and/or drying of the fill by conditioning (see Section 10.5).

The undrained shear strength and MCV of the fill material can be used to inform assessments of trafficability. Further discussion on this topic, including typical requirements for common earthworks plant, is provided in Sections 8.4.4 and 8.4.5, respectively.

3.4.3 Under-compaction

For hard clay, mudstone or siltstone fill, uniform compaction may be difficult if not impossible to achieve at normal borrow pit water contents, and much will be excavated as large chunks. The shear strength of the chunks is likely to be in excess of 150–200 kN/m² (*The Highways Agency Advice Note* HA 44/91 (HA, 1991)), and as a consequence the fill may not be kneaded into a dense, uniform and homogeneous body. The authors' view is that an MCV value greater than 15 would be sufficient for under-compaction but with the development of compaction plant the upper bound value may increase in the future. Corrective treatment would include the procedures outlined in Section 10.5.

3.4.4 Mattressing

'Mattressing' is, for the most part, recoverable deformation, due to high pore pressures set up in the current and previous layers of a fine fill. When mattressing occurs, it is a sign that the fill has been compacted to refusal at its particular water content and that the pore pressures induced by the compaction plant are preventing the fill particles from adopting a closer packing arrangement. The most effective treatment, if still closer packing (higher density) is necessary, is to reduce the water content; using lighter plant will not usually assist because excess pore pressures will still persist and prevent the soil particles from forming a denser packing (see Section 10.5).

3.4.5 Compaction-induced shear surfaces

At water contents considerably wet of optimum, sub-horizontal shear surfaces may be produced in the more plastic clay fills and some weak mudstones by the action of compaction plant (Whyte and Vakalis, 1988). Depending on their continuity, these shear surfaces may hinder vertical drainage and reduce the efficacy of any horizontal drainage layers installed to reduce pore pressures and to increase the stability of wet clay fill (see Section 10.5.6). The surfaces will usually be of lower shear strength than the surrounding fill and may tend to residual strength due to their greater deformation. This could lead to the possibility of the formation of failure surfaces along them within the design life of the earthworks if loading conditions are unfavourable during construction or operation. Vaughan (1994) reports that part of the slide that formed in the weak mudstones comprising the Carsington embankment dam occurred parallel to construction surfaces, and the possibility exists that it coincided with construction-induced shear surfaces in some cases.

Should shear surfaces be noted during construction, the sheared material should be remoulded by appropriate scarification before the next layer is placed. This is especially necessary at the edge of embankment construction, where the possibility of these surfaces

contributing to instability is greatest; if not, a reduction in design strength for the embankment fill may be necessary. Vaughan (1994) also reports that the risk of formation of such surfaces decreases as the layer thickness increases. Compaction using sheepfoot or padfoot rollers may also inhibit their formation as the material is 'kneaded' during compaction (see Section 10.4.2 and Appendix 2).

3.4.6 Desiccation

If a layer of fill is left exposed during periods of dry weather (e.g. there is a significant hiatus in construction during fill placement), then it is likely to lose moisture. This may lead to the development of shrinkage cracks and an increase in the strength of the surface of the fill. Such features are collectively known as 'desiccation'.

If the desiccated layer is not removed prior to recommencing fill placement, then construction difficulties may be experienced because a homogenous fill body will not be achieved. In addition, the surface cracking may provide a means for surface water to infiltrate deeper layers of the fill material or natural subgrade soils, which may result in unexpected softening.

3.5. Behaviour of fine (cohesive) fill: permanent works

3.5.1 Heave and settlement

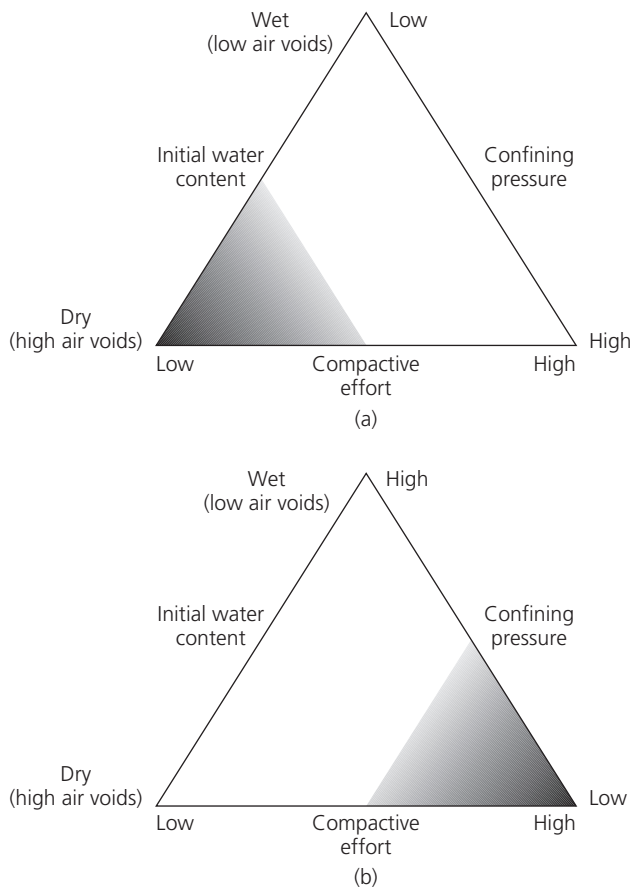
Fine fills can heave and settle just as natural fine soil, depending on external agencies such as climatic variation or vegetation growth. Unlike most naturally occurring fine soils in northwest Europe, fine fills can exhibit 'collapse settlement' if placed dry of optimum. Natural soil exhibits this phenomenon in arid lands because it is frequently unsaturated. In such environments, collapse settlement may occur in response to a rising water table, often provoked by construction works in which proper drainage design was overlooked. It is unusual with natural soils in northwest Europe, which are normally saturated because of the climate. However, the phenomenon is common with some clay fill compacted dry of optimum water content and with some weak rocks (see Section 4.5.3).

Whether a clay fill of given plasticity will exhibit heave or settlement depends on the following factors:

- the initial water content (or air voids)
- the compactive effort applied
- the overburden/confining pressure (i.e. the pressure due to overlying fill and foundation loads).

The relationship between the above factors is summarised in Figure 3.8. For a fine fill of given plasticity, compacted to a low compactive effort and dry of optimum, collapse settlement may be expected at high confining pressures; the trigger mechanism usually being inundation by groundwater rising into the base of the fill (Figure 3.8(a)). If a fine fill of given plasticity is compacted to a high compactive effort, dry of optimum, heave may be expected at low confining pressures; the trigger mechanism again being water (Figure 3.8(b)). Because confining pressures are low, the water source is often shallow depth run off. Further details are presented in Chapter 8.

Figure 3.8 Relationship between critical factors affecting (a) collapse settlement (the darker the shading the more prone is the fill to collapse settlement); and (b) heave (the darker the shading the more prone is the fill to heave) for clay fill of given plasticity



The propensity for swelling or heaving of clay fill increases with plasticity: for this reason, clay soils with a liquid limit in excess of 50% are not recommended as fill in areas where heave could be a problem (see Section 8.3.4). There is a comprehensive discussion of clay heave and settlement in Cox (1978).

3.5.2 Chemical attack by sulfur-bearing minerals

3.5.2.1 General

Sulfur-bearing minerals contained within over-consolidated clay, argillaceous rocks (Chapter 4) and some industrial waste fills (Chapter 6) may release sulfate and sulfide ions under certain conditions, which can have the following deleterious effects on earthworks:

- chemical attack of any buried concrete or other construction materials
- expansive reaction of finished earthworks, with consequences for overlying structures

- volume reduction of finished earthworks, with consequences for overlying structures
- chemical attack of limestone with the possible formation of noxious carbon dioxide.

Many British clays of Tertiary age and older contain significant quantities of sulfur-bearing minerals. The two most common forms in Britain are iron pyrites (iron sulfide) and gypsum (hydrated calcium sulfate) but other more soluble salts such as magnesium and sodium sulfates also occur. Iron pyrites and gypsum may be present as nodules or concretions or they may be finely disseminated within the soil. Leaching often ensures that sulfates are absent in the first metre or so, but concentrate at some 2 or 3 m depth, reducing in concentration at greater depths. The reason for their concentration at this depth interval may also be the oxidation (during weathering) of the sulfides. Therefore, low sulfide contents may be expected to some 3 m depth, but possibly increasing in concentration below. Trial pits (or trial excavations) are the best means of examining the soils in the borrow areas and of mapping sulfur-bearing minerals. Securing representative samples of soil at the borrow for chemical analysis is notoriously difficult because, as noted above, sulfur-bearing minerals may be concentrated at certain depths and locations. Unless there is careful mapping of the trial pits to locate the sulfur-bearing minerals it is easy to arrive at the wrong conclusion on the correct chemical content for design purposes.

Further difficulties may arise if soil samples are not correctly preserved and tested as soon as possible after retrieval from the ground. With time, sulfides in soil samples oxidise with the formation of sulfuric acid, thus lowering the pH value. Hawkins and Wilson (1990) found that over a 5-month period, a significant increase in the laboratory acid soluble sulfate content of certain Lias clay samples occurred, sufficient to raise the aggressivity of some of the samples by one class of the (then) BRE classification. Undesirable chemical changes may also be reduced, if not avoided, by keeping samples in air-tight containers at as low a temperature as practicable until testing is performed. Assuming its presence, groundwater, over time, will have permeated relatively large volumes of soil in the borrow area, thus providing the opportunity for groundwater samples to take up a more representative sulfate content than individual soil samples. Consequently, there is usually less scatter in the groundwater test results. For this reason, greater emphasis is often placed on groundwater test results than on those of the corresponding soils, but care must be taken to see that the groundwater samples themselves are not contaminated by surface water during sampling. This is often difficult to accomplish during conventional cable tool boring, and a more reliable method may be to sample from standpipes, after having air-lifted the water column in the standpipe several times to remove surface water or any water used during boring or drilling. Note that the solubility of magnesium and sodium sulfates is markedly higher than that of calcium sulfate (150 and 250 times more soluble, respectively). Therefore the sulfate concentrations measured in groundwater samples may relate more to the magnesium and sodium sulfates than to the calcium variety.

BRE Special Digest 1 (BRE, 2005) suggests that, when analysing the results of chemical tests on groundwater samples, the characteristic value should be used for design purposes.

When the results of a large number of soil samples are to hand, the Digest recommends that the mean of the highest 20% should be adopted as the characteristic value.

3.5.2.2 Attack of buried construction materials

It has been known for many years in the UK that concretes made with Portland cement are vulnerable to attack by sulfates in the ground. This problem is not exclusive to fine soils but is certainly more common and the effects are often more pronounced than with coarse soils. Guidance on designing concretes to resist conventional sulfate attack was developed in a series of BRE digests, the most recent of which is BRE Special Digest 1 (BRE, 2005). This document is a development of the earlier BRE Digest 363 and defines concrete protection for both greenfield and brownfield sites. Sulfate concentration is generally measured on groundwater or soil/water extract samples to BS 1377:1990 Part 3 (BSI, 1990) supplemented by methods set out in BRE report BR279 (Bowley, 1995).

BRE Special Digest 1 also defines the deterioration of concrete as the result of thaumasite as a separate form of sulfate attack. The distinguishing features of thaumasite sulfate attack are:

- occurs preferentially at low temperatures (below 15°C, such as typically found in the ground)
- requires availability of carbonate ions, as well as sulfate ions, from sources including limestone aggregate, limestone filler in cement and bicarbonate in groundwater
- targets the calcium silicate phases within hardened cement paste, potentially reducing concrete to a mush.

Where material may contain sulfides, the Transport Research Laboratory (TRL) Report 447 (Reid *et al.*, 2001) recommends the determination of total potential sulfate. This is usually determined by X-ray diffraction or scanning electron microscopy methods. This method gives the total volume of sulfide present. This value may, however, be an overestimate with respect to the total volume of sulfide that could potentially oxidise. For a particle size greater than 15 µm only the surface area is potentially available for oxidation rather than the full particle volume.

Special Digest 1 also differentiates the potential for sulfate migration through the ground by definition of 'static' and 'mobile' water. The former is considered to prevail where bulk permeability is less than 10^{-7} m/s.

The SHW (HA, 1998, 2013) requires that fill shall not be deposited within 500 mm of concrete, cement bound materials or stabilised capping layer if:

- the water-soluble sulfate content exceeds 2.3 g of sulfate (as SO₄) per litre (Test 1, TRL Report 447), or
- the oxidisable sulfides content exceeds 0.46% of sulfide (as SO₄) (Tests 2 and 4, TRL Report 447), or

- the total potential sulfate content exceeds 0.6% of sulfate (as SO_4) (Test 4, TRL Report 447).

Similarly, fill shall not be deposited within 500 mm of metallic items forming part of the permanent works if:

- the water-soluble sulfate content exceeds 0.3 g of sulfate (as SO_4) per litre (Test 1, TRL Report 447), or
- the oxidisable sulfides content exceeds 0.06% of sulfide (as SO_4) (Tests 2 and 4, TRL Report 447).

3.5.2.3 Expansive reaction of finished earthworks

Any fills containing sulfur-bearing minerals, which could undergo expansive reaction in-service are suspect; they include some Tertiary and older clays, pyritic shales, colliery discard and some of the other materials discussed in Chapter 6 (e.g. steel slag). Swelling of coal shale fill (by up to 70 mm) has been attributed to the expansive alteration of sulfides to sulfates by Caldwell *et al.* (1984). Similar problems occur with soils or weak rocks in situ. Quigley *et al.* (1973) described how an area of black pyritic shale (above the water table and near heating ducts) heaved up to 76 mm, while other areas below the water table (and consequently out of the oxidising environment) showed no movement. The opportunity for oxidation (by excavation during basement construction) and heating (by the presence of boilers in basements) appears to play a major part in expansive reactions, and situations such as these are typical of some domestic, commercial and industrial structures. Quigley *et al.* (1973) also noted that heave of black pyritic shales was greatly reduced by gunite or bitumen coatings, which helped to prevent oxidation of the pyrite. Hawkins and Pinches (1987) recorded structural damage to a hospital due to gypsum growth in black pyritic mudstones.

Czerewko *et al.* (2011) discuss the rapid formation of selenite from sulfide-rich Lias Group mudstones on the A46 Newark to Widmerpool Improvement Scheme. Material was stockpiled to allow oxidation, crushed and then heavily compacted, using a sheepfoot roller, to produce a dense, low permeability fill to prevent further significant oxidation.

Lime-stabilised soils are among those most at risk from volume changes of this sort. Sulfate and lime react in an expansive manner causing the stabilised layer to heave or crack. Snedker (1996) describes how heave of up to 60% of the initial layer thickness and an increase in water content to some 60% occurred for Lower Lias clays and mudstones. Proposals to use lime stabilisation in clay fill from any Tertiary age or older clays, or any industrial fill should be accompanied by thorough investigation of the sulfur-bearing minerals present. Investigation should also determine whether fresh sulfate growth could be produced by the changing conditions (particularly oxidation) caused by construction itself.

3.5.2.4 Volume reduction of finished earthworks

Volume reductions can occur due to the dissolution of calcite (or other soluble carbonate) in argillaceous rocks containing pyrite in finely divided form. Following oxidation of

the pyrite, the carbonates are attacked with volume reduction as a consequence. Pye and Miller (1990) believe that there was a volume loss of up to 10% of part of a Derbyshire dam, constructed of Carboniferous mudstones, which could have occurred in this way. Although collapse settlement is a possible alternative explanation, it would nevertheless be prudent to keep the possibility of such occurrences in mind when using fill of the type that Pye and Miller describe.

3.5.2.5 Attack of limestone

Limestone is frequently employed as rock fill, capping layers and drainage blankets and surrounds. As such, it may come into contact with fills containing sulfur minerals. The chemical reaction between oxidising sulfur minerals and the limestone could produce carbon dioxide in significant quantities. In most circumstances, the carbon dioxide vents to the atmosphere and little harm is done, but there have been tragic fatalities, such as those in inspection chambers at the downstream toe during construction of the Carsington dam in Derbyshire, UK (Bromhead, 1992).

Great care should be taken where limestone and sulfur-bearing fills are concerned. A risk assessment of the consequences to operatives and maintenance workers should always be undertaken, particularly with man-access points to underground works.

3.5.3 Frost-heave susceptibility

According to Croney and Jacobs (1967), clay fill may be regarded as being non-frost-heave susceptible provided that its plasticity index exceeds 15% for well drained materials (groundwater greater than 600 mm from the formation) or 20% for poorly drained materials (groundwater within 600 mm of the formation). They also noted that the susceptibility of clay fill to frost heave decreased as the degree of compaction increased.

Further guidance on the potential for frost heave is provided by BS EN ISO 13793:2001 (BSI, 2001).

Frost heave has been recorded in lime stabilised fill, depending on the fill type, the amount of lime employed and the stage in the treatment (see Section 10.5.2).

3.6. Clay fill as landfill liner

3.6.1 Required properties

There has been heightened interest in the use of clay fill for landfill liners in the past decade because it provides the low permeability necessary to restrict leachate egress. There is also evidence that the clay minerals in the liner absorb certain ions from the leachate, depending on the pH of the system, with an alkaline environment being beneficial. Tests with the organic chemicals that may occur in leachates are reported to bring about clay mineral flocculation and an increase in liner permeability; however, most tests use much higher concentrations of chemicals than occur in practice in domestic leachates and consequently the representativeness of these test results may be questioned.

There are limits for the suitability of clays used in landfill liners and they were brought together by Murray (1998) in the following.

Table 3.3 Engineering property limits for clay fill for landfill liner purposes (based on Murray, 1998)

Engineering property	Reference	Limit/criterion
Plasticity	DETR (1995)	30% > plasticity index > 10%
	Daniel (1993)	Plasticity index > 7–10%
	EA (2013)	Liquid limit < 90%
		Plasticity index < 65%
	Murray <i>et al.</i> (1992)	Plasticity index > 12%
	Gordon (1987)	Plasticity index > 15%
	Williams (1987)	Plasticity index > 15%
Percentage fines	Daniel (1993)	Clay and silt > 20–30%
	EA (2013)	Clay > 10%
	Gordon (1987)	Clay and silt > 50%
Activity	DETR (1995)	> 0.3
Percentage gravel	Daniel (1993)	Gravel (> 4.8 mm) < 30%
Maximum particle size	Daniel (1993)	< 25–30 mm

Table 3.3 shows that the use of a plasticity index above the range 12–15% would appear appropriate for landfill liners in the UK. There is differing advice for fines content (<0.063 mm), varying from at least 20% to 50% and over. The minimum percentage clay (<0.002 mm) is 10% according to the Environment Agency (EA), which should be compared with an activity of greater than 0.3 recommended by the Department of the Environment, Transport and the Regions (DETR); assuming a plasticity index of 15%, this implies a clay content above 50%, substantially in excess of the EA figure. Murray (1998) reports that permeability of clay fills increases rapidly at a gravel content of about 50–60%.

According to Privett *et al.* (1996), the United States Environmental Protection Agency (USEPA, 1991) requires a suite of soil properties in order to achieve a minimum permeability of 10^{-9} m/s, as follows:

- percentage fines (passing the 0.075 mm sieve) $\geq 20\%$
- plasticity index $\geq 10\%$
- percentage gravel $\leq 30\%$
- maximum particle size 50 mm.

Note that the minimum percentage fines recommended by USEPA is expressed in terms of the 0.075 mm sieve, rather than the 0.063 mm sieve. Depending on the actual particle size distribution, this means that the USEPA requirement of a minimum of 20% fines becomes a minimum of roughly 16% fines using the alternative definition. The plasticity index is also a little low, when set against the recommendations of Murray *et al.* (1992), Gordon (1987) and Williams (1987). The percentage gravel accords with the

recommendation of Daniel (1993), although the maximum particle size is larger (50 mm against 30 mm). Site and laboratory trials should help to determine which of the above specifications to adopt for the particular circumstances at hand.

Murray *et al.* (1992) demonstrated that the permeability of clay fill at optimum water content (OWC) decreased as the plasticity index increased. A maximum permeability of 10^{-9} m/s is often specified (EA, 2013) and they found that clay fill having a plasticity index lower than 12% could not be relied on to produce permeability values of this magnitude (see Table 3.3). They also showed that permeability decreased as OWC increased.

In some cases, clay soils at the borrow within economic haul distance do not have the desired properties for liner formation and the use of bentonite enhanced sands (BESs) as a liner material may be considered. The ability to add sodium (or sodium-activated) bentonite in varying proportions (usually 5–20%) means that the important properties of strength and permeability may be ‘tailor made’ for the particular engineering requirements of the facility. Kraus *et al.* (1997) determined that BES is less frost susceptible than compacted clay and according to Dixon *et al.* (1985) suffered less shrinkage on drying.

A study of the engineering properties of BES has been reported by Stewart *et al.* (1999).

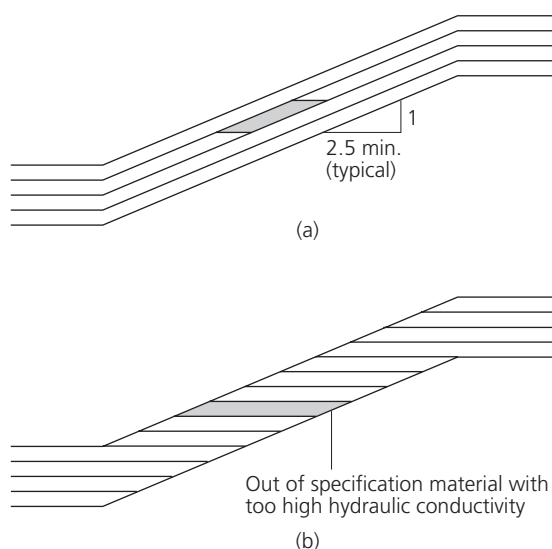
3.6.2 Aspects of liner construction

Murray (1998) suggested compacting to the 4.5 kg rammer specification for low plasticity clays and to the 2.5 kg rammer for high plasticity clays, if the desired permeability characteristics (usually 10^{-9} m/s) were to be met. As far as practicable, clay layers for landfill liners should be placed in parallel rather than horizontal lifts (Figure 3.9). In this way, a defective layer will have no immediate access for leachate migration. Privett *et al.* (1996) recommended 1 (vertical) in 3 (horizontal) side slopes for landfill liners but, in practice, the characteristics of the compaction plant as well as the fill must be taken into account in determining the side slopes that can be safely and effectively placed and compacted.

As noted, over-consolidated clays such as London, Gault and Lias and weak rocks such as shales and mudstones may be particularly difficult to compact. Recent work suggests that water or permeant flows between ‘clods’ of hard, poorly compacted, cohesive fill. The more these clods are broken down, the more homogeneous and uniform the fill will be and the lower the overall fill permeability. A rotavator can reduce clods to more manageable sizes (see Section 10.5.3). Compaction using thin layers may then produce a more uniform, homogeneous and impermeable fill.

Settlement or heave of the liner should be considered. Settlement of a soft clay foundation could be sufficient to disrupt the integrity of the basal clay liner under the weight of the overlying landfill, causing cracking and permitting leachate egress. Where present, artesian or sub-artesian water pressures could lift the basal clay liner, particularly at the early stages of operation and before the weight of landfill increases the effective stress on the liner foundation, again with cracking as a consequence. Liner sides should also be checked. Failure mechanisms for steep-sided liners, such as would be employed to line

Figure 3.9 Placing fill for landfill liners: (a) defective material is confined in slope-parallel construction method; (b) defective material may provide preferential leakage path in horizontal lift construction method (from Privett *et al.*, 1996)



existing quarries or pits, include, for example, bulging of the liner into the fill. Bearing capacity failure of a soft soil foundation supporting the liner sides is also possible. Both mechanisms could cause cracking of the liner. Cracking could also result from the differential movement between the liner side wall and the waste contained within it. For a more complete discussion, see Edelmann *et al.* (1999).

As with any earth built water-retaining structure, clay liners can erode. Fills most at risk are silts and uniform silty sands; clays of low plasticity such as some glacial tills, dry of optimum, and weathered argillaceous rocks such as siltstones and silty mudstones. Dispersivity tests should be performed to assess the suitability of the material (Murray, 1998, Table A1 and BS 1377:1990: Part 5: Test 6 (BSI, 1990)).

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Chapter 4

Some characteristics of coarse (granular) and weak rock fill

4.1. Introduction

Naturally occurring granular materials tend to have been derived from a water-borne origin (river, marine or glacial melt-water) or wind-blown origin. Recent materials tend to be uncemented but, in a desert origin, cemented 'hard pans' may have formed layers close to the surface.

In the UK, geological strata younger than Permian age tend to be un-cemented although some Cretaceous strata, such as the Greensand, Hythe Beds and Folkestone Beds may contain cemented layers up to 500 mm thick that, historically, were used as building stone locally.

Coarse (granular) soils are normally free draining and easier to handle, in an earthworks context, than fine (cohesive) soils. Apart from uniformly graded sands, silty sands, and some weak rocks, commented on further below, they are relatively trouble free when employed as fill.

Weak rocks are defined here as material falling within the 'ripping' range shown in Figure 9.1 (Chapter 9) although, in some cases, material falling within 'hard digging' would also qualify. The latter would apply to the strata of Permian and Triassic age in the UK. Harder chalks (see Section 5.3) may also be classified as weak rocks. For rocks older than Carboniferous age and igneous and metamorphic rocks ripping is likely to be efficient only in the weathered profile of those rocks. Slightly weathered or fresh rock would require blasting.

4.2. Soil description and fill classification

4.2.1 General

Coarse (granular) soils are defined in BS EN ISO 14688-1 (BSI, 2002a) as those that exhibit non-plastic behaviour during the re-moulding of a moistened sample, following the removal of cobbles (63–200 mm) and boulders (larger than 200 mm). As reported in Section 3.1, BS 5930 (BSI, 1999) describes coarse soils as being 'over about' 65% coarser than the 0.063 mm particle size (i.e. sand (0.063–2 mm) and gravel (2–63 mm)). Therefore, in terms of soil description, coarse soils will typically contain less than approximately 35% fines (<0.063 mm), although the actual value

will vary depending on the grading of the coarse fraction and the plasticity of the fines.

In terms of earthworks fill classification, the SHW (HA, 1998, 2013) defines material with less than 15% fines as ‘granular fill’. Consequently there is no direct correlation between the descriptive ‘coarse soils’ provided in BS 5930 (BSI, 1999)/BS EN ISO 14688-1 (BSI, 2002a) and the fill classificatory ‘granular fill’ referred to in the SHW (HA, 1998, 2013).

Refer to Section 3.1 for further discussion regarding soil description and earthworks fill classification.

4.2.2 Description

A satisfactory description for a granular soil destined to be used as a fill should contain at least:

- the distribution of particle sizes
- the shapes of gravel particles
- an approximation of the mineralogy of the particles.

As with fine soils, Section 6 of BS 5930 (BSI, 1999) provides a basis for such estimates. The way that the description is formulated indicates the proportion of particle sizes present: materials written in upper case constitute more than 65% of the total (not including any cobbles and boulders) and the lower case qualifiers indicate the nature and amount of any subordinate materials.

For example:

- slightly sandy GRAVEL (sand-size particles present up to 5%)
- sandy GRAVEL (sand-size particles present between 5% and 20%)
- very sandy GRAVEL (sand-size particles present in excess of 20%).

Again, and as discussed in Section 3.1, the percentage figures are qualified in BS 5930 (BSI, 1999) by the mass behaviour of the soil.

An estimate of the particle size distribution of a granular soil can be made by attaching the qualifiers ‘well graded’, ‘uniformly graded’ and ‘gap graded’ to the above. ‘Well graded’ means particles reasonably evenly distributed throughout the particle size range; ‘uniformly graded’ has the overwhelming proportion of its distribution in one (or more) restricted size; and ‘gap graded’ has a marked lack of particles in one or more size range.

For example:

- sandy GRAVEL; well graded (material roughly equally distributed between the sand and gravel sizes)

- slightly silty fine SAND; uniformly graded (sand restricted to the fine sand range, containing up to 5% silt).

Although adopted in *Soil Mechanics for Road Engineers* (Road Research Laboratory, 1952) and the previous edition of BS 5930 (BSI, 1981), the qualifiers ‘well graded’, and so on, are not used in BS 5930 (BSI, 1999) because it is considered that there may be a misunderstanding of the terms. Their use is retained in this book because they provide a link to the SHW (HA, 1998, 2013), which also uses these qualifiers. Nevertheless, the geotechnical engineer or geologist should be aware of the possibility of confusion and should always define terms clearly, if necessary by reference to Figure 4.1. The particle shape is valuable information that may readily be obtained for most granular soils using a hand lens. Section 6 of BS 5930 (BSI, 1999) provides examples of both angularity and form.

4.2.3 Classification

The alternative to the descriptive approach is given in the SHW (HA, 1998, 2013), in which coarse (granular) soils are classified in terms of certain laboratory tests. Acceptable limits are quoted within which the results must fall. The soils are subdivided into classes and the names of the classes themselves are general, conveying only a broad indication of the nature of the fill (e.g. well-graded granular material, uniformly graded granular material, coarse granular material, etc.). Uses for the fill classes are also given in general terms.

As already noted, one of the major differences between the descriptive approach of BS 5930 (BSI, 1999) and the classificatory approach of the SHW (HA, 1998, 2013) is the difference between the limits adopted for ‘coarse soils’ and ‘granular fill’, that is, maximum of about 35% and 15% fines, respectively. The different maximum fines content adopted in the SHW is a response to the fact that, at fines contents greater than about 15%, the soil is often no longer free draining.

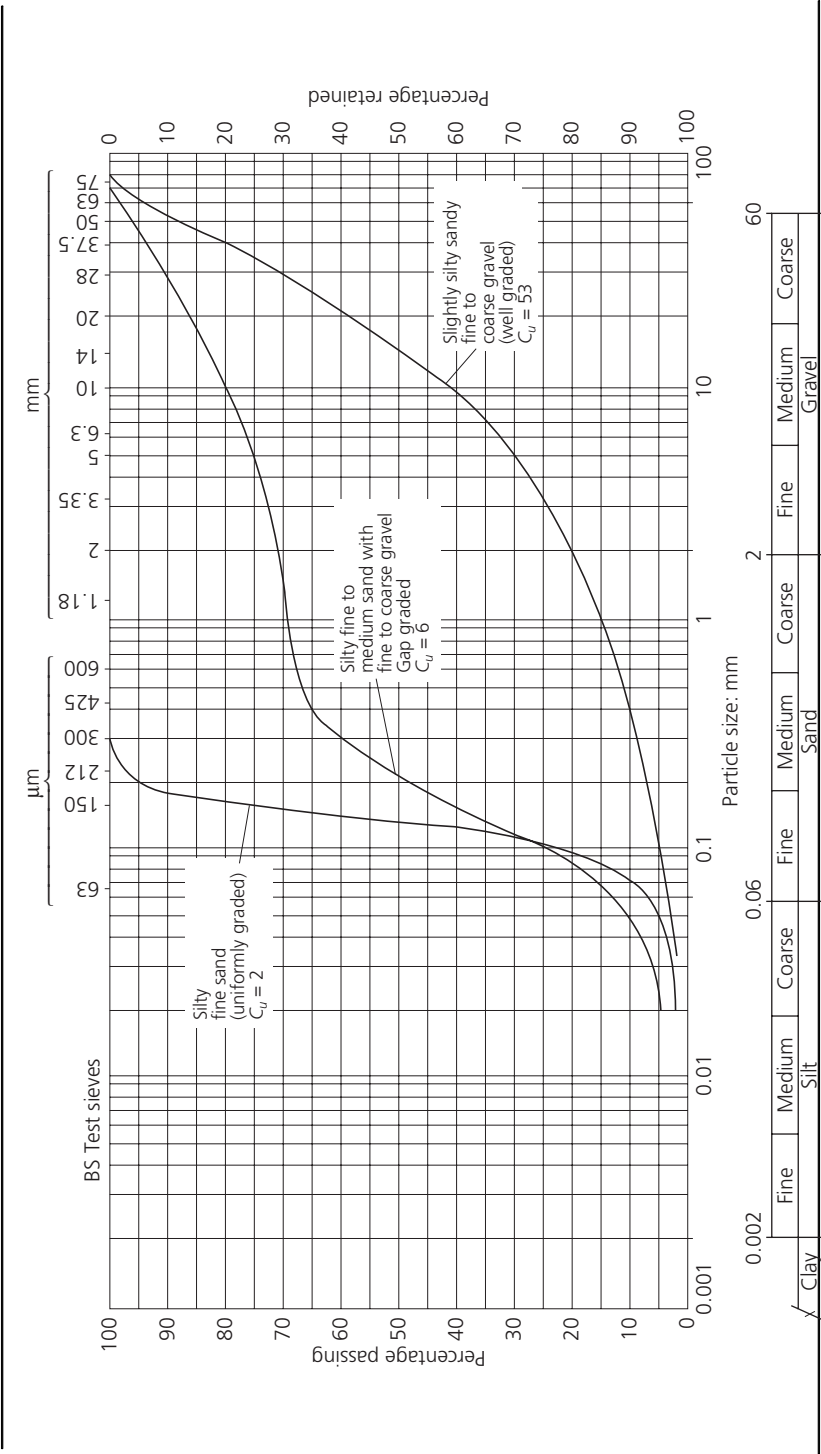
An important classifier used in the SHW (HA, 1998, 2013) is the uniformity coefficient of the soil, C_u (previously designated U_c)

$$C_u = D_{60}/D_{10} \quad (4.1)$$

Reference to Figure 4.1 demonstrates limitations to the use of C_u : the gap-graded nature of the particle size distribution of the example illustrated cannot be identified, although it could have a significant effect on its performance as a fill. Consequently, C_u has to be used with care. A visual description of a representative hand specimen made by an experienced geotechnical engineer or engineering geologist can often be at least as useful a device as the uniformity coefficient.

The approach to adopt, whether descriptive or classificatory, depends on the nature of the context of the engineering situation, as described in Section 3.1.2 and BS 6031 (BSI, 2009b). If the SHW (HA, 1998, 2013) is adopted, the classificatory approach must also be followed.

Figure 4.1 Particle size distribution of granular soils and the terminology used to describe them (from BSI, 1981)



Note: boundaries at 0.06mm and 60 mm are now set at 0.063 mm and 63 mm in BS 5930 (BSI, 1999).

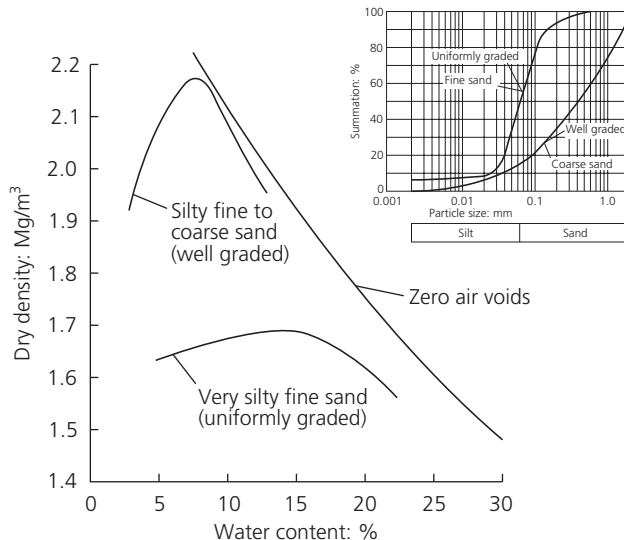
4.3. Properties

4.3.1 Particle size distribution and shape

With respect to use as engineering fill the following points should be considered.

- Well-graded granular fill is likely to possess the best engineering properties for most applications. Abundant particle contacts reduce inter-particle stresses and accompanying breakage; furthermore, by filling void spaces, the smaller particles help to increase density.
- Well-graded granular fill materials with sub-rounded to rounded particle shape, such as river or marine-derived gravels, are usually more difficult to compact than those of similar particle size distribution with angular to sub-angular particle shape, such as rock-derived material, as the mechanical interlock between the grains is more difficult to achieve on compaction.
- Uniformly graded fill has relatively few particle contacts and generally demonstrates poorer engineering properties than its well-graded equivalent. This is illustrated in Figure 4.2, where there is a markedly lower density for a given water content and compactive effort.
- Well-graded granular fill materials with a significant fines content are more likely to generate high pore water pressures if compacted wet of the optimum water content as they are of lower permeability than uniformly graded granular materials. This can lead to difficulty in applying the necessary compactive effort and is likely to lead to loss of strength.
- Gap-graded fill excavated from some Glacial Till may not always possess the mix of particles necessary for the best engineering performance.

Figure 4.2 Dry density against water content relationships for well-graded and uniformly graded granular soils (from Road Research Laboratory, 1952)



- For rocks, Vaughan (1994) describes how ripping fills the interstices between the larger particles with smaller grains, forming a relatively well-graded fill with abundant particle contacts. Blasting produces a more uniformly graded assortment of blocks or particles, with a relatively small component of fines, and a more uniform fill. Vaughan referred to these two types as 'filled' and 'underfilled', respectively. The 'filled' and 'underfilled' materials may be expected to have characteristics broadly similar to those of 'well-graded' and 'uniformly graded' granular fills. Because of the high stresses produced at the few particle contacts, underfilled material may not always reflect the higher strength and hardness of the rock from which it was composed.

4.3.2 Effect of large particle sizes

Section 2.6 described methods of correcting for oversize particles or stones (greater than 20 mm) when determining dry density in the standard compaction test mould. However, there is still a problem in determining how strength and compressibility properties of granular fill are affected by oversize material. Winter and Suhardi (1993) suggest that provided the proportion of the soil having particles larger than 20 mm is smaller than between about 45% and 50% of the total (F less than 0.45 or 0.5 in Section 2.6), then the properties of the matrix (i.e. the material finer than the stones) will control the behaviour of the fill as a whole.

If the proportion of the stones rises significantly above 45–50% of the total, then the results of strength and compressibility tests may not be representative and some form of large scale testing, including in situ test equipment built for the purpose, may be necessary. This was done by Bolton and Lee (1993) who demonstrated that for a given relative density, the angle of shearing resistance (ϕ') increased as the maximum particle size increased. It was suggested that large particle sizes acted as reinforcement as the smaller particles flowed around them. Large scale tests are seldom practicable for most projects and the results of tests on matrix materials or correlations (see below) are normally accepted for design purposes.

The work of Winter and Suhardi (1993) also showed that, as the stones increased above the 45–50% level, the density began to decrease. This might seem at first sight anomalous in that an increasing content of coarse particle sizes might be expected to result in increasing, not decreasing, densities. However, for large particles with limited fines content there are fewer particle contacts and hence there is greater opportunity for the large particles to rub together and even crush. Much of the energy applied during compaction is lost due to the friction between, and crushing of, the larger particles and therefore a smaller proportion of energy is available to reduce the void spaces between the particles. Substantially heavier equipment and/or more passes and/or thinner fill layers are necessary to secure high levels of compaction in coarse granular fills than in their finer grained, or matrix dominant, counterparts. Recognition of this fact means that many specifications require a maximum particle size no greater than two-thirds of the layer thickness in order to reduce the amount of compactive effort absorbed in handling the large particle sizes.

Mixing fills is seldom advantageous with soils (see Section 3.3.2), but Marsal and Fuentes de la Rosa (1976) demonstrated that the addition of finer granular material ranging from 20% to 50% by weight to an otherwise uniform basalt rock fill improved compaction

performance significantly under vibrating rollers. They also found that when some 30–40% of finer granular material was added to the rock fill, grain breakage and compressibility were reduced.

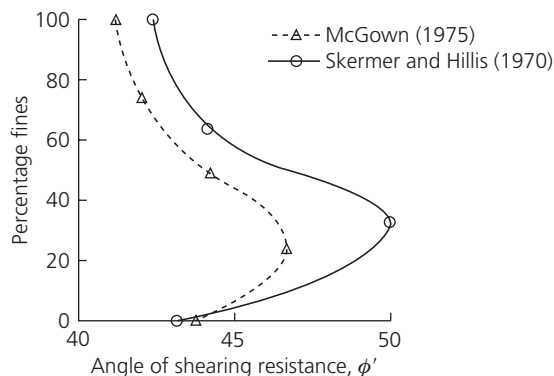
Care should, however, be exercised when the performance of a mixed fill is considered. Although the mixing of a finer grained granular material into a coarse grained material may achieve better compaction and density of bulk engineering fill, the use of a mixed material may result in segregation of fine particles if used as a running surface, with a subsequent loss of relative density and strength. This is particularly relevant in the context of unpaved roads and temporary haul roads.

Also, the addition of coarse material to a fine grained material to ‘produce’ a better material to the SHW should be avoided as this usually results in gap-graded material with no additional strength benefit to the parent material. An example is the use of Thanet Sand, of Eocene age, which is a uniformly graded silty fine sand. Historically, single size crushed rock material has been added to convert a Class 1B, uniformly graded granular fill, to the SHW, to a Class 6, special granular fill material, based on grading. The addition of the single size material did not, however, improve the material strength and the fill performed as discrete core stones in a predominantly silty sand matrix.

4.3.3 Shear strength

McGown (1975) suggested that there was an optimum proportion of fines (less than the 0.063 mm particle size), which would produce the minimum void ratio and the maximum angle of shearing resistance (ϕ') for granular fill; work by McGown and others indicated that this optimum proportion varied from about 20% to 40% (Figure 4.3). Similar results were established by Irfan and Tang (1993) who tested Hong Kong soils; these authors found that shear strength increased sharply if the coarse fraction increased above about 30%. This suggests in turn that provided coarse material does not amount to more than say 25% of the test specimen, its removal should not significantly affect the measured shear strength of the remainder.

Figure 4.3 Variation of angle of shearing resistance (ϕ') with percentage fines less than 0.063 mm (from McGown, 1975)



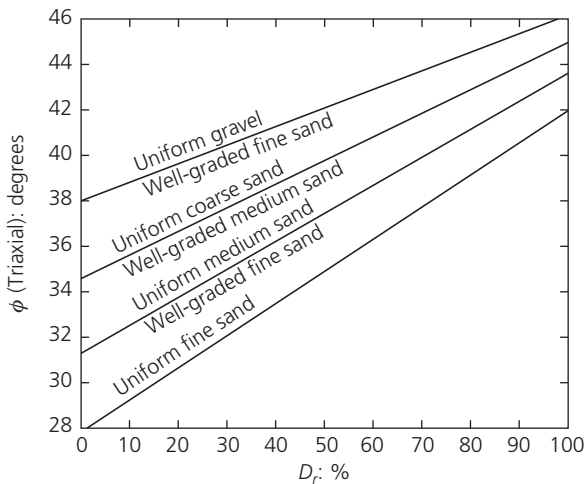
Most shear strength testing devices available in commercial laboratories are capable of testing triaxial specimens no larger than 100 mm in diameter and 300 mm square for the shear box. A ratio specimen size to maximum particle size no smaller than 6 is usually regarded as being appropriate for shear strength testing purposes. This is approximately the figure adopted by Irfan and Tang in their tests. For such a ratio, the maximum particle sizes that can be tested as routine in commercial testing laboratories are small, even allowing for the removal of a proportion of the coarser soils. For this reason, when designing granular fill for strength purposes, correlations between shear strength and properties such as the relative density are usually relied on.

Relative density D_r may be expressed as

$$D_r = \frac{\rho_{d \max}}{\rho_d} \cdot \frac{(\rho_d - \rho_{d \min})}{(\rho_{d \max} - \rho_{d \min})} \quad (4.2)$$

where ρ_d is the dry density of the fill, as measured in situ (see Section 8.4.3), $\rho_{d \max}$ is the corresponding maximum and $\rho_{d \min}$ the corresponding minimum dry densities of the fill, both measured in the laboratory (BS 1377:1990: Part 4: Tests 4.2 to 4.5 (BSI, 1990)). Usually expressed as a percentage, D_r obtained by this means may be used to find the angle of shearing resistance ϕ by employing a correlation such as that by Parkin (1988). This correlation is reproduced in Figure 4.4, where ϕ (triaxial) values corresponding to different granular soil types are given. The ϕ values obtained this way are applicable to normally consolidated material. It is likely that most compacted granular fill will be over-consolidated, at least throughout most of the layer thickness, so conservative results will be produced when using this correlation.

Figure 4.4 Variation of angle of shearing resistance ϕ (triaxial) with relative density D_r for normally consolidated granular soils (from Parkin, 1988)



Care should be taken to see that relative density tests are carried out on representative samples. If corrections were made when performing the maximum dry density tests (see Section 2.6), the corresponding changes to the particle size should also be made to the materials used in the other tests that have to be performed to calculate D_r .

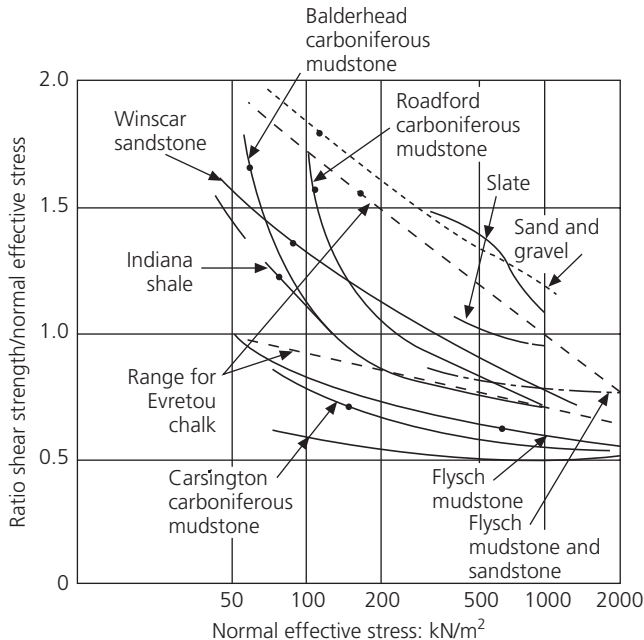
Charles and Watts (1980) proposed a non-linear relationship between shear strength τ and normal effective stress σ' , as follows

$$\tau = A(\sigma')^b \text{ (MN/m}^2\text{)} \quad (4.3)$$

where A and b are constants. Tests were performed on sandstone, slate and basalt fill compacted in a large diameter triaxial device. It is likely that a relationship of this form also applies to coarse and very coarse granular soil fill as well as rock fill. In a test series involving crushed basalt, Al-Hussaini (1983) found that measured ϕ values increased with sample relative density and also deduced evidence that ϕ increased as the coefficient of uniformity increased (Equation 4.1), that is, strength was larger for well graded as opposed to uniformly graded materials, when tested under the same conditions.

Shear strengths of a number of weak rocks are illustrated in Figure 4.5 (Vaughan, 1994); the ratio of shear strength to normal strength varies strongly with normal stress. This implies in turn a strongly non-linear failure envelope.

Figure 4.5 Shear strength of fills from strong and weak rocks (from Vaughan, 1994)



4.4. Behaviour of coarse (granular) and some weak rock fills: temporary works

4.4.1 General

Because of its free draining properties, coarse (granular) fill does not normally suffer from the construction problems already described for fine (cohesive) fill (see Section 3.4). However, and as noted in Section 4.3.1, uniformly graded fine sand, when used as fill, demonstrates a markedly lower density for a given compactive effort than other granular soils, and a common problem during construction with such soils is ‘over-stressing’. Because the normal stress at the top of the current layer is low, the shear strength of the granular soil is also low; therefore, under the action of some compaction plant, loosening of the layer surface may occur as the shear strength is exceeded by the applied shear stress. The problem can be reduced, if not avoided, by limiting the applied shear stresses, for example by switching off the vibratory component of a vibrating roller for the initial passes or by using lighter equipment.

4.4.2 Construction settlement

The maximum internal self-weight settlement ρ of a wide body of granular fill of height H occurs at mid-height and is (Charles, 1993)

$$\rho = 0.25 \frac{\gamma H^2}{D} \quad (\text{m}) \quad (4.4)$$

where D is the constrained modulus ($1/m_v$) and m_v is the coefficient of volume compressibility.

On the basis of field measurements, Charles and Penman (1988) proposed modifying Equation 4.4 to the following relationship

$$\rho = 0.30 \frac{\gamma H^2}{D^*} \quad (\text{m}) \quad (4.5)$$

where D^* is the constant equivalent constrained modulus. Typical values of D^* are given in Table 4.1 for a variety of fills, including a clay fill, and for three different embankment heights.

Table 4.1 Typical values of constant equivalent constrained modulus D^* for various fill types and embankment heights (Charles, 1993)

Fill type	D^* : MN/m ²		
	$H = 10 \text{ m}$	$H = 30 \text{ m}$	$H = 100 \text{ m}$
Sandy gravel ($D_r = 80\%$)	50	90	170
Sandy gravel ($D_r = 50\%$)	30	50	90
Sandstone rock fill ($D_r = 80\%$)	15	25	45
Sandstone rock fill ($D_r = 50\%$)	6	10	20
Clay (plasticity index, 15%; liquidity index, 0.1)	6	10	18

Table 4.2 Typical values of secant constrained modulus D_s (stress increment from 30–130 kN/m²)

Fill type	D_s : MN/m ²	ε_v : %
Sandy gravel ($D_r = 80\%$)	50	0.2
Sandy gravel ($D_r = 50\%$)	25	0.4
Sandstone fill ($D_r = 80\%$)	12	0.83
Sandstone fill ($D_r = 50\%$)	6	1.67
Colliery spoil (compacted)	6	1.67
Colliery spoil (uncompacted)	3	3.3

Most practical engineers are not so much interested in the maximum internal self-weight settlement as the total self-weight settlement, because this will yield the amount needed to top up at the end of construction. Simple integration will show that the total self-weight settlement is

$$\rho_t = 0.50 \frac{\gamma H^2}{D} \quad (\text{m}) \quad (4.6)$$

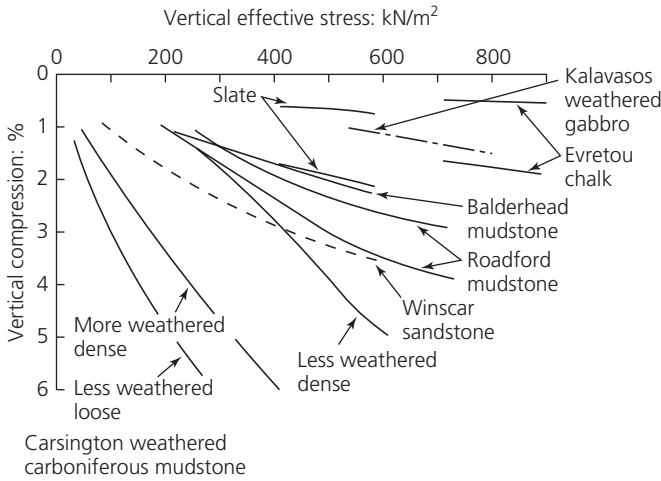
where the symbols have the significance previously defined. It is suggested that the values of D^* quoted in Table 4.1 be adopted when using Equation 4.6.

For granular fill used as a foundation, Charles (1993) proposed the use of a secant constrained modulus D_s . Values of this property, based on the results of large 1 m dia. oedometer tests over a stress range from 30 kN/m² to 130 kN/m², are reproduced in Table 4.2 (the vertical strain ε_v is also quoted).

Note that the secant constrained modulus depends heavily on the relative density (D_r). Increasing D_r from 50% to 80% doubles the modulus of the fill concerned. However, note also that heavy compaction plant and relatively thin layers may be necessary to produce a relative density of 80% in very coarse granular or rock fill, depending on the maximum particle sizes. The values quoted in Table 4.2 will not necessarily hold for different stress increments; Charles (1993) gives empirical formulae that may assist in such cases. Methods of estimating settlements using standard penetration test (SPT) data have been proposed by Burland and Burbridge (1985).

The compression of some weak rocks is illustrated in Figure 4.6 (Vaughan, 1994). As might be expected, there is a non-linear relationship between vertical compression and vertical effective stress. It is worth noting the range for mudstones, from relatively low for Balderhead to high for Carsington. Use of these data for the appropriate stress range, together with Equation 4.6, may help in estimating construction settlements. Such settlements can be of value to a contractor who may wish to know the quantity of additional material to haul to the embankment to achieve design elevation.

Figure 4.6 Variation of compression with vertical effective stress for weak rock fills (from Vaughan, 1994)



4.5. Behaviour of coarse (granular) and some weak rock fills: permanent works

4.5.1 General

During the design life of earthworks constructed of argillaceous rocks, there is a potential for creep and collapse settlement to occur due to the degradation of particles. These phenomena are particularly relevant to the bulk filling of abandoned previous excavations such as open-cast quarry sites or gravel pits. Erosion may also occur, particularly with poorly compacted granular fill or fine grained fill having a uniform particle size distribution. These phenomena are commented on below.

4.5.2 Creep settlement

Experience indicates a linear relationship between creep settlement and the logarithm of time elapsed since construction. This takes the form

$$\alpha = \frac{100\Delta_s}{H \log(t_2/t_1)} \quad (4.7)$$

where Δ_s is the settlement between times t_1 and t_2 since completion of construction, and H is the embankment height at the end of construction. The relationship is normally expressed as a percentage.

In a study of creep settlement for a variety of British Coal Measures open-cast fills, Hodgetts *et al.* (1993) and Hills and Denby (1996) found that a 'full scheme of backfill compaction' produced an α factor of about 0.2%, 'partial backfill compaction' of about 0.4% and 'uncompacted backfill' of about 0.8%. Values of α varying from 0.2% to 1.0% were reported for rock fill dams in the USA (Charles, 1993) and Sharp (1996) published

values of 0.34 and 0.48 for rock fill used in the construction of quays in the Gulf and in Ghana, respectively. Care must be taken in interpreting these data because the value of α may not simply be related to the nature of the fill, but also to its method of placement. For the US examples, the smallest values of post-construction settlement (and hence α) were obtained when the fill had been sluiced during compaction to remove fine particles (Charles, 1993).

4.5.3 Collapse settlement

Collapse settlement can occur with some weak rocks as well as with cohesive fills. If there is any likelihood of water rising up into the base of the fill, monitoring is essential and construction at the surface should be postponed for as long as possible to reduce the possibility of volume change during the in-service period. This phenomenon is unlikely to occur with engineered fill embankments for highway or railway construction as the equilibrium groundwater level is in the natural ground below the embankment toe. Collapse settlement has been attributed to earthworks fill due to infiltration of precipitation, but this mechanism should be treated with care as the infiltration is likely to be of only a transient nature.

There is, however, evidence (Charles, 1993) that surface water entering open trenches during construction may be sufficient to cause such settlement, albeit locally. More severe consequences may be expected from leaking mains and drains or the careless and ill-considered location of soakaways. Attention should always be paid to the performance of engineering structures such as these, both during construction and over the long-term permanent works condition. It should not be assumed that collapse settlement is entirely restricted to water rising up in the base of fill.

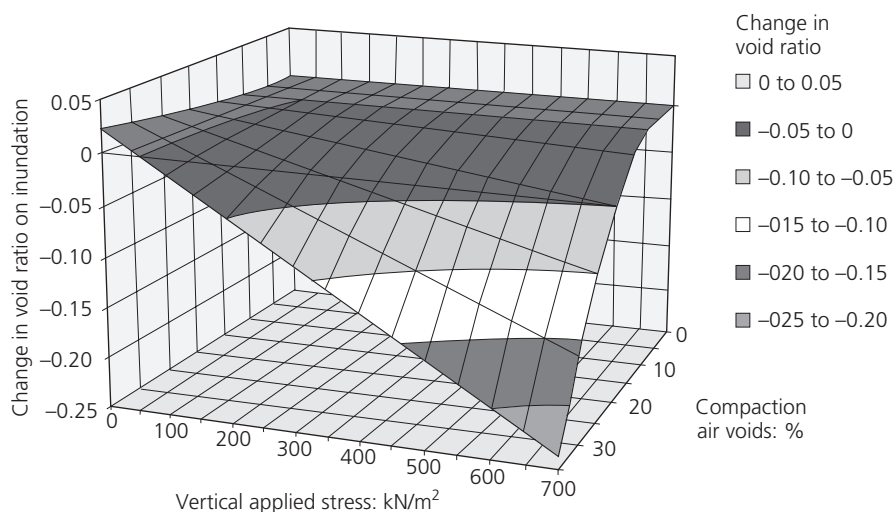
With respect to bulk fill of abandoned quarry sites, Hills and Denby (1996) measured the collapse settlement response of a number of Coal Measures open-cast fills, largely from the same study areas as discussed under creep settlement above, and proposed a relationship between collapse settlement and air voids. They proposed that collapse settlement increased non-linearly as air voids content increased and suggested that a site compacted to air voids contents between 0% and 10% would represent full backfill compaction, between 10% and 15% partial backfill compaction, and greater than 15% uncompacted backfill.

This work was extended by Blanchfield and Anderson (2000). On the basis of laboratory tests and field measurements made on a site in the English Midlands, they considered that collapse settlement was related not only to air voids content but also to the magnitude of the overburden pressure acting on the fill (as proposed in Section 3.5.1). Their relationship is illustrated in Figure 4.7. Most available records for collapse settlement in the UK concern open-cast materials but the results obtained are likely to be applicable to other weak rocks.

4.5.4 Erosion

Erosion may be a major problem with uniformly graded sand; in part, this is due to the low dry densities achievable with fill formed from these materials (see above) but also

Figure 4.7 Three-dimensional representation of void ratio change with compaction air voids and vertical stress at inundation (from Blanchfield and Anderson, 2000)



because the few particle contacts generated by its uniform particle size distribution mean that individual grains are poorly anchored one to the other and are easily disturbed by flowing water. Where the use of such fill is inevitable and erosion is a risk, such as with river training, canal works, adjoining drains or soakaways, some form of geotextile protection should be considered to avoid erosion of the face due to fast flowing water. In addition, for underwater slopes formed in dry conditions the stable submerged slope angle will be considerably shallower than the stable constructed slope angle, for example, 1V:2H dry compared with 1V:6H when submerged, and slumping may occur with resultant loss of stability of the overall slope. Gravity protection, in the form of a rock fill or concrete filled mattress, will be required on the slope face to prevent this occurring.

An additional erosion problem can occur on finished earthworks where uniformly graded sands have been compacted wet of their optimum water content. The pressure applied by subsequently compacted layers causes migration of the free water in the wet layers to the slope face, causing slumping of the material immediately adjacent to the face with the occurrence of extensive topsoil translation failures and an additional maintenance requirement. This was experienced on the original construction of the M25 motorway between junctions 12 and 13 in fine grained Bagshot Formation sands, and also on the M65 Blackburn Bypass in Glacial Sand layers overlain by subsequently compacted layers of cohesive Glacial Till.

Where adequate bearing capacity for road sub-bases is needed, or to support low rise foundations, some form of soil stabilisation may be required, provided that full mixing of the constituents can be assured. *Soil Mechanics for Road Engineers* (Road Research

Laboratory, 1952) suggests that cement stabilisation can be adopted economically for granular soils having the following characteristics:

- maximum size: 76 mm
- passing 5 mm: >50%
- passing 0.425 mm: >15%
- passing 0.075 mm: <50%
- liquid limit: <40%
- plasticity index: <18%
- organic matter: <2%.

Most uniform fine sands in the UK will not meet the coarser end of this particle size distribution requirement (greater than 50% passing the 5 mm sieve) and the proportion of cement required may not make this a viable economic option.

4.5.5 Frost-heave susceptibility

Croney and Jacobs (1967) found that uniformly graded silts and silty sands were among the materials most prone to frost heave. They should not be used within 450 mm of finished pavement level (HA, 1991).

Croney and Jacobs (1967) found that granular fill may be regarded as non-frost-heave susceptible provided that 10% or less passed the 0.075 mm sieve (i.e. the silt content was small). The authors also noted that the degree of compaction did not significantly affect frost-heave susceptibility for the granular soils tested. Lambe (1962) remarked that frost-susceptible soils (such as silty fine sands) were not necessarily made less frost susceptible by treatment with cement. This advice differed to that of Croney and Jacobs (1967), who believed that a small proportion of cement (or bituminous) additive was sufficient to prevent heave in such soils. However, they stated that the effect of repeated freezing and thawing on such stabilised materials needed investigation.

Limestone gravels produced much greater heave than sandstone or flint gravels in the tests conducted by Croney and Jacobs (1967). They noted that all Oolitic and Magnesian limestones having an average saturation water content in excess of 3% must be regarded as frost-heave susceptible. This is due to the high absorption of water into the structure of the gravel particles, causing mechanical break down on freeze/thaw cycles. 'Hard' limestones (minimum dry density reported as 2.05 Mg/m³) with less than 2% average saturation water content (SWC) and with 10% or less passing the 0.075 mm sieve may be regarded as non-frost-heave susceptible.

4.5.6 Durability

The durability of particles is particularly important for coarse granular fill materials to ensure that the placed fill remains stable in the long term. This can be a particular problem when argillaceous rock fill is used as a granular fill with limited matrix materials because the particles are subject to high contact pressures and can in some cases deteriorate to a residual soil. This aspect is assessed by durability testing to determine suitability for use, particularly in specialist (Class 6) fill materials and bound aggregate.

BS EN 13242 (BSI, 2002b) sets the tests that relate to aggregate durability for coarse fill materials, which include:

- water absorption – BS EN 1067-6 (BSI, 2013).
- Los Angeles abrasion (resistance to fragmentation) – BS EN 1097-2 (BSI, 2010).
- micro-Deval coefficient (resistance to wear) – BS EN 1097-1 (BSI, 2011).

The Los Angeles abrasion test is extensively used as a classification test for Class 1C and Class 6 granular materials in the HA's SHW (HA, 1998, 2013).

Other tests that are sometimes employed to help assess aggregate durability include:

- freeze-thaw resistance – BS EN 1367-1 (BSI, 2007).
- Magnesium sulfate soundness – BS EN 1367-2 (BSI, 2009b).
- slake durability (see below).

The slake durability is useful for assessing the long-term durability of weak mudstone and siltstone aggregates. The test method is detailed by the International Society of Rock Mechanics (Ulausa and Hudson, 2007) and is also specified by the ASTM (2008).

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Chapter 5

Some characteristics of soft rocks

5.1. General

Chapters 3 and 4 cover fine (cohesive) and coarse (granular) soils used as engineering fill. This also includes excavated rock material used as granular fill, and highly to completely weathered rock utilised as cohesive and granular engineering fill, depending on the parent rock type.

A number of 'soft' rocks also occur in the UK and on continental Europe that have been used in earthworks in the past. The strata are generally of younger than Permian age.

5.2. Mercia Mudstone

5.2.1 General

The Mercia Mudstone is a series of red-brown mudstones with subordinate siltstones and sandstones of Triassic age. In the UK it crops out on both sides of the Pennines, in the Midlands and extends to the Bristol Channel. There are also outcrops in the Cheshire Plain and in the Mersey area. Only highly weathered Mercia Mudstone has soil-like consistency and engineering properties. Chandler (1969) produced a descriptive classification, grading the material into five zones, depending on the degree of weathering. This scheme is reproduced in Table 5.1, where it has been supplemented with index properties for various Mercia Mudstone zones published by Chandler and Davis (1973) and updated by Chandler and Forster (2001).

Grade 1 may behave like weak rock, along with any unweathered siltstones and mudstones. It is likely that the slightly weathered and drier Grade 2 material will show some granular compaction characteristics, giving way with weathering to the cohesive properties of Grade 4. This latter material may be expected to display the compaction characteristics of similar cohesive fill having a similar consistency index (see Section 3.3.2).

5.2.2 Excavation

Excavations in Grade 1 Mercia Mudstone tend to be reasonably stable in the short to medium term, when the slope is above the groundwater table, but the designer must still assess the required measures to ensure stability of the face. However, the stability will deteriorate when groundwater inflows are allowed to soften the mudstone.

Grade 2 to 4 mudstone excavations will be more difficult when seepages can result in internal erosion and failure of unsupported faces.

Any excavation for structural foundations in Mercia Mudstone must be covered immediately with blinding to prevent softening by water due to its low plasticity index.

Table 5.1 A classification for Mercia Mudstone (from Chandler and Forster, 2001)

Grade number and description		Notes	Water content: %	Plasticity index: %	Aggregation ratio
Fully weathered	4b	Matrix only	18–35	17–33	32.5
	4a	Matrix with occasional claystone pellets less than 30 mm dia., but more usually coarse sand size	18–35	17–33	32.5
Partially weathered	3	Matrix with frequent lithorelicts up to 25 mm. Lithorelicts become less angular with weathering	12–20	10–18	10–2.5
	2	Angular blocks of un-weathered marl with virtually no matrix	5–15	10–15	10–2.5
Un-weathered	1	Mudstone (often jointed and fissured)	5–15	10–15	10–2.5

Earthworks in Mercia Mudstone can usually be effected using normal earthworks plant unless layers of mudstone, siltstone or sandstone are encountered and ripping or blasting may be locally required.

5.2.3 Compaction

When used as engineering fill, the behaviour of Mercia Mudstone reflects its low value of plasticity index. When compacted at natural water content at around the plastic limit, it is an acceptable engineering fill. With less weathered mudstone, however, a small increase in water content raises the liquidity index with a subsequent loss of strength and can render an area under compaction untrafficable. Stockpiling of unsealed or un-compacted mudstone can also become unacceptable as engineering fill if left exposed to the elements.

With respect to classification under the SHW (HA, 1998, 2013) Mercia Mudstone falls into Class 2A to 2D depending on weathering, percentage of lithorelicts and quantity of silt.

5.2.4 Potential for heave

Chandler and Davis (1973) performed compaction tests on Mercia Mudstone of widely varying plasticity. The amount of heave was found to depend on what was termed the placement moisture deficiency (liquid limit less the placement water content). The greatest heave occurred with the samples of highest plasticity and with the highest placement moisture deficiency. The authors recognised that the heave resulted from residual suctions prior to wetting. Their work was expanded by Cox (1978). The comments on heave (Sections 3.5.2 and 8.3.4) apply here.

5.2.5 Frost susceptibility

As Mercia Mudstone is predominantly silty it is generally susceptible to frost action. Care should be taken with road pavement construction to provide sufficient pavement thickness in order that Mercia Mudstone subgrade is protected from frost action.

5.3. Chalk

5.3.1 General

The Upper and Middle Chalk occurs in the south, south east and east of England and is a fine grained porous and remarkably pure limestone of Upper Cretaceous age. The Lower Chalk, while containing much calcium carbonate, also contains a high clay fraction giving it engineering properties more akin to the cohesive soils. For brevity the Upper and Middle Chalk will be referred to simply as 'chalk'. Further information about the nature of the chalk and its stratigraphy is provided by CIRIA C574 (Lord *et al.*, 2002) and Mortimore (2012).

Chalk porosity can vary, giving rise to a wide range of water contents, densities and strengths; 'hard grounds' are reported, in which the void ratio is as low as 0.1, making excavation difficult without special equipment. For the more normal lower strength materials, water is released on excavation, and subsequent handling and trafficking can make them difficult to work; for this reason, double handling should generally be avoided, especially with the softer, higher water content chalks.

In addition to its finely porous structure, chalk contains discontinuities that account not only for its high permeability in situ, but also facilitate its break down on excavation to fragments from boulder to sand and gravel sizes. Free water, which may be contained within the discontinuities, also contributes to the water released on break down. Chalk weathers along discontinuities, frequently widening them to solution features filled with collapsed material (see Section 10.6.6). Weathering breaks down its porous structure, resulting in the formation of ‘putty chalk’, which has a structure and consistency quite unlike the intact chalk from which it was derived. Therefore, far from being the uniform substance, which a glance at a quarry or borrow pit would suggest, chalk can be highly variable in its consistency and its engineering properties.

5.3.2 Chalk characterisation

The characterisation of chalk historically is presented by CIRIA C574 (Lord *et al.*, 2002). Early specifications defined the characterisation on the basis of:

- degree of pulverisation necessary
- maximum possible thickness of each compacted layer
- number of passes of the roller necessary to achieve target dry density.

Chalk earthworks classes were developed in the UK on the basis of saturation water content and chalk crushing value (CCV). Excavation and compaction methods were suggested by Ingoldby and Parsons (1979) and were later clarified in the Highways Agency Advice Note HA 44/91 (HA, 1991).

Current British practice, as described by the SHW (HA, 1998, 2013), employs the properties saturation water content (BS 1377:1990: Part 2: Test 3.3 (BSI, 1990)) and CCV (BS 1377:1990: Part 4: Test 6 (BSI, 1990)). The latter test measures the susceptibility of the chalk to crushing and is taken as the rate at which a 1 kg sample of single size chalk lumps (passing the 20 mm and retained on the 10 mm aperture sieves, respectively) contained in a 100 mm dia. mould crushes under the impact of a 7 kg rammer, falling from a height of 250 mm (Ingoldby, 1979). Using this test, a classification system for chalk earthworks involving four classes, ‘A’ to ‘D’, was developed (Figure 5.1). A flow chart for the use of chalk as fill is given in the Highways Agency Advice Note HA 44/91 (HA, 1991), together with recommendations for the compaction method to be used in Table 6/4 of the SHW (HA, 1998, 2013).

French earthworks practice for chalk is described by Rat and Schaeffner (1990). It has been developed on the basis of intact dry density (IDD) and natural water content (NMC).

Greenwood (1993) considered the CCV should not be used to classify chalk in the UK and developed a classification based on IDD and natural water content, similar to that developed in France. He recommended the relationship given in Figure 5.2. Greenwood also related his scheme to the compaction methodology quoted in the SHW (HA, 1998, 2013) and given in Table 5.2. CIRIA C574 contains the most recent guidance for chalk classification based on IDD.

Figure 5.1 Chalk classification chart for use with SHW (HA, 1998, 2013) (from Advice Note HA 44/91 (HA, 1991))

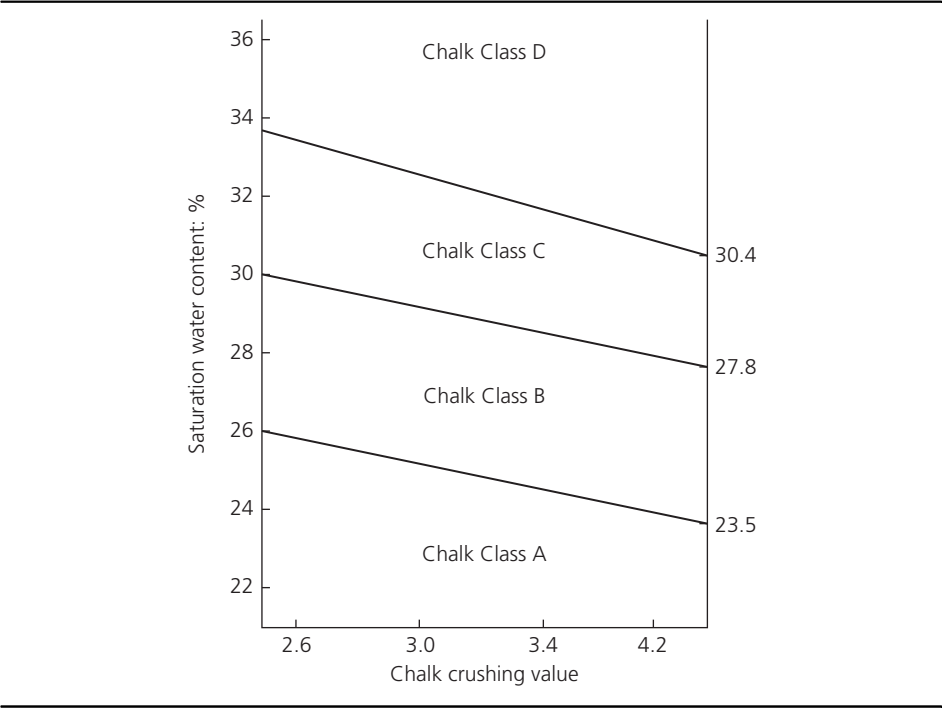


Figure 5.2 Proposed classification chart for chalk fill (from Greenwood, 1993)

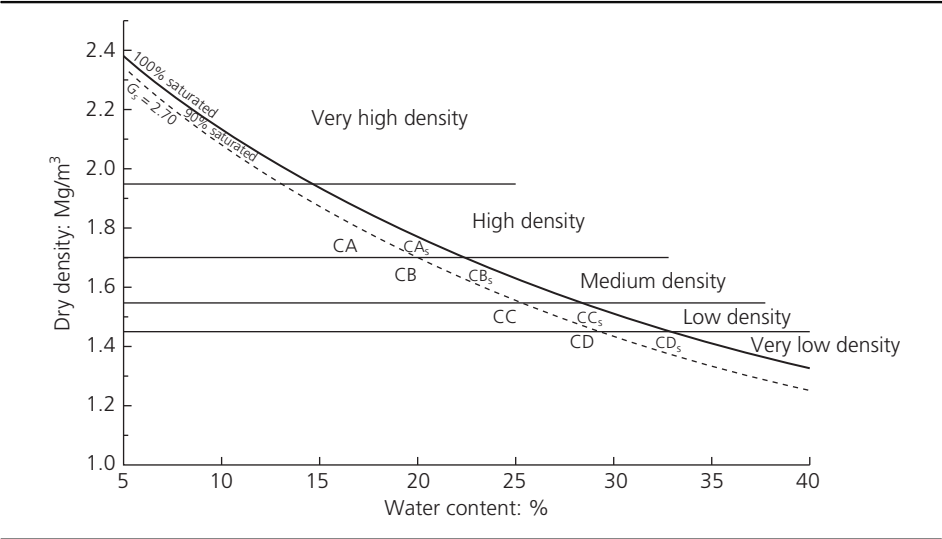


Table 5.2 Guidance on chalk compaction (Greenwood, 1993)

Chalk class ^b	Preferred compaction method	Likely excavation plant	Temporary Instability ^d	Workable in light rain
CA	2	Any	Not expected	Yes
CA _s	2 ^c	Any	Not expected	Probably
CB	1	Any	Not expected	Yes
CB _s	1 ^c	Face shovel	Unlikely	No
CC	1	Face shovel	Unlikely	Possibly
CC _s	4	Face shovel	Probable	No
CD	4	Face shovel	Possible	No
CD _s	4 ^a	Face shovel	Expected	No

Preferred compaction method relates to the SHW (HA, 1998, 2013), Table 6/4

^a Use may not be economical in embankment fill

^b Use of subscript 's', as in CA_s, indicates chalk material between 90% and 100% saturation

^c Amend to compaction method 2 if water content is less than 22%

^d Chalk particles tend to re-cement with time

The Greenwood classification was further developed by Toombs and Snowdon (1997).

Greenwood also offers a flow-diagram to facilitate the use of his approach. While Greenwood's method appears to offer advantages, particularly because of the few tests necessary to perform the classification, a check against the existing SHW (HA, 1998, 2013) procedures would be prudent, particularly for larger projects.

CIRIA C574 (Lord *et al.*, 2002) suggest that adoption of the Highways Agency method specification has the potential to generate a high percentage of unacceptable fill, and advocate that a more efficient classification of chalk for earthworks is using the Greenwood or Toombs and Snowdon classifications.

Whichever the system adopted, the aim of the earthworks engineer is to produce a workable material and in practice this must be done by selecting the right combination of layer thickness and equipment to suit the particular chalk water content and hardness (or density). In this respect, chalk is no different from other materials the earthworks engineer has to handle, but the consequence of getting the combination wrong is probably more severe with chalk than with most other materials. Site trials are therefore considered essential (see Section 8.5).

5.3.3 Excavation and compaction

Dependent on block size, fracturing and discontinuity spacing grade CA and CB chalk may require loosening with rippers before using scrapers and dozer-assisted scrapers for excavation. A face shovel of at least CAT 245 size or equivalent is likely to be required. For grade CC to CD_s chalk face shovels or backacters are usually acceptable for excavation.

Great care is needed when placing poor quality Class C and D chalks. The following, taken in part from the SHW (HA, 1998, 2013), are suggested precautions:

- no earthworks to take place in winter (and in frost or periods of heavy rain)
- excavating and/or mixing poor (Class C and D) with better quality (Class A and B) chalks to be avoided
- trafficking by heavy (greater than 15 m³ struck capacity) construction plant to be avoided
- trafficking in winter months only if covered by protective layer, with layer trimming delayed by at least 4 weeks
- at the end of each working day, the work to be sealed by two passes of a 2100 kg smooth-wheeled roller
- no stockpiling or multiple handling
- work to be halted if temporary instability occurs
- trials to determine how long it may take for poor quality chalks to harden after initial disturbance, and such times written into the contract documents
- use of a face shovel (or backacter) for excavation to produce the minimum amount of tracking and hence the minimum amount of break down, with the machine chassis stationary while working and swinging to excavate and unload.

The length of time necessary for wet chalks to dry out and to harden can be critical to a construction contract. Measurements of the water content loss with time were made by Heath (1966) on wet chalk fill delivered to the site of the M40 High Wycombe Bypass during dry sunny weather, when the temperature was reported as varying from 24°C to 28°C. After 1 h the water content of the top fill layer reduced by about 3 percentage points, after 2 h by about 4 and after 4 h by about 6.5. Wet weather can of course cause the opposite effect.

Although rightly regarded as a potential problem, high water content chalks need not be a deterrent to successful earthworks, provided that the appropriate site trials are undertaken and there is strict site control throughout. Designated as Class 3 on the SHW (HA, 1998, 2013) classification, Rail Link Engineering were able to handle chalk with water contents varying from 20% to 28% for one of the Channel Tunnel Rail Link contracts. It was also possible to use chalk as engineered fill with water contents up to 30% by placing and compacting at a controlled rate, which lowered its water content and reduced instability (rutting) to an acceptable level. Air voids contents of 8% or lower were reportedly achieved.

To minimise problems in compacting wetter and finer chalks, large layer thicknesses have been used with little compaction. Earthworks practice from TGV construction in northern France is detailed by Quibel (1990).

Substantial collapse settlement can, however, occur with improperly compacted chalk fill. Clayton (1980) observed settlements up to 6% of embankment height because of this mechanism. Careful mapping and classification of the chalk during site investigation should help to identify the higher water content and potentially unstable chalks and to

permit planning of excavation and compaction during the main works. This is especially important when construction in autumn is expected. As demonstrated in the Rail Link Engineering example quoted above, site trials are essential, especially where wetter chalks are in contention.

In order to avoid damage to a chalk sub-formation, Lord *et al.* (2002) recommend that a 600 mm thick protection layer should be provided at the base of excavations and the top of embankments and stripped immediately prior to starting pavement construction.

Under the SHW (HA, 1998, 2013), Class 3 materials should not be stockpiled or subjected to multiple handling. Low and medium density chalk has, however, been stockpiled and re-used in chalk earthworks satisfactorily if the following precautions are taken.

- When placed, each layer of chalk should receive sufficient light compaction to minimise air voids.
- At the end of each day, or on stoppages for rain, the surface should be sealed by rolling.
- The surface should be profiled to generous falls (>1 in 40) to assist surface runoff.

5.3.4 Conditioning of chalk fill

The use of lime (or cement) stabilisation procedures may be considered for wet chalks (say water content in excess of 28%). Lime (to improve handling) and cement (to increase strength) of chalk is frequently adopted in France, and Mudet (1993) and Puiatti (1995) may be consulted for French experience. Further details on British techniques in handling chalk may be obtained from the Highways Agency Advice Note HA 44/91 (HA, 1991), SHW (HA, 1998, 2013) and from Greenwood (1993).

Lord *et al.* (2002) suggest that the Highways Agency Advice Note HA 74/00 (HA, 2000) gives guidance on conditioning by lime/cement and cite classification of modified chalk fill on the Channel Tunnel Rail Link project. Grade C chalk from stockpile was successfully conditioned using lime and placed as embankment fill on the M25 Junction 16 to 23 Widening project.

5.3.5 Bulking

It should be noted that chalk is one of the few earthworks materials to reduce in volume from cut to fill. This is because of the blocky, fractured nature of Grades B and C chalk that is broken down during excavation and compaction. Jenner and Burfitt (1975) report a volume reduction of 10% from cut to fill. On the M11 Cambridge Western Bypass contract, an increase in density from cut to fill of some 4–5% was recorded.

5.3.6 Frost-heave susceptibility

All crushed chalks were found to be frost-heave susceptible by Croney and Jacobs (1967) and the magnitude of frost-heave increased linearly with the saturation water content of the chalk fragments. The authors reported that neither the water content nor the size of the chalk fragments, nor the degree of compaction, had much influence on frost-heave susceptibility.

Rat and Schaeffner (1990) report frost action by cryo-suction, whereby water is pumped from the underlying soil and concentrates near the surface in the form of ice particles. The formation of these particles destroys the structure of the chalk and leads to quasi-liquefaction during subsequent thaw.

5.4. Other soft rocks

The Edlington Formation (Permian Marl) occurs in northern and north eastern England and is very similar in nature to Mercia Mudstone.

The Magnesian Limestone (Cadeby and Brotherton Formations), of Permian age, occurs in northern and north eastern England. During diagenesis the calcium carbonate cement was largely replaced by magnesium carbonate. The result is that the Magnesian Limestone has a lower strength than that of calcium carbonate cemented limestones. When excavated, Magnesian Limestone can produce a granular material of low fines content (less than 10% silt and clay). Other layers when excavated, however, will produce up to 50% fines. This makes it difficult to classify excavated material under the SHW. It is usually classified as Class 1 or Class 2 material depending on fines content. It is not generally acceptable as Class 6 selected granular fill. The potentially high fines content and softer nature of sand and gravel sized material makes the Magnesian Limestone susceptible to increases in water content once compacted. This can lead to rutting, and break down of particle size can occur in compacted layers when trafficked.

Carstone is of Cretaceous age and occurs in eastern England as a weakly cemented sandstone. On excavation it can be classified as Class 1A, well-graded granular fill, but on compaction the gravel sized particles break down to produce a Class 1B, uniformly graded granular fill, comprising fine to medium sand-sized particles.

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Chapter 6

Some properties of industrial fill

6.1. Introduction

The use of waste industrial and construction products for engineering fill has been practised for at least the last 40 years. Legislation over the last 20 years has, however, made their use more attractive in the UK for both financial and sustainability reasons.

The UK government introduced an aggregate tax on the exploitation of natural aggregates in April 2002, and the rate currently stands at £2.00 per tonne as of January 2015. Recycled waste products are exempt from the tax. In 1996, the UK government introduced a landfill tax to discourage the disposal of waste to landfill. As of January 2015 charges stand at £80.00 per tonne for active waste and £2.50 per tonne for inactive waste (generally concrete, brick, glass and soil). Similar taxes also apply in New Zealand and California, USA.

In some cases, the industrial materials concerned may be only a short haul from the works, so additional environmental benefits accrue. Unfortunately, industrial materials may themselves be unsuitable by virtue of their chemical or physical properties, and so what appears to be an excellent environmental solution may bring other engineering problems. Nevertheless, provided that the necessary comprehensive test programme is performed, industrial materials may offer an attractive solution when used as engineered fill. The test programme should be specified recognising that the tests utilised may need to vary from standard soils testing to allow determination of the chemical properties of the material.

The directives above have increased the use of waste materials especially when the contract earthworks balance is in deficit. When planning an earthworks project it is important to recognise that aggregate tax may apply to fill imported from borrow pits and surplus fill material taken off site can often be subject to landfill tax. These factors can make the use of recycled materials more attractive. In all of these cases, the advice of a professional experienced in waste management regulations should be obtained.

In addition, the Waste and Resources Action Programme (WRAP) was established in the UK in 2000 to promote a reduction in the use of natural primary resources and the re-use of waste products. WRAP is an independent body but includes representatives of the UK Department for Environment, Food and Rural Affairs and Scottish and Welsh governments. Some of the aims of their 2011–2015 business plan are:

- 7 million tonnes less carbon dioxide equivalent each year
- 3 million tonnes less primary resources each year
- 2 million tonnes less waste produced each year.

WRAP launched a quality protocol (QP) in 2004 covering the production of aggregates from inert waste. The QP provides a quality assurance and control pathway from waste to product. The main criteria to consider when waste has been through an authorised recovery process are:

- certainty of use
- suitability of use
- no greater risk to the environment and human health than equivalent primary aggregates.

To achieve this aim, aggregates are required to be produced under a factory production control system under European standards to achieve:

- a defined production control
- classification of a product through standard tests
- a defined quality management system
- a record management system to demonstrate reliability and consistency.

The use of waste materials in construction is considered in British Standard BS 6543 (BSI, 1985), in which some guidance is given on their chemical content in relation to their potential to attack building fabric. A flow chart for determining the suitability of waste materials for engineered fill purposes is given in Figure 6.1. Additional information on the type and appropriate testing of recycled and secondary aggregates is provided by Steele (2004). It is important to note that this covers the engineering aspects of the re-use of waste materials, not the environmental and legislation soil/groundwater contamination aspects.

A review of industrial materials used in highway construction is given by Sherwood (1995).

6.2. Colliery discard (minestone)

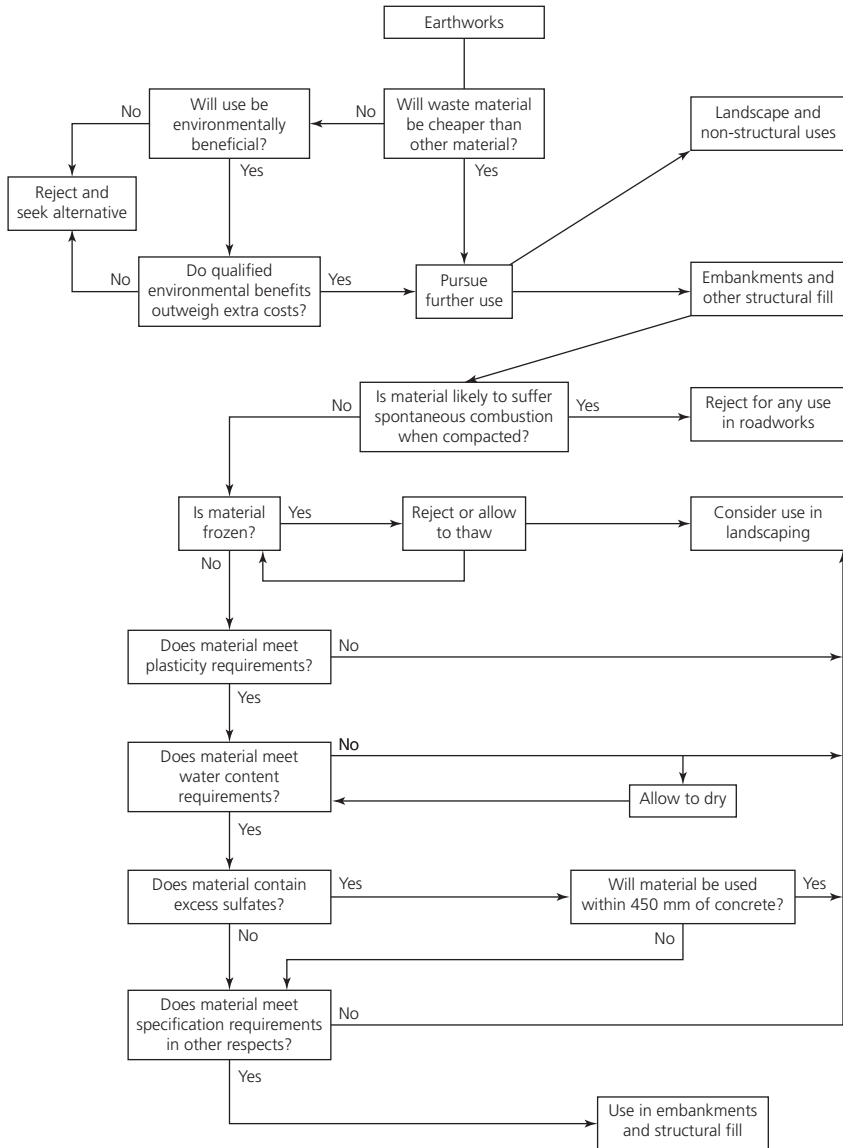
6.2.1 General

There are large quantities of colliery discard or ‘minestone’ forming spoil heaps or tips, both in former and in existing coal-mining areas of the UK. It normally comprises shales, mudstones, siltstones with some sandstones and arises from both coal face working, roadway and heading construction, drift and shaft sinking.

Colliery discard (unburnt colliery shale in the UK or Black Blaes in Scotland) is the coarse by-product of the colliery workings and ranges predominantly from sand to cobble sizes. Fine discard is a washery by-product, either a slurry (fine coal particles) or tailings (fine rock particles), and was deposited in lagoons either on the colliery discard spoil heap or on adjacent land. When taking delivery of colliery discard for fill purposes, care should be taken that quantities of fine discard from lagoons on the spoil heap have not become mixed with coarser material.

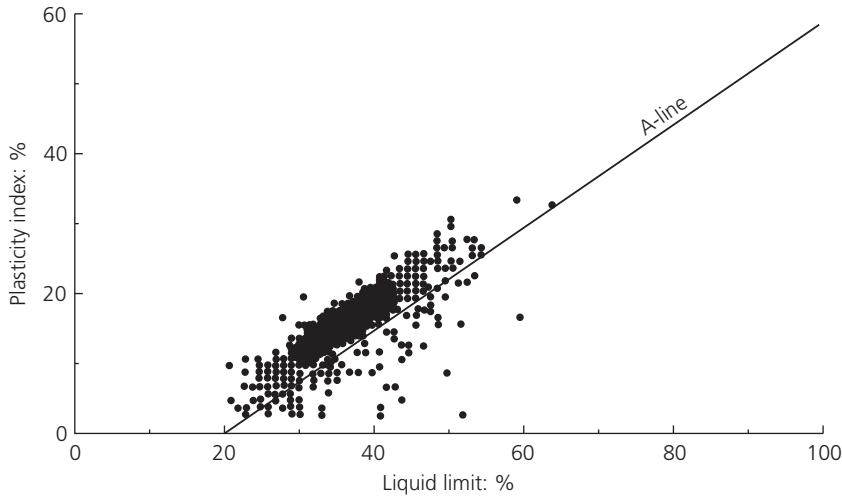
Many old spoil heaps were created by loose tipping, leaving the materials in a loose condition and at their natural angle of repose. The coal content in some of the discard

Figure 6.1 Flow chart for determining suitability of waste materials for use as engineered fill (from BS 6543 (BSI, 1985))



was high. Firing was common, sometimes caused by the negligence of operatives and sometimes due to the heat caused by the oxidation of pyrites present in the discard. Large quantities of 'burnt colliery shale' or 'Red Blaes' were produced as a result. This material has somewhat different properties from the unburnt colliery discard. In addition, in Scotland, Red Blaes comprises colliery shale that was burnt as an industrial process to extract oil.

Figure 6.2 Atterberg limit properties for coarse unburnt colliery discard (from Taylor, 1984)

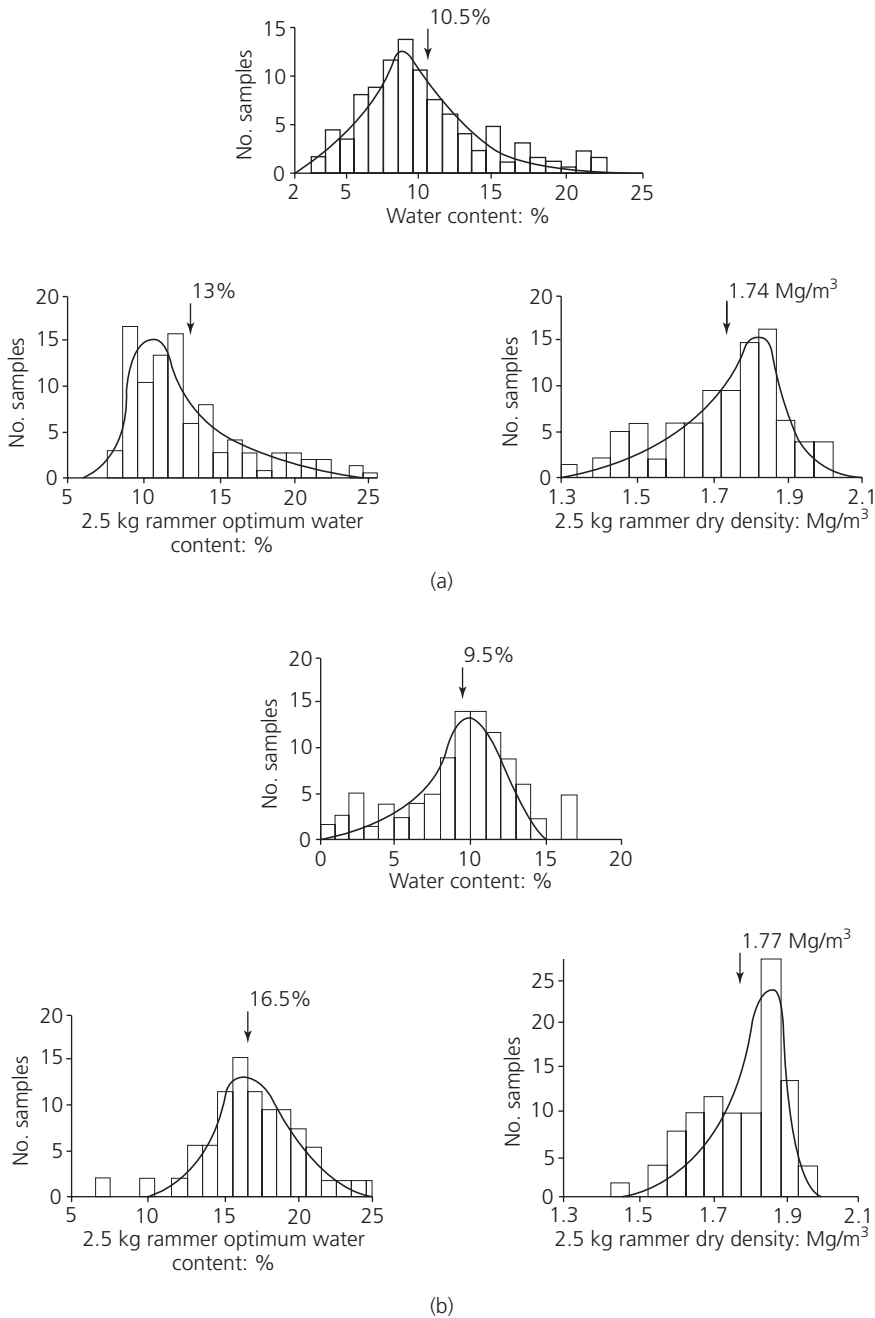


Coal content is one of the main factors affecting the performance of unburnt colliery discard. Organic carbon content can vary from virtually zero to 40%, with the specific gravity varying from 2.65 to 1.7 as a consequence (Taylor, 1984). The plasticity indices of several hundred coarse unburnt colliery discards from all over the UK were measured and the results are given in Figure 6.2 (Taylor, 1984). The majority of the plasticity indices fall below about 20%, indicating low plasticity clay behaviour, but a notable proportion fall above 20%. Note that the plasticity indices are measured on a fine grained particle range but in many instances the coarse discards break down to fine particle sizes during placement and compaction, so these data may be significant for design purposes. The break down of coarser particles was noted during construction of the Sellindge Electricity Converter Station in Kent in the early 1980s where unburnt colliery shale had been constructed as an enabling works construction platform. The material deteriorated considerably on trafficking by tracked crane and piling plant, resulting in a significant loss of strength to some 1.0 m below formation level and a break down of particle size by one order of grain size, that is, coarse gravel to fine gravel and fine gravel to sand size.

The natural water content with respect to optimum is of practical use for design and construction using colliery discard (both unburnt and burnt). Average data from the results of a large number of tests (Taylor, 1984) are plotted in Figure 6.3.

The average water contents (i.e. as received) of both the unburnt and the burnt colliery discards are dry of the average optimum, as illustrated in Table 6.1, particularly so for the burnt material. Significant addition of water may be necessary when using these materials as fill, especially in the dry season. As some of the discard particles are relatively small in size, the addition of water may prove effective in improving compaction performance.

Figure 6.3 Compaction characteristics of (a) burnt colliery discard and (b) unburnt colliery discard (from Taylor, 1984)



Note: arrows refer to mean value for each plot.

Table 6.1 Compaction test data for colliery discard, 2.5 kg rammer method (Taylor, 1984)

Material	Average water content: %	Average optimum water content: %	Average maximum dry density: Mg/m ³
Unburnt colliery discard	10.5	13.0	1.74
Burnt colliery discard	9.5	16.5	1.77

Unburnt colliery shale will usually classify as Class 1A, well-graded granular, or Class 2C, stony cohesive fill, to SHW (HA, 1998, 2013) depending on the clay and silt content, which is usually in the range 10–20%. Burnt colliery shale/Red Blaes will usually classify as Class 1A as the silt and clay fraction has been ‘fired’ and is present as a sand fraction so the percentage of the material passing the 63 µm sieve is usually less than 15%. SHW precludes the use of argillaceous material, such as colliery discard, as Class 6 material.

On the A1(M) Dishforth to Darrington contract the embankments for the A1(M)/M62 junction were constructed using unburnt colliery shale from the nearby Knottingley colliery. The material was classified as Class 2C although some 30% was classified as Class 1A. Large scale shear box testing of the material indicated a peak angle of friction (ϕ') value of 35°. Similar angle of friction results were achieved from shear box testing on imported Black and Red Blaes embankment material from the Midland Valley of Scotland coalfields on the M74 Completion contract in Glasgow.

Both burnt and unburnt colliery discard typically contain sulfates in higher concentrations than most other soils and this will need to be considered as part of the design of any buried concrete or steel structures, see Section 6.2.3. Taylor (1984) collected the average sulfate contents for a large number of unburnt and burnt samples, summarised in Table 6.2 (ranges in brackets).

The substantially higher average sulfate contents of the burnt colliery discard should be noted and is probably due to the oxidation of pyrites to sulfates during firing. Further data on the sulfur contents of unburnt colliery discards are shown in Figure 6.4; it will be seen that the maximum water-soluble content (0.2%) is substantially higher than that of the data provided by Taylor (1984). Scatter must be expected and each discard should be thoroughly tested.

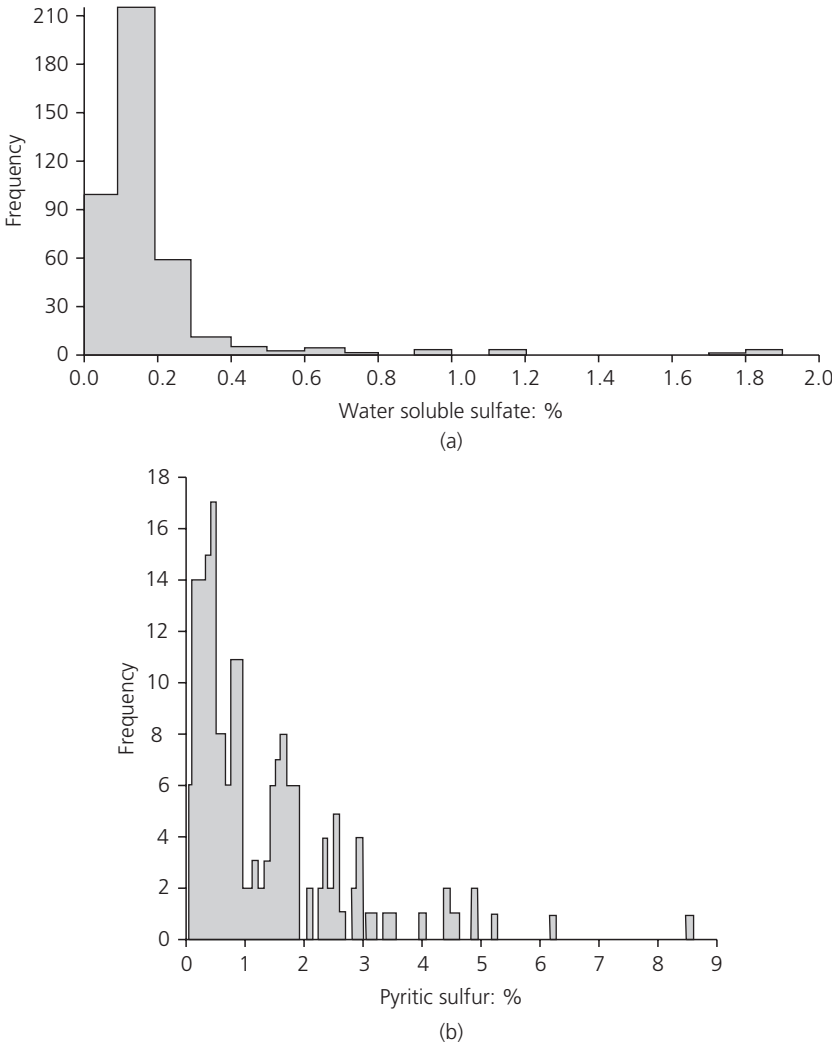
6.2.2 Use of colliery discard as fill

Hird *et al.* (1998) describe the use of colliery discard as a landfill liner. They conclude that discard varies in suitability depending on several factors and that each must be

Table 6.2 Results of sulfate tests on colliery discard (Taylor, 1984)

Material	Sulfate (% by weight) acid soluble	Sulfate (% by weight) water soluble
Unburnt colliery discard	0.20 (0.04–3.20)	0.10 (0.01–0.96)
Burnt colliery discard	0.70 (0.02–4.00)	0.18 (0.01–1.02)

Figure 6.4 Sulfur-bearing mineral contents in unburnt colliery discard (from Sherwood, 1995): (a) water soluble sulfate: %; (b) pyritic sulfur: %



investigated in detail to ascertain a discard's suitability. They believe that for a permeability requirement of 10^{-9} m/s, it is unlikely that consistently satisfactory results would be obtained with less than 20% fines, although given good mixing and compaction, a maximum particle size up to 50 mm would not have a significant effect on permeability. Compaction at, or slightly wet of, optimum water content, should be attempted and the influence of confining stress on permeability of discards is significant, as it is with clays. This should be recognised when specifying tests and interpreting test results.

There are frequently large quantities of colliery discard available in the former British coalfields, and for this reason its use as bulk infills for derelict colliery shafts and similar has been investigated. Cole and Figg (1987) describe the use of a mixture of colliery discard, pulverised fuel ash (PFA), lime and water as an infill for limestone caverns in the English midlands. The strength of the resulting mixture increased with time and was reportedly due to the pozzolanic reaction of its constituents. The use of this kind of mixture for other bulk filling would seem appropriate, particularly in those areas close to large discard supplies. The performance of the mixture will depend in part on the grading of the discard as well as the properties of the PFA, and pre-construction trials are essential.

Oxidation of pyrites is expansive and may be partly responsible for the break down of discards at shallow depth during weathering. The use of unburnt colliery discard as fill should be avoided beneath sensitive structures, particularly if a rise in temperature is likely (e.g. beneath a base slab for a warehouse, boiler, industrial, commercial or residential property) because of the possibility of damage due to a potentially expansive reaction.

6.2.3 Frost-heave susceptibility and chemical attack of buried construction materials

Croney and Jacobs (1967) state that burnt colliery discard is very susceptible to frost-heave and therefore should not be used in the top 450 mm of road construction. Care should be taken when either unburnt or burnt colliery shale is used close to structure foundations and the requirements of SHW should be followed. The material is likely to classify as DS-4 or DS-5 to BRE Special Digest 1 (BRE, 2005). In these cases a minimum thickness of 450 mm of an inert barrier material should be placed immediately adjacent to the structural concrete. This should ideally comprise Class 6N crushed limestone or crushed sandstone aggregate.

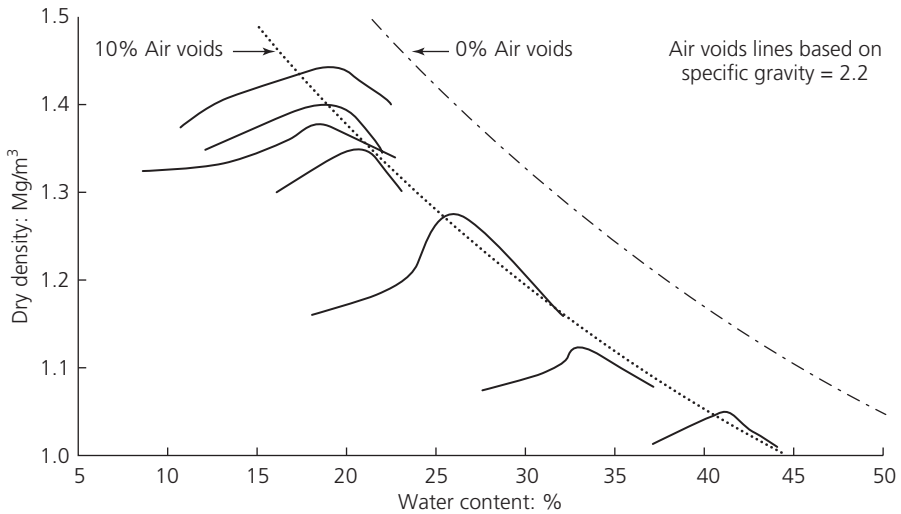
6.3. Pulverised fuel ash (PFA)

6.3.1 General

Coal-burning power stations produce PFA, which comprises well rounded and largely fine to coarse grained silt-size particles with various proportions of fine sand-size particles; sizes vary significantly from power station to power station, depending on the nature of the coal burnt. In the UK, PFA consists typically of silica (50%), aluminium oxide (30%) and ferric oxide (10%), together with smaller percentages of calcium, magnesium and potassium oxides and other substances in trace quantities. PFA is usually conditioned with water to improve its handling properties before delivery to site (termed 'selected fill') and some is pumped to storage in lagoons ('lagooned fill'). It is classified as Class 2E to the SHW (HA, 1998, 2013).

Another product of coal-burning power stations is furnace bottom ash. It is the coarser residue that falls to the bottom of the furnace and may be stored with PFA in lagoons. For this reason, lagoon PFA from some power stations may be substantially more varied in its composition than from others. Furnace bottom ash may be considered as a fill in its own right although some of the coarser particles may break down on compaction.

Figure 6.5 Variation of dry density with water content for pulverised fuel ash from a range of sources



Experience indicates substantial variation between the properties of PFA from different sources, and a significant variation in the engineering properties of a given PFA when sampled at different times. Results of compaction tests made on PFA from various sources are illustrated in Figure 6.5 (data from Eggborough power station, reported by Cabrera *et al.*, 1984 and the United Kingdom Quality Ash Association, 2007). There is considerable variation between the maximum dry density and optimum water content results, and the curves themselves are often relatively flat. Air voids contents are also typically higher than those of most 'conventional' fill materials, but are similar to other uniformly graded fills. Variations in properties may also arise depending on whether identical material was employed for each point of a compaction test or whether fresh material was adopted throughout. Care is necessary in sampling and testing PFA if reliable and consistent results are to be obtained.

6.3.2 Use of PFA as fill

Two attractive features of PFA as engineered fill are its low weight (specific gravity typically from 1.9 to 2.4) and its self-cementing (pozzolanic) properties.

In addition, the material tends to be inexpensive at source as most power stations are running out of lagoon storage space that was generally constructed in the 1960s and 1970s with no further room for expansion. Use of the material of the plant site extends the life of the storage facility. Whereas the original material was pumped as a slurry to the settling lagoons, the modern power generation process results in dry material that is transported in lorries or dump trucks to be deposited in a standard place and spread operation. This has implications for the use of PFA as an engineering fill as the addition of water is required to achieve optimum water content rather than reduction in water content if lagoon PFA is employed.

The low bulk density of PFA, typically 16–17 kN/m³, has obvious advantages for filling over soft soils and it has been used as backfill behind bridge abutments for this reason. Clarke and Coombes (1996) investigated the strength properties of PFA. Strength was found to be due partly to suction and partly to pozzalanic effects and, when the specimens were saturated to remove suction, there was a substantial loss of strength. There was also a significant hardening with age, together with an increasingly brittle response for the PFA tested, with some ashes reaching peak stress at less than 0.5% strain. The authors considered that SHW (HA, 1998, 2013) could be unsafe and that the recommendations given in BS 8002 (BSI, 1994) for earth retaining structures could be too conservative, because strength was likely to increase with time. For full details see Clarke and Coombes (1996). Testing for use on the M74 Completion and M25 Widening projects indicated peak ϕ' values in the range 31–34° from different sources.

Substantial hardening of conditioned PFA may be expected when mixed with 10% cement and 28-day strengths varying from 7.1 to 13.8 mN/m² have been reported (Sherwood, 1995). This property permits the use of cement stabilised conditioned PFA as a capping layer (HA, 1998, 2013). Its use has also found favour as backfill for mine workings. Grout mixtures comprising 1 part sulfate-resisting cement to 12 parts PFA have been employed for bulk filling of underground limestone workings in the West Midlands, with clean water added to produce a pumpable consistency (flowability between 300 and 600 mm in a concrete flowmeter). Sand may be used to replace the PFA (to a maximum of 50%) if a limit to the spread of the grout from the target area is desired.

The particle size and shape of PFA mean that the upper layers can be difficult to spread and compact: it may become highly unstable and erosion prone in wet weather and dust nuisance may be common when dry. The use of a starter layer is required, partly to provide a good working platform and partly to provide a capillary cut-off against upwards migration of groundwater. Likewise, a thickness of general fill (often about 600 mm) is recommended between PFA and the underside of the capping layer or sub-base. The Highways Agency document BD 30/87 (HA, 1987) reports that PFA backfill can be washed out by accidental flooding, forming voids. This is also commented on by Head (1994). In this respect, PFA is similar to some natural uniformly graded fine granular soils (see Section 4.3.1). Comprehensive detailing of the drainage system is essential when PFA fill is to be employed.

Environmentally, there are good reasons for treating PFA with caution, particularly if it is likely to come into contact with water sources. After initial placement, quantities of leachate with elevated levels of boron, sulfates and other salts may be released (Halcrow, 1994). This may be a function of conditioning, storing and stock-piling. The chemical content of the PFA should be carefully investigated before a decision to use it is made. If it is decided to use PFA on a project and sampling and testing have shown it to be satisfactory, then that particular PFA should be employed and no other, unless a correspondingly careful test programme has demonstrated its suitability.

Useful guidance with regard to the use of PFA as fill material is provided on the UK Quality Ash Association website (<http://www.ukqaa.org.uk>).

6.3.3 Frost-heave susceptibility and chemical attack of buried construction materials

Croney and Jacobs (1967) state that coarse PFA (less than 40% passing the 75 μm sieve) is unlikely to be frost-heave susceptible. Fine PFA may be susceptible ‘probably dependent upon the coals from which they were derived’ and should not be used within the top 450 mm of road construction unless frost-heave susceptibility tests indicate otherwise. The authors remark that the frost-heave susceptibility of fuel ashes can be reduced by cement stabilisation.

Note also the requirements of the SHW (quoted in Section 3.5.2) regarding deposition of sulfur-bearing fills adjoining buried construction materials.

6.4. Construction and demolition waste

Waste from the demolition of old buildings has been used for many years to provide a stable running surface on construction contracts as temporary haul roads and platforms for the working of tracked craneage and piling plant. The material was, however, highly variable and contained contaminant material in the form of timber, bricks and steel reinforcing bars so was not suitable for use as engineering fill.

Over the past 15 years, as a result of the legislation described in Section 6.1, selected crushed concrete material has been generated as a granular fill material. This is both by the use of on-site crusher and screens or as an off-site commercial operation selling the processed material to the wider construction market, as long as they are produced under a quality control procedure in accordance with the WRAP QP. The material is usually used for unbound applications such as capping (Class 6F5), unbound sub-base (Type 1) and drainage material, and in 2010 accounted for 28% of all aggregate sold in the UK.

On the M25 Widening design, build, finance and operate (DBFO) contract, crushed concrete was used as a Class 1A embankment extension material and large shear box testing of compacted samples indicated a peak ϕ' value in excess of 45° .

6.5. Other natural waste materials

The Department of the Environment, Transport and the Regions survey of derelict land in England (DETR, 1991) highlighted the existence and potential utility of many natural wastes and industrial by-products, which could be used as engineered fill. Natural wastes include:

- slate quarry wastes
- china clay pit wastes
- fluorspar mine waste
- tin mine waste.

These are not generally employed as engineering fill as they are specific to particular regions of the UK and, hence, high transport costs do not make them economically viable unless utilised close to the original source.

There are numerous slate waste tips in North Wales, some of which are in national parks, and comprise principally '500 mm down' material. The slate would be durable and is reportedly relatively chemically inert but, obviously enough, it has a very high flakiness index (Smith and Collis, 1993). It is also highly likely to break down to smaller (flaky) sizes on transport, placement and subsequent compaction. Nevertheless, it was employed as a dam fill at Dinorwig power station and its use as a granular sub-base has been reported (Sherwood, 1995); therefore, it would appear suitable for bulk fill purposes. Its geographical location is such that transportation costs to most parts of the country would be expensive, which would have to be borne in mind when considering its use.

China clay waste is the result of the extraction and production of china clay and comprises overburden, micaceous residue, sand-size granitic rock fragments and coarser waste rock materials (stent).

The sand-size granitic rock fragments have been employed as fill, although their mica content may give rise to compaction problems.

When screened and crushed, stent has been used to produce aggregates for various uses. Again, the location of china clay wastes in the extreme southwest of the UK means that transportation costs to most other parts of the country would be prohibitive.

6.6. Other industrial waste products

6.6.1 Steel and blast furnace slags

Steel slag has a high specific gravity and typically costs more to transport than most other fills. It can be chemically active having residual iron, free lime (CaO) or free magnesia (MgO), and therefore has the potential to produce undesirable leachates and expansion. BS 6543 (BSI, 1985) warns that it should be allowed to weather before being used as general fill in construction to allow an opportunity for the free lime and magnesia to reduce by leaching. Some countries specify a maximum free lime content of 4.5–5.0% (Smith and Collis, 1993). Steel slag should not be employed under buildings where groundwater is a possibility because of potentially expansive reactions following hydration of the free lime and magnesia it contains. Cracking in the lightly reinforced concrete floor slabs in council housing in the West Midlands of England was put down to heave of fresh steel slag (*New Civil Engineer*, 1984). Expansion of the free lime and magnesia was believed to be responsible.

Blast furnace slag has a lower specific gravity than steel slag but is still likely to be more expensive to transport than most industrial fills. Older stock-piled blast furnace slags may:

- contain undesirable chemicals
- contain steel slags
- vary in quality from batch to batch.

For these reasons, each batch should always be tested for undesirable properties.

6.6.2 Glass sand

Glass sand generally comprises crushed and screened bottles as a uniformly to well-graded material varying in grain size from fine gravel to fine sand, with a uniformity coefficient usually in the range 5 to 8. Due to the nature of the production process the grains are very angular. As this is a waste product the WRAP QP needs to be applied and the relevant production records verified prior to using the material.

Glass sand has been used for a number of applications on highway schemes, namely:

- paving material for pedestrian walkways on the A13 DBFO project
- drainage backfill material on the M1 Widening, junctions 6A to 10 project, and
- embankment widening (Class 1B), structural backfill (Class 6P) and drainage backfill material on the M25 Widening DBFO, junctions 16 to 23 project.

On the M25 Widening DBFO, large shear box testing on compacted samples of the material indicated a peak ϕ' value in excess of 45° .

6.6.3 Spent railway ballast

This material is generally of 40 mm single size but may contain smaller particle sizes, depending on age, due to crushing at grain boundaries during its primary use. It is frequently contaminated with oils, tars and coal, and may require cleaning prior to use. Its availability is, however, intermittent depending on track replacement activities in the railway industry, and it is seldom freely available for sale in significant quantities.

6.6.4 Road planings

This material would initially seem to be ideal as a fill material because in some cases it may be produced from highway renewal construction activities, with consequent reduction in transport costs. There may, however, be concerns over its possible toxicity, and appropriate chemical analyses are essential before any decision on its use as a fill are taken.

It is not usually available in significant quantities and may exhibit ductile behaviour under applied load. It is, however, acceptable as Classes 6F1 and 6F2 in quantities of less than 50% by mass and as Class 6F3 in quantities of over 50% by mass, under the SHW (HA, 1998, 2013).

6.6.5 Tyre bales

Recycled tyre bales have been utilised as embankment shoulder fill, combined with an acceptable bulk fill in the heart of an embankment. Their use on the A421 contract at Bedford is reported by Goldfingle (2009).

The use of tyre bales is covered by a publicly available specification produced by the British Standards Institute (BSI, 2013).

6.6.6 Wood chips

In areas of the world where timber is readily available and a timber industry has been established, wood chips have been used historically as a lightweight fill for embankment

construction over soft ground. The use of wood chip fill in the Fraser Delta area south of Vancouver, Canada is described by Brawner and Lea (1963) and Nichol (1998).

6.7. Use of material as working platforms

As stated in Section 6.4 above, waste materials have been used for the construction of haul roads and working platforms. The generation of waste materials that comply with the requirements of SHW means that these materials are now commonly left in place as part of the permanent works. Examples of this are:

- as drainage material below embankment construction following the installation of band drains
- as part of a load transfer platform following the installation of vibro stone columns or driven precast piles.

In addition, with the introduction of the construction and design management regulations (see Chapter 16), all working platforms for tracked plant now require a formal design. The design guidelines for working platforms are presented in BRE document BR470 (BRE, 2004). An approach to economic design is presented by Corke and Gannon (2010) and guidance on testing reliability is presented by Fountain and Suckling (2012).

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Chapter 7

Earthworks specifications

7.1. Introduction

BS 6031 (BSI, 2009) states that a specification for earthworks should adequately describe the design requirements, be easily understood by all parties to the contract, be practicable and capable of enforcement and measurement, and not be unnecessarily costly or time consuming in its application. The specification should be developed as a collaboration between the client, designer and contractor in order to optimise the use of excavated materials for use as engineering fill and define the properties of imported material. The specification should be formulated to address:

- the provision of a well constructed excavation or formation of a reliable platform for the subsequent construction of engineering fill
- the provision of a sound and durable fill, capable of being handled, placed and compacted to a standard that is appropriate to the requirements of the earthwork as a whole and any structures that it may support
- provision of a means of compaction to enable the serviceability (and if necessary the ultimate) limit state criteria to be met.

For the construction of embankments or engineering fill below foundations three fundamental types of earthwork compaction specifications have been developed, namely:

- the method specification
- the end-product specification
- the performance specification.

A discussion of the various types of earthworks specifications is given by Parsons (1992) and BS 6031 (BSI, 2009).

Many countries have developed standard earthworks specifications over time, generally with respect to the construction of highways. These standard specifications have been used by other parties as a base document from which a more contract-specific specification has been generated.

In the UK, the Highways Agency has developed the SHW (HA, 1998, 2013). Other specifications have been developed in France, Spain, the USA and Russia to suit the particular climatic conditions and likely nature of earthworks materials.

The European Committee for Standardisation (CEN) is currently endeavouring to standardise earthworks specifications across Europe through its technical committee CEN/TC396. The progress of this standardisation process is reported by Gilbert and Kidd (2012).

In general, the standard earthworks specifications provide guidance on the winning of material from cuttings suitable for use as embankment fill and the provision of a stable formation. They also give guidance on pre-treatment of ground for embankment construction where the excavation of soft material or pre-earthworks drainage is required.

When contract-specific specifications are compiled these aspects are usually taken from standard specifications and modified to reflect specific site conditions. An example is presented in Appendix 1.

7.2. Method specification

A method specification defines how compaction should be conducted in terms of the types of compaction plant, the number of passes and the compacted layer thickness. The Highways Agency's SHW has been developed into a method specification from its 4th edition in 1969 (*Specification for Road and Bridge Works* – 'Pink Book'). The current format was first introduced in the 6th edition in 1986 ('Brown Book') based on work in the 1960s and 1970s by the Transport Research Laboratory. This work is collated by Parsons (1992).

The Highways Agency SHW classifies material to be used as engineering fill on its physical properties; for example, Class 1 granular fill and Class 2 cohesive fill. The specifier is required to supply material data to determine the nature of the material that would be acceptable for use as engineered fill. This would typically be grading, water content and optimum water content for granular materials and grading, water content, moisture condition value and/or plastic limit for cohesive materials.

The test data on the proposed fill source and the setting of acceptability limits characterise the material into a specific material class from which a specified method of compaction is required with respect to the depth of compacted layer, the type of compaction plant and the required number of passes.

For the SHW, materials are specified to Table 6/1 and methods of compaction are given in Table 6/4. Table 6/4 is summarised in Table 10.1 in Chapter 10.

The Highways Agency's SHW has predominantly been developed from previous monitoring with respect to material type and compaction method to produce a compacted material of less than 10% air voids, determined using a 2.5 kg rammer (BS 1377, Part 4 (BSI, 1990)) for general earthworks fill materials, Classes 1 and 2, and greater than 95% maximum dry density and generally less than 5% air voids for specialist fill materials, Class 6.

The French earthworks specification (Laboratoire Central de Ponts et Chaussées, 2003) is a method specification but soil classification is based on soil type and moisture condition, the latter in terms of 'wet', 'average' or 'dry' depending on the relationship between the actual water content and the optimum water content from the 2.5 kg rammer compaction test. Weather conditions are also considered in the classification in terms of the amount of precipitation and rate of evaporation to reflect the climatic conditions, particularly with respect to the more arid conditions and drier soils found in the south of France when compared to the UK.

Site testing is required with a method specification for site quality assurance and control to ensure that the prescribed compaction method is being correctly applied. This will take the form of testing the potential fill at the source or point of deposition with respect to acceptability and compliance with the specification limits and testing subsequent to compaction with respect to achievement of the required density. The form and types of testing are described more fully in Chapters 8 and 10.

7.3. End-product specification

For the end-product specification the designer defines a minimum density for the engineered embankment fill material. It is considered that a stable engineered embankment has been formed if this target value is exceeded. Three basic forms of end-product specification are commonly used, namely:

- relative compaction
- air voids
- relative density or density index.

When a target percentage of maximum dry density only is specified a stable fill may not result as compaction dry of optimum water content will achieve the density target but with high air voids (greater than 10%) that may become unstable if saturated. A maximum air voids value should also be quoted to produce a stable fill, see Charles *et al.* (1998).

It is also common that the stiffness of the compacted layer is defined in terms of Young's modulus derived from plate load testing of the compacted layer. However, stiffness alone should not be considered as this will not control the air voids in the compacted material, which is important in restricting the potential for loss of strength if an increase in the water content of the material occurs subsequent to placing.

The national specifications used in Spain (2003) and Russia (SNiP, 1987) require compacted density to be greater than 95% and 92–98% maximum dry density, respectively. The Spanish specification predominantly requires the use of the 4.5 kg rammer whereas the Russian specification predominantly utilises the 2.5 kg rammer.

The material acceptable for use as engineering fill is usually determined in a similar manner to that for a method specification. Compaction to achieve the required density with respect to layer thickness, plant type and weight and number of passes is the choice

of the contractor, but may be based on a method specification and verified by a series of site trials.

Generally, more rigorous testing is required for an end-product specification, both during site trials and during construction, than that required for a method specification.

An example of an end-product specification for engineered fill supporting low rise structures is given in Appendix 1 (based on Trenter and Charles, 1996).

7.4. Performance specification

A performance specification is normally designed in terms of the required serviceability limit state as follows.

- Maximum differential settlement not to exceed 1 : 200 over a defined length 5 years after construction and the maximum settlement in any one area not to exceed 25 mm.
- An allowable bearing pressure of 75 kN/m² with a settlement not exceeding 15 mm over the working life of the structure to be constructed on the compacted fill.

Care should be taken in the use of these types of specification as the use of an allowable bearing is settlement dependent and is also dependent on the size of the foundation applying the load. Testing for performance specifications may involve more than proof of acceptable compacted density. Zone loading testing, similar to that required for vibro-compaction, may be required to determine settlement of the compacted fill under applied load similar to that which will be applied under permanent loading conditions.

Changes in UK practice with respect to performance-related design concerning highway construction are detailed by Fleming *et al.* (2008).

This form of specification may be considered onerous from the contractual viewpoint as it seeks to place the risk of all future events on the contractor and may be very difficult to monitor in practice, see McGuire and Filz (2005). It is also unlikely to take account of future events over which the contractor may have no control, such as vegetation planted by the client that may produce volume change and result in greater movement of the material within the earthwork.

The development of continuous compaction control is likely to make the use of performance specifications more attractive in the future (see Section 10.7.6 for further details). A compaction control approach using modulus and air voids determination and monitored in real time using roller-mounted devices has the potential to mitigate many of the risks currently borne by the contractor under this form of specification (see Mooney and White, 2007).

7.5. Hybrid types of specification

There are also numerous hybrid forms of earthworks specification. The most popular approach is to use the SHW (HA, 1998, 2013) as the basis, while instituting relatively

small changes, affecting for example plant type, number of passes and layer thicknesses. These changes are often made from local knowledge of fill type and experience with particular equipment, but introduction of these changes may complicate the issue of liability under the contract and this should be borne in mind.

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Chapter 8

Earthworks fill design and setting specification limits

8.1. Introduction

8.1.1 Interaction between theory and practice

In no part of geotechnical engineering are design and construction and, in some instances, monitoring, more intimately linked than in earthworks. The control of fill placement may be considered an extension of design and is so treated here. The majority of earthworks (leaving aside cuttings and excavations to form the foundations for embankments, structures and landfills) deals with remoulded or partly remoulded soils. Consequently the fill, when placed, has to be continuously checked to ensure that the remoulded properties selected for design (such as: in situ density, air voids content, shear strength, stiffness, and moisture condition value (MCV) or California bearing ratio (CBR) values) are being obtained in the field. Good earthworks design goes further, requiring performance monitoring in addition to construction control.

Engineered fill may be defined (Charles, 1993) as fill that is selected, placed and compacted to an appropriate specification so that it will exhibit the required engineering behaviour. For a given project, the designer must normally

- consider the nature of the earth structure (and its intended function), the loading condition that will be experienced, and consequently the level of compaction required
- assess the factors affecting the ultimate limit states (ULS) and serviceability limit states (SLS) of the finished earthworks
- select suitable fill (to the extent that there is a choice available)
- select the initial water content for the level of compaction and the confining pressures concerned
- consider the use of treatment processes to improve fill material properties.

Preliminary design is done on the basis of site investigation information. However, the link between design and construction already mentioned means that information affecting design may be obtained from compaction trials and/or from the main works themselves. Therefore, earthworks design is often iterative, with the preliminary design altered as experience accumulates on site. This can have contractual consequences if the Contractor is asked to undertake work for which it has not priced; this should always be borne in mind when preparing the contract documents.

The earthworks designer will tend to draw on experience supported by relatively simple soils tests to consider the issues listed above. Relationship testing is often employed to assess the available data (as described in detail later in this chapter), but it is relevant to comment at the outset on an underlying theoretical problem for earthworks that is resolved by a practical solution. The problem is the lack of laboratory tests well suited to measuring the stiffness of re-compacted partially saturated soils as part of the relationship testing process at the design stage. Plate load tests can be undertaken during earthworks construction to give a subgrade modulus, which gives a reasonable indication of stiffness for geotechnical engineering purposes. The practical solution that the industry relies on at the design stage is to use correlations between simple tests such as undrained shear strength and CBR to give an indication of likely stiffness. It is recognised that this is not a good theoretical approach, but it is the best available in practice and so is followed in this chapter.

8.1.2 Comment on Chapter 8 layout

Updating the design-related aspects of this chapter has proved difficult for two reasons. First, although the first edition was only prepared 13 years before the second, there have been some fundamental changes in the industry that have a significant impact on various aspects of earthworks: the implementation of Eurocode 7 (BSI, 2004) (which changes many design concepts and soil descriptive terms); BS 6031 has been completely rewritten (partly to come in line with EC7); and the fact that most major projects are now run using design and build forms of contract. Second, earthworks are required across a wide range of situations, and different engineers bring experiences from different types of projects and fill types. The authors have sought to keep, as far as possible, to the original structure of the chapter and have added content where this is considered appropriate. Neil Trenter covered 'relationship testing' near the end of this chapter (see Section 8.4.8); however, this approach is the best option available for a designer to assess a fill's potential behaviour prior to the commencement of the earthworks on site. Therefore, examples based on relationship testing are included throughout this chapter to illustrate different fill behaviours.

8.1.3 The concept of 'design' and role of 'designer' in the field of earthworks

The form of contract used on a project has a fundamental influence on the earthworks (a factor that is commonly underestimated). Traditional forms of contract set clear roles for: the 'Engineer', who would normally be thought of as the 'designer' but only really designed the permanent works; and the 'Contractor', who would decide how to construct those permanent works and design any required temporary works. This seemingly clear division of roles is not ideal for earthworks, partly because the properties of a fill material depend on its water content and how it is handled. This commonly resulted in contractual disputes relating to earthworks.

The design and build form of contract can provide a good framework for the leading parties involved with earthworks design and construction to work together as a team, although unfortunately this does not always prove to be the case (Jarvis and Gilbert, 2003).

Whatever form of contract is employed the earthworks design is an iterative process in which design decisions are often taken by various parties: employer, consultant, main contractor, sub-contractors and the construction validation team (BSI, 2009). The Construction (Design and Management) Regulations (CDM) 2007 set a very broad definition of those who will be acting as a ‘designer’. Designers are those who prepare or modify designs for construction work, which includes: preparing drawings, design details, specifications, bills of quantities, as well as the related analysis, calculations and preparatory work. Effectively any party who makes a decision about what is to be constructed (whether the instruction is communicated in written form or orally) is acting as a ‘designer’ (HSE, 2007).

Following the spirit of CDM the second edition of this book has been modified to use the contract neutral phrase ‘designer’, and assumes that there may be more than one earthworks designer on any single project.

BS 6031:2009 covers all aspects of earthworks design, which can be separated into two distinct elements: geotechnical design and fill design for earth structures, which are summarised in Figure 8.1, and are discussed further in Sections 8.3 and 8.4 of this chapter.

The delineation shown in Figure 8.1 will be used in future European earthworks standards (currently in preparation). These standards will cover earthworks fill design and earthworks construction practice (to complement the geotechnical design within Eurocode 7) (Gilbert and Kidd, 2012).

A common misconception in the civil engineering industry is that the earthworks design is finished once a required end product is specified, and that how to achieve this can simply be passed to the ‘earthworks contractor’ as being construction. Yet the earthworks contractor must then determine which fill material to use, and how best to treat, place and compact that fill to meet the specified parameters; this should be determined based on the information available in a logical manner prior to placing the fill and therefore can be considered as a design process.

Figure 8.1 Summary of earthworks design elements

BS 6031:2009 covers all aspects of earthworks design, which can be summarised as follows

- **Geotechnical design:** determination of the earth structure requirements in terms of stability and deformation (as covered by Eurocode 7), including defining the required long-term soil properties (such as strength, stiffness and permeability) and drainage for the finished earthwork.
- **Earthworks fill design:** the assessment and selection of fill material properties to enable construction and to satisfy the required long-term properties for the earth structure; and establishment of an appropriate earthworks specification covering fill properties and compaction requirements.

8.2. Investigations for earthworks

In principle, a site investigation for any earthwork structure should provide the following:

- Information to assist with planning of the project, which can be at various levels including: formal planning approval processes with regulators, early stage considerations to help minimise environmental impact and assessing an optimum muck shift programme.
- The information required by the designer to design the permanent earthwork, which will tend to be focused on the geotechnical design to achieve long-term stability (to satisfy the requirements of EC7).
- Data on fill material properties to enable a specification for the earthworks to be developed that will satisfy the required geotechnical design properties and maximise the use of locally available fill material.
- The information that will be needed by other parties involved with elements of design: for example, the selection of fill materials to satisfy the permanent works, design temporary works, etc.
- The information required by the contractor to plan the site works, determine the optimum muck shift strategy and programme all the various construction activities, including the temporary works.

Most structures involving earthworks, like roads, railways and airports, landfills and residential, commercial or industrial estates, require other features to be investigated. They include

- cut formation level for pavement design and cut slopes for stability
- landfill liner foundation level for settlement and possible heave
- embankment foundations for bearing capacity, settlement and stability
- strength and compressibility characteristics of the natural soils forming the foundations to the fill platform.

These features require the characterisation of the soil and rock in their natural rather than in their remoulded condition. Earthworks are near surface structures: embankments, for example, are usually placed at as shallow a depth as the stability, bearing capacity and settlement properties of the underlying soils permit. Although deep cuts are the obvious exception, even they have shallow transitions at the entry and exit points. For this reason, the natural ground at shallow depth is of great interest to the earthworks Engineer, who concentrates on shallow depth features, such as

- artificial changes to the ground by human activity (excavation, filling and mining)
- fluvial and solifluction deposits and/or glacial or periglacially induced soil fabric.

The site investigation for an earthworks project should utilise a range of methods to obtain the required information, these will normally comprise: desk study, site inspection, ground investigation site work and laboratory testing. In most cases this part of the project will be undertaken in accordance with Eurocode 7 (BSI, 2007), a commentary on how to apply the 'dry rules' within EC7 for the specific needs of an earthworks project is clearly set out within BS 6031 (BSI, 2009), with Clause 6 of that document covering site investigation.

The desk study should be fully utilised to obtain existing information on the site. For projects that affect, or are close to existing earthworks, the as built records and any asset management records held by the earthwork operator should be obtained to understand any known geotechnical issues. The site inspection is an opportunity to gather information about both the site and the adjacent land; the use of geomorphological mapping techniques can often prove advantageous in order to capture the site information in a systematic manner and identify features of interest.

Following the desk study and site inspection it is advisable to prepare a preliminary geotechnical risk log in order to capture the various issues that should be covered by the ground investigation. The ground investigation should then be designed in order to optimise the information obtained and to clarify the geotechnical risks identified. Whoever designs the ground investigation should endeavour to capture the data that other parties will require at some stage (e.g. a contractor who will plan the construction works).

The fieldwork and testing phases of the site investigation for earthworks should, whenever possible, capture information relating to the following:

- Ground investigation by trial pits to enable: visual inspection of soil fabric, and the recovery of large bulk disturbed samples of soils for earthworks laboratory testing of recompacted soil.
- Ground investigation by boreholes for both geotechnical and earthworks design purposes (with bulk disturbed samples recovered for earthworks laboratory testing of recompacted soil).
- Investigation of potential fills over whatever depth is necessary to secure the required quantity of fill material. This may be limited to the extent of the cut depths, where balanced cut and fill is planned. Where this will only yield part of the total fill requirement, there should be investigations at other borrow locations.
- The natural ground forming the cut slope and the cut formation, together with foundations for structures like embankments and landfill liners.
- Groundwater conditions for all structures, noting that artesian (or sub-artesian) pressures can cause significant problems because of the possibility of uplift pressures (e.g. for deep excavations, landfill liners, etc.).
- The potential for interlayered soils of variable permeability, as these can provide an opportunity for excess pore water pressures to be generated during construction resulting in the potential for both softening of the ground and slope instability.
- The presence or otherwise of sulfate or sulfide-bearing materials in the proposed fill, which could have deleterious consequences for buried concrete or for lime-stabilised work such as capping layers (see Section 3.5.2) (Czerewko *et al.*, 2011).
- The potential for physical and chemical changes following excavation of a fill from a cutting, particularly if over-consolidated clays or weak rocks are proposed for use as fill materials.
- Laboratory testing to assess earthworks properties (with an emphasis on the relationship testing approach described at Section 8.4.8).
- Instrumentation to determine the groundwater conditions (including any perched water zones) and whenever possible record the likely seasonal variations.

The text *Effective Site Investigation*, prepared by the Site Investigation Steering Group (2013), provides a valuable source of information on site investigation in general. Clayton *et al.* (1995) is a slightly older text but is still frequently consulted by practising Engineers.

8.3. Preliminary design considerations

This section considers the preliminary design that is undertaken based on site investigation information (i.e. prior to any earthworks construction). It includes the range of topics that the earthworks designer will need to consider to assess the earthworks issues that will be relevant to the project. It is at this stage that the geotechnical design requirements for the earthworks are considered (see Figure 8.1).

8.3.1 Fill suitability

A fundamental part of the design process is fill selection. Fill may be regarded as both ‘unsuitable’ and ‘suitable’ and, within suitable fill itself, ‘hierarchies’ of suitability for various engineering purposes exist. Note that the *Specification for Highway Works* (HA, 1998, 2013), referred to here as ‘SHW’, recognises ‘acceptable’ and ‘unacceptable’ fills, as defined in its Clause 601. These terms are only used in this book when direct reference is being made to satisfying a specification requirement. In all other cases the broader terms ‘suitable’ and ‘unsuitable’ are adopted to indicate whether a material should be suitable to use as fill in some form.

8.3.1.1 Unsuitable fill

A list of unsuitable/unacceptable materials based on the SHW is as follows

- clay soils having a liquid limit (BS 1377:1990: Part 2: Test 4 (BSI, 1990)) in excess of 90% and a plasticity index (BS 1377:1990: Part 2: Test 5 (BSI, 1990)) in excess of 65% (some Engineers might find these figures rather high and would prefer to adopt lower limits, especially in wetter terrains, or to contemplate the use of lime stabilisation procedures)
- peat, materials from swamps, marshes and bogs (i.e. bio-degradable materials)
- logs, stumps and perishable material
- material susceptible to spontaneous combustion (except adequately compacted unburnt colliery spoil)
- materials in a frozen condition
- contaminated materials, including controlled wastes (as defined in the Environmental Protection Act 1990 Part IIA) whose level of contamination is above the level stated in the specification
- hazardous waste (as defined in the Hazardous Waste (England and Wales) Regulations 2005) and radioactive waste (as defined in the Radioactive Substances Act 1993).

It is also important to note that the permitted constituents for each class of fill are defined with Table 6/1 of the SHW; for example, argillaceous rock is excluded for structural fill. For many classes of fill, potentially damaging sulfate and sulfide-bearing materials are excluded.

The SHW distinguishes between unacceptable fill that can be treated and rendered acceptable for the permanent works and unacceptable fill that cannot. The latter includes fill having hazardous chemical or physical properties, which requires special measures for its handling and disposal.

The above list provides a useful summary of materials that are not suitable for use as engineered fill, but in some cases the list may need to be more extensive to suit more demanding end uses of the earthwork, such as high-speed railways.

8.3.1.2 Suitable fill

Deciding on suitable fill is one of the key operations in an earthworks project. In practice, fill selection is a matter of deciding what is unsuitable on a site and making the best of the materials remaining. Factors that may influence a designer's decision on suitability will include:

- the practicability or otherwise of conditioning to acceptable limits (see Section 10.5)
- haul distances and the ease of transport of alternatives
- the influence of construction traffic on the fill
- frost-heave susceptibility
- presence of deleterious substances such as sulfate/sulfide-bearing materials.

There are substantial environmental pressures to avoid importing large quantities of fill from off site and, unless there is a shortfall in volume, the designer will be expected to persevere with conditioning on-site materials to the extent necessary to provide the required material.

8.3.1.3 Hierarchy of fills

Depending on the engineering requirements, there is usually a hierarchy of suitable fills at a given site, drawn up on the basis of their potential use. The following is one example that may be relevant to a larger fill operation:

- rock armour: coarse angular rock fill of both high strength and durability
- structural fill: for such uses as corrugated steel structure surround, reinforced earth and gabion infill
- road aggregates: sub-base and surfacing material for unpaved roads
- capping and drainage: suitable to form capping or drainage layers in cuts or embankments
- bulk fill (1): best quality; suitable for use in embankments (particularly close to structures), and to support shallow foundations
- bulk fill (2): as above, but requiring conditioning prior to use
- bulk fill (3): suitable for mine-shaft, mine workings or other bulk infill (may need some structural fill depending on the nature of the workings)
- containment fill: suitable to form containment bunds in which landscaping fill may be placed. Must be sufficient for compaction in thin layers to form satisfactorily stable side slopes
- landscaping fill: suitable fill that may be used to form landscape areas

- agricultural or horticultural fill: otherwise unacceptable fills but suitable for soiling or topsoiling landscape and other areas
- unsuitable fill: fill incapable of being processed to form material suitable for use in the works.

It will be evident that further hierarchies exist within the above classification. For example, the best containment fill would be used alongside public spaces or close to sensitive structures where stable side slopes were necessary, while less good material would be used in areas inaccessible to the public.

8.3.2 Factors affecting the ultimate limit states (ULS) and serviceability limit states (SLS)

When considering the suitability of a fill material for a given earthwork the designer must consider whether the soil properties of that fill will be sufficient to satisfy the ULS and SLS. Details of these limit states are set out within Bond and Harris (2008). This section provides an interpretation from an earthworks perspective. The earthworks designer normally undertakes the design of an earth structure and defines the required properties that must be satisfied by the fill within the earthworks specification for the works.

Defining the fill properties required to satisfy the ULS is easier than for the SLS. The common ULS conditions relating to fill properties are as follows.

- The ULS requirement for global stability of the earthwork during construction and on application of permanent loading (as a consequence of the end use of the earthwork) will dictate the required fill's strength properties. The fill material's strength properties are assessed based on undrained shear strength and angle of friction in the drained condition. These properties will reflect the index properties of the soil, particularly water content, plasticity and grading.
- Some engineering structures will include a ULS condition in which failure is judged based on exceeding a defined movement tolerance on first time loading. In these cases the relevant fill properties will be stiffness of the fill. The difference here between ULS and SLS is essentially related to the time period or number of load cycles before the movement becomes unacceptable.
- The required function of the earthwork, or particular zones of fill, will often dictate permeability requirements. Some fill zones may be required to perform functions such as: drain water, prevent the build-up of excess pore water pressures, or act as a low permeability barrier. This can be considered a ULS condition and will be dominated by the soil's grading (particularly the fines content); and for low permeability soils will also depend on plasticity, water content and the degree of compaction achieved.
- An additional property to consider for granular fills is the strength and durability of the particles; this will influence both ULS and SLS criteria.

The SLS is normally considered to be governed by deformation under operational loading conditions. The deformation will occur sometime after the initial loading of the

structure, and will be sufficient to influence performance adversely, but insufficient to be assessed as failure of the ground. The most common SLS criteria of settlement due to self-weight of the fill and load placed on that fill can be relatively easily calculated as part of the geotechnical design. Designers should consider other ground risks that could influence performance during the earthworks operation, including:

- changes in effective stress: due to variations in groundwater level within the fill or to changes in imposed loading for which the fill was not designed
- changes in fill water content (a): due to inundation caused by a rising water table or by concentrated run off into low-lying filled areas causing collapse settlement (especially relevant to cohesive fill and to some weak rocks) (Charles, 2005)
- changes in fill water content (b): due to vegetation growth or by climatic variations; settlement due to desiccation together with heave due to water ingress can occur (especially relevant to cohesive fill)
- chemical changes leading to heave: many chemical changes in potential fill (natural and industrial materials) are expansive and can cause heave of the fill and any overlying structures (this is a particular concern for over-consolidated clays and mudstone and industrial slag, see Chapters 3 and 6)
- chemical changes due to oxidation: some fill materials such as mudstones, and glacial sands and gravels contain pyrite or other sulfide/sulfate minerals that can oxidise as a result of excavation, triggering acidic reactions that can be extremely corrosive and give off significant heat in the process (Czerewko *et al.*, 2011)
- changes due to decomposition of organic matter: such changes cause volume reductions as well as gaseous omissions; fill containing a significant organic matter content is generally considered to be unacceptable other than as landscape fill
- erosion potential: although the engineering characteristics of granular fill, when placed and compacted to appropriate standards, are generally good, the erosion potential of silty soil and uniform fine and fine to medium sand is relatively high and special measures may be necessary (see Section 4.5.4).

The designer should consider each of the above factors in relation to the structure, identifying any that could impact on the SLS and the appropriate precautions taken.

It will be evident from the discussion above that various soil properties have an influence on either the ULS or SLS, and many properties influence both states at different times. This situation is complicated by the fact that soil properties can change with time, a problem that could not be predicted by civil engineers until Karl Terzaghi's pioneering work to establish the science of soil mechanics in the first half of the twentieth century, which demonstrated the fundamental role of pore water pressure and seepage.

Experience shows that addressing the drainage requirements is fundamental to ensuring the stability of any earthwork. The earthworks designer should consider the appropriate drainage measures at different stages: pre-earthworks, during construction and in the long term; and give particular consideration to maintenance requirements and the potential consequences of a failure of some element of the drainage system. BS 6031:2009 Clause 7.5 should be referred to for guidance on this topic.

Figure 8.2 Definitions of fine grained soils for geotechnical applications

1. For geotechnical design a soil is generally treated as being fine grained if it has >35% fines.
2. During earthworks construction any fill with >15% fines is classified as fine grained, commonly referred to as 'cohesive' fill.

In earthworks the changes in fill properties and behaviour with water content are most evident for fine grained soils. In the field of earthworks the assessment of whether the fines fraction (that passing a 63 μm sieve) is sufficient to determine the engineering properties of the soil will differ between two major cases as shown in Figure 8.2 (BSI, 2009). The difference in the two cases is because of the dynamic loading conditions experienced during earthworks construction, which can result in elevated pore water pressures, even at relatively low fines contents. See Section 3.1.2 for further commentary on this topic.

The following Sections, 8.3.3 to 8.4.7, review various soil properties that are particularly significant to earthworks. These could be treated in many orders but we have continued to follow the sequence adopted by Trenter in the first edition of this book. At Section 8.4.8 the important topic of relationship testing is presented, which is the key to mastering earthwork fill behaviour, especially for fine soils.

8.3.3 Nature of structure and compactive effort required

The nature of each earth structure and the intended end use of that earthwork should be carefully considered, particularly its sensitivity to settlement and differential settlement. Consideration should be given to the acceptable magnitude of self-weight settlement within the body of fill material during and after construction. If the intended end use of the earth structure has strict settlement tolerances then it will be necessary to give careful consideration to the permissible fill materials utilised and the compactive effort to be applied to those fill materials (clearly the depth of fill material is also an important factor).

It is important to remember that the compaction methods set out within the SHW (HA, 1998, 2013) for general fill (bulk fill) were determined for UK soils placed as fill material at an appropriate water content to form embankments of up to 10–15 m high to carry highways. This end use means that a limited amount of post-construction settlement can be tolerated within general fill. For fills around structures there is a need for a greater degree of compaction to be achieved, which is generally set based on end product compaction requirements. This can be illustrated by summarising the target criteria for some of the standard compaction methods of SHW Table 6/4, which assumes the specification water content limits are appropriately established (HA, 1995), along with some of the end product requirements of SHW Table 6/1 for comparison.

- Methods 1, 2 and 4: aim to limit air voids to 10%, which was judged applicable for well-graded general granular fill, most general cohesive fills and chalk.

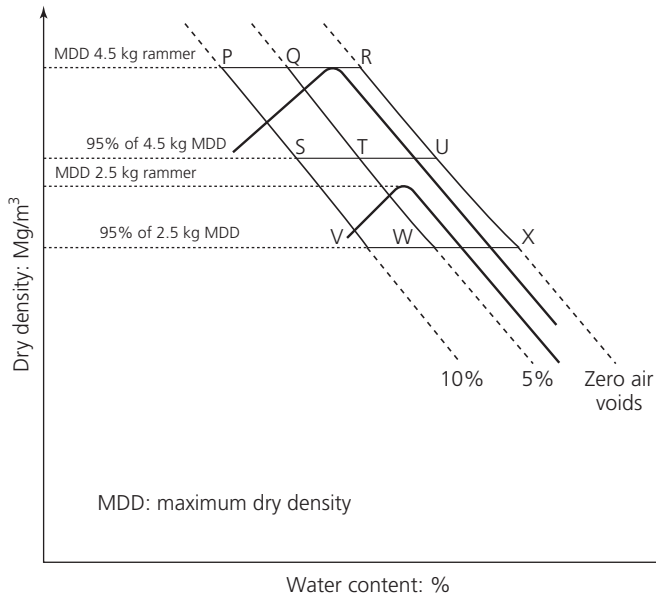
- Method 3: was established to achieve 95% of standard Proctor maximum dry density (MDD), it is applicable for uniformly graded granular fills (Class 1B, 6C and 6J) and silty general cohesive fill.
- Method 6: aims to limit air voids to 5%, it applies to capping (Class 6F).
- End product requirement for selected granular fills to structures (Classes 6M, 6N and 6P) is 95% of MDD by the vibrating hammer method (i.e. the most demanding compaction test method).
- End product requirement for selected cohesive fill to structures (Class 7A) is 100% of standard Proctor MDD.

While the standard compaction methods set out within the SHW form a useful basis for a relatively wide range of end uses, the designer must consider whether these methods are appropriate for their particular earthwork. Some examples of cases where the SHW methods might not be appropriate include the following.

- Flood defence earthworks can tolerate a lower compactive effort because the nature of the structure is such that the focus should be on achieving low permeability of the placed fill, which will tend to mean working wet of optimum water content (OWC).
- Use of over-consolidated clay or weak mudstone fills where careful consideration must be given to the potential for heave of the fill post-excavation, rather than simply focusing on compaction relative to MDD of the excavated material.
- Embankments greater than 15 m high may require a higher compactive effort (depending on the nature of the fill placed).
- Infill of deep excavations that are likely to become inundated by rising groundwater post-construction will generally require a higher compactive effort to prevent collapse settlement on inundation. Achievement of 95% of MDD will not be an adequate control; an additional requirement of achieving less than 5% air voids will be appropriate (BSI, 2009; Charles *et al.*, 1998).
- Earthworks platforms constructed to enable the construction of structures with shallow foundations within that fill (such as buildings and industrial facilities) will require a high compactive effort to limit settlement within the fill due to the structural load applied.
- High-speed railway embankments where the required stiffness of the fill must satisfy settlement criteria, and also dynamic load criteria relative to Rayleigh waves.

For the case of building structures it is relatively easy to obtain guidance on the permissible differential settlements that the structure can tolerate; for example, the structural engineering aspects of low rise structures may be obtained from Tomlinson (1995), Tomlinson *et al.* (1978) and Atkinson (1993). These references can be used to help assess the permissible settlement of the ground beneath the structure, and therefore if the structure is to be constructed directly on an earthworks platform it is possible to determine the tolerable settlement within that body of fill. This settlement information can be used by the designer to help assess the appropriate type of fill and compactive effort required.

Figure 8.3 Selection of compactive effort for engineered fill. For explanation see text (modified after Trenter and Charles, 1996)



A problem that designers face before construction is that the methods available for assessing the required compactive effort for a particular fill to achieve a desired stiffness are relatively limited. Relationship testing is the best approach, but practicalities of what test can be readily undertaken on each compacted sample leave the designer having to utilise empirical correlations between soil properties, such as undrained shear strength and stiffness. Although far from ideal, it is an adequate approach for most earthworks purposes, but becomes inadequate for more complex cases such as high-speed railway engineering.

As a general guide to assess the appropriate level of compactive effort for a given fill, the method illustrated in Figure 8.3 may be followed. Figure 8.3 illustrates two compaction curves corresponding to the 2.5 and 4.5 kg rammer methods, together with the 0%, 5% and 10% air voids content lines. For this approach the 4.5 kg rammer test method is used because for general fill it is still considered to be representative of the highest level of compaction that can be achieved on site using standard earthworks heavy compaction plant. The areas enclosed between the maximum dry densities for the 2.5 and 4.5 kg rammers, 95% of the maximum dry densities for the 2.5 and 4.5 kg rammers and the air voids content lines may be subdivided as shown.

The consideration of air voids is standard practice in the UK, and is enshrined within the method compaction approach set out within the Highways Agency's SHW (as summarised above). The SHW standard compaction methods were determined based on extensive testing by the Transport Research Laboratory (TRL). However, it is important to remember that some fill materials will not readily fit this generalised model.

Furthermore, the calculation of air voids requires accurate determination of the specific gravity applicable for the fill material, which can prove to be very difficult. Typical specific gravity values for a range of minerals and rocks are discussed in Section 8.4.3 (Table 8.3).

Depending on the nature of the problem in hand, the following levels of compactive effort may be used as a general, albeit relatively conservative, guide:

- area QRTU (air voids content 5% or less): foundations for buildings (granular fill); capping layers (from selected granular fill); granular fill on the passive side of a gravity retaining structure
- area TUWX (air voids content 5% or less): foundations for buildings (cohesive fill); capping layers (lime stabilised cohesive fill); general cohesive fill for 600 mm depth below a capping layer; granular fill for the active side of gravity retaining walls; cohesive fill for both sides of gravity retaining walls; fills liable to collapse settlement with sensitive overlying structures
- area STVW (air voids content between 5% and 10%): mass fills for embankments (granular and cohesive), not liable to collapse settlement. Consider upgrading to area TUWX for mass fills for railway and airport runway embankments
- area PQST (air voids content between 5% and 10%): mass fills for embankments (granular and cohesive) provided checks are undertaken regarding the risk of post-construction changes in volume (see Section 8.3.4)
- area below WX (air voids content 5% or less): clay fills in this zone may perform well as low permeability barriers such puddle clay and even flood bunds
- area below SVWX: non-structural fills for landscaping purposes (where air voids greater than 10% will generally be acceptable).

The general guidance set out above is deliberately conservative to minimise the risk of fills being used for an inappropriate purpose. The designer should not blindly follow this guidance; instead the limits of the various areas, quoted above, may be altered to suit circumstances. For example, 90% of the 2.5 kg rammer method may be substituted for 95%, if conditions warrant. If a particular fill appears suitable for a given end use then the designer should obtain appropriate laboratory test results to judge whether it is appropriate to deviate from the general guidance. For example, some reasonably well-graded granular fills will prove very difficult to compact to 5% air voids, and uniformly graded sand will not achieve 10% air voids regardless of the compactive effort, yet both can provide an acceptable structural platform. It is good practice to include a field trial at the start of the works to check whether the fill behaves as expected under site conditions.

Different recommendations are given for foundations on granular and on cohesive fills, because cohesive fills can suffer changes in water content and volume post-compaction either as heave or settlement (see Section 8.3.4). The designer should assess the potential for such changes. Heave is best avoided when the fill is to carry structures, and in the case of settlement designers can follow the criteria of Charles and Burland (1982). These authors suggest that for settlements corresponding to volumetric strains less than 0.5%, fill should provide a good foundation for overlying structures, and that at less than

1% problems are unlikely. At settlements corresponding to volumetric strains in the fill greater than 2%, the authors suggest that there might be serious difficulties with overlying structures. Laboratory tests using the procedures outlined by Head (1994) should always be conducted to determine whether or not the given fill, compacted to the efforts described above and to the water contents outlined in Section 8.3.4, will meet these criteria. Note that BS 1377:1990: Part 5: Test 4 (BSI, 1990) also describes tests to measure swelling pressure and settlement on saturation.

The weight and vibration of construction traffic can be a problem when compacting hard up against a structure, whether existing or under construction. For this reason lower compactive efforts may be appropriate for fills of all types on the active side of such structures, including gravity retaining walls; for fills on the passive side, higher compactive efforts can be applied (strength of the structure permitting), but the possibility of heave occurring with low water content cohesive fills placed at high compactive effort should be borne in mind (see also Section 10.6.3). Depending on construction sequence, propped retaining walls are likely to be less sensitive to compaction method.

8.3.4 Initial water content and changes post-construction

8.3.4.1 General

Fine grained fill materials (>15% fines fraction) tend to be the most complex earthworks fill materials because the fill's behaviour will vary greatly depending on the water content, both during and after compaction. Initial water content must be considered in light of the compactive effort and the confining pressure. The relationship between these three factors was discussed in Section 3.5.1.

A common cause of disputes during earthworks projects occurs because the suitability of a soil for use as fill is assessed during the design stage, based on soil properties revealed by the ground investigation, but the final suitability will be dependent on the construction methods employed by the earthworks contractor. The most important property to change as a result of the earthworks processes is the water content. A fundamental aspect of good earthworks practice is the management of surface water and groundwater to prevent deterioration of the ground, as discussed in BS 6031:2009 Clause 7.5.4 (BSI, 2009). If appropriate measures are not employed then the water content of soil can increase prior to it being used as fill, the consequence of which will be that the strength of the fill material will be lower than anticipated at the design stage. Some soils are described as being moisture susceptible because a small change in water content will result in a significant change in strength. The most extreme cases tend to be silts, but various soil conditions can lead to such problems and the best way to assess the risk in advance tends to be by means of trial pits extending into saturated soils. This chapter is written from a design perspective, exploring the changes in physical properties of a fill that can occur as a consequence of changes in moisture, with the focus being on changes after the fill is placed in the works.

Fine grained fills that have sufficient fines to behave essentially as a silt or clay soil in engineering terms after the earthwork construction is complete (typically >35% fines fraction, see Figure 8.2) require the greatest consideration by the designer. If these fills are compacted dry of OWC then there will be a potential for changes in physical state due

to changes in water content after the earthwork has been constructed (post-compaction), in particular significant loss of strength, collapse settlement or heave. The potential for such changes is greatest in fine fills with a high clay mineralogy (and hence high plasticity), because these soils show a natural tendency for the soil water content to increase towards the plastic limit. When using clay fills and the water content is dry of the OWC determined by the standard Proctor compaction test (which as a rule of thumb is commonly around plastic limit minus 4%), consideration will need to be given to whether the confining pressure is too low to resist the soil suction pressure.

Working with clay soils wet of OWC presents other construction difficulties such as: over-compaction, matting, rutting and the formation of construction shear surfaces (see Section 3.4).

Confining pressure depends on the overall design of the structure (e.g. the height of the embankment or the depth of the excavation to be backfilled) and is something over which the geotechnical engineer normally has little control. However, there may be some room for manoeuvre with compactive effort and initial water content because they can often be manipulated to give the best engineered fill for the purpose. An additional aid is the possibility of stabilisation.

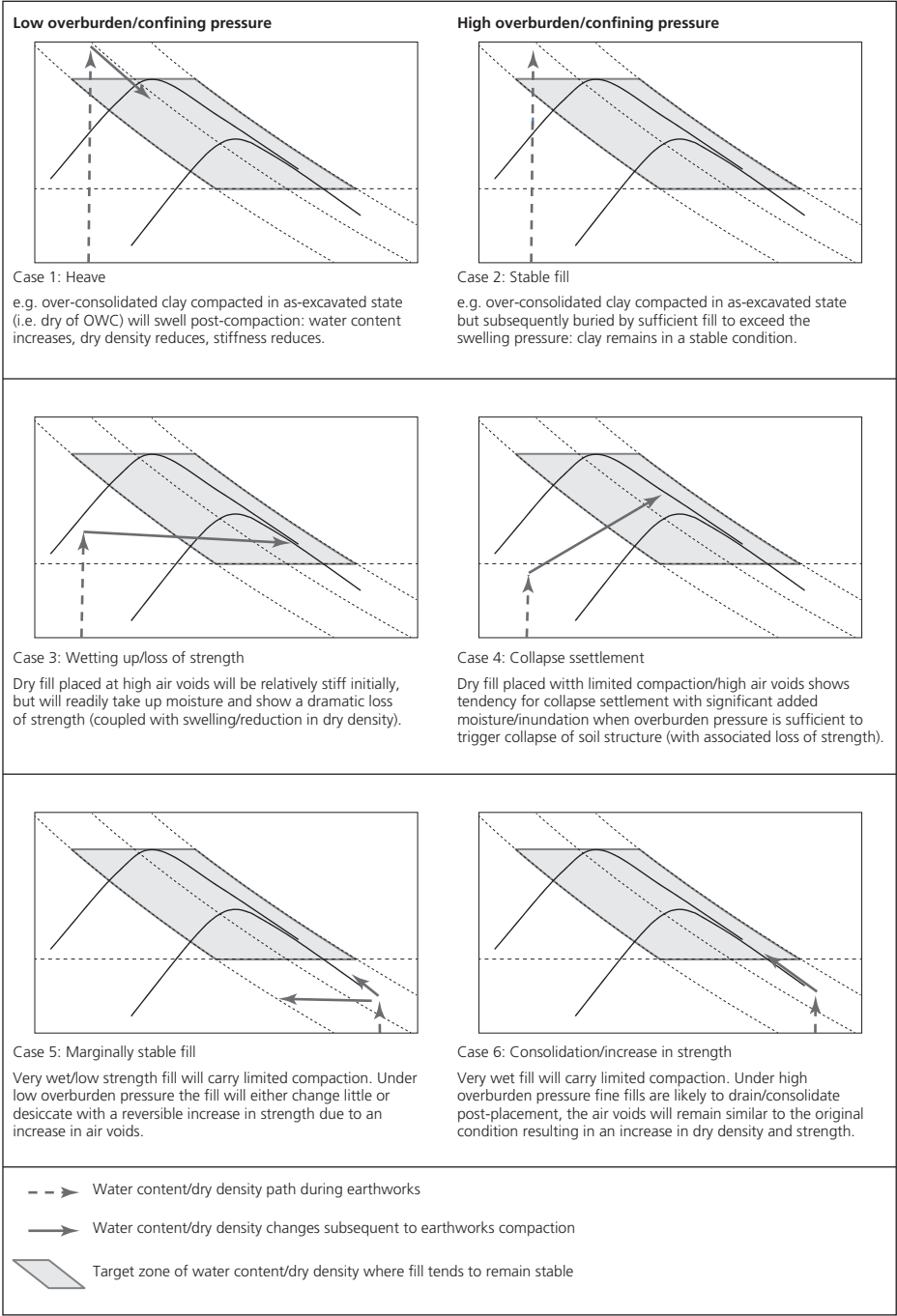
In soil mechanics confining pressure within an over-consolidated clay can exceed the overburden pressure, but in an earthworks context the past history of the soil or rock is largely lost on excavation. Consequently the earthworks designer will generally work in terms of overburden pressure within a body of fill, and give consideration to the potential for lumps of intact soil or rock to swell.

Figure 8.4 provides a simplistic (hypothetical) summary of some of the main changes in physical state that can occur with fine grained fills as a result of changes in water content, after the fill has been placed and compacted. The intention of this figure is to help the reader visualise potential changes in water content, dry density and air voids, which might occur if the compacted fill is outside of the normal 'target zone' described in Figure 8.3. The assumption made for this generalised summary is that a fill placed within the target zone will remain in a stable state. The solid arrows showing changes are only intended to represent the typical direction of change and order of magnitude of potential change. The illustrations on the left side of the figure are all for fill remaining under a low overburden/confining pressure, while those on the right are under a high overburden/confining pressure. A brief description of the likely changes in overall volume and strength/stiffness of the body of the fill is included under each illustration.

Figure 8.4 should only be used as an illustration to help visualise the risks for a given fill. Design decisions for potentially difficult fills should be based on relationship testing and, where appropriate, laboratory testing to assess collapse/heave potential for the fill under consideration.

The remainder of this section gives a more detailed description of some of the main issues illustrated in Figure 8.4. However, it is important to remember that the assessment of

Figure 8.4 Idealised illustration of potential post-compaction changes of state for fine fills compacted outside the target zone (air voids lines shown: 10, 5 and 0%)



likely behaviour based on ground investigation data needs to be confirmed by actual fill behaviour during full-scale earthworks construction, at which stage the designer may need to revisit the data (see Section 8.1.1). Effectively, earthworks are both ‘art’ and ‘science’; this section explores the science, which is fundamental and should be used as a guide to what might happen during the art of successful earthworks operations on site.

8.3.4.2 Reduction in soil strength

In extreme cases, some fine fills placed dry of OWC can be prone to collapse settlement, and those with a high clay mineral content are also at risk of heave (see cases 1 and 4 in Figure 8.4); these volume changes will result in an associated loss of strength post-construction.

A far more common problem in civil engineering is that relatively fine grained fills (where the clay, silt and fine sand fraction dominate the behaviour) are often compacted dry of optimum in order to achieve a high strength/stiffness during construction. However, if the compacted fill has a high air voids content then this initial high strength will be lost when the water content of the soil increases, as illustrated by case 3 in Figure 8.4. The higher the air voids the more dramatic the loss of strength (coupled with slight swelling/reduction in dry density). This will be a particular problem for pavement engineering as the greatest strength loss will be at very shallow depth (i.e. low confining pressure). The reason for the loss of strength of the type shown in case 3 of Figure 8.4 is illustrated with some example data that are set out in the form of a relationship test at Figures 8.5 and 8.6.

Over-consolidated clays and weak mudstones when excavated will have a low water content and high strength and are difficult to compact. If poor compaction is achieved in the field then the result is often large air voids between intact lumps of the original material; with time those lumps will tend to break down resulting in an overall change in volume and a considerable reduction in strength. The lumps of soil will expand, but if the voids between the lumps are large then this expansion may simply fill those voids (effectively away from the contact points between lumps the soil is under a low confining pressure), making it difficult to predict the overall change in soil mass volume. The stiff clays and soft argillaceous rocks of eastern Europe that are described as ‘flysch’ are particularly problematic in this regard; one problem site in Slovenia recorded water content at the time of construction of 15–20% increasing to an average of 24–30% one year after embankment construction, with significant reduction in dry density and large lenses of very soft clay (Petkovsek and Majes, 2012).

8.3.4.3 Collapse settlement

Collapse settlement may be a problem for cohesive fill (and some weak rocks) placed dry of optimum, which has been relatively lightly compacted leaving high air voids within the completed fill. For collapse settlement to occur there is a need for an amount of overburden pressure sufficient to trigger collapse and an increase in water content (see case 4 in Figure 8.4). The magnitude of the settlement is governed primarily by the soil properties, degree of compaction and thickness of the layer that becomes saturated

Figure 8.5 Case study: relationship testing to assess loss of strength due to soil saturation (see Figure 8.6 for results)

Relationship testing was undertaken on a low plasticity fine sandy silt/clay (PI of 9%).

The compaction tests were undertaken in CBR moulds to enable the CBR testing of each compacted sample. Three additional samples were prepared dry of optimum water content, using the same compactive effort, and then soaked (following the BS 1377 method) and a soaked CBR value determined. While this test method is very onerous it has the advantage of showing a lower bound value that might be experienced at a poorly drained site. The resulting strength loss is shown in Figure 8.6, e.g. sample 2 was prepared at OWC -3.5%, which resulted in 15% air voids, this gave an unsoaked CBR of 10%, but during soaking the water content increased by 8% up to almost complete saturation (2% air voids as shown on the compaction curve); the sample swelled slightly (as shown by the slight reduction in dry density); and as a result the soaked CBR reduced to just 3%.

On the relationship test plot the soaked CBR curve is plotted relative to the water content that the sample was prepared at so that the worst case reduction in CBR that could occur during operation is clearly visible. The horizontal construction lines on the CBR graph show that the soaked CBR values plot slightly above the unsoaked CBR curve, i.e. some of the benefit of the compaction effort has been retained.

It is relevant to note that in this example all of the soaked CBR values recorded were either at or below the long-term 'equilibrium moisture content CBR' that could be predicted for this fill based on plasticity (HD25/94 (HA, 1994b) and LR 1132 (Powell *et al.*, 1984)).

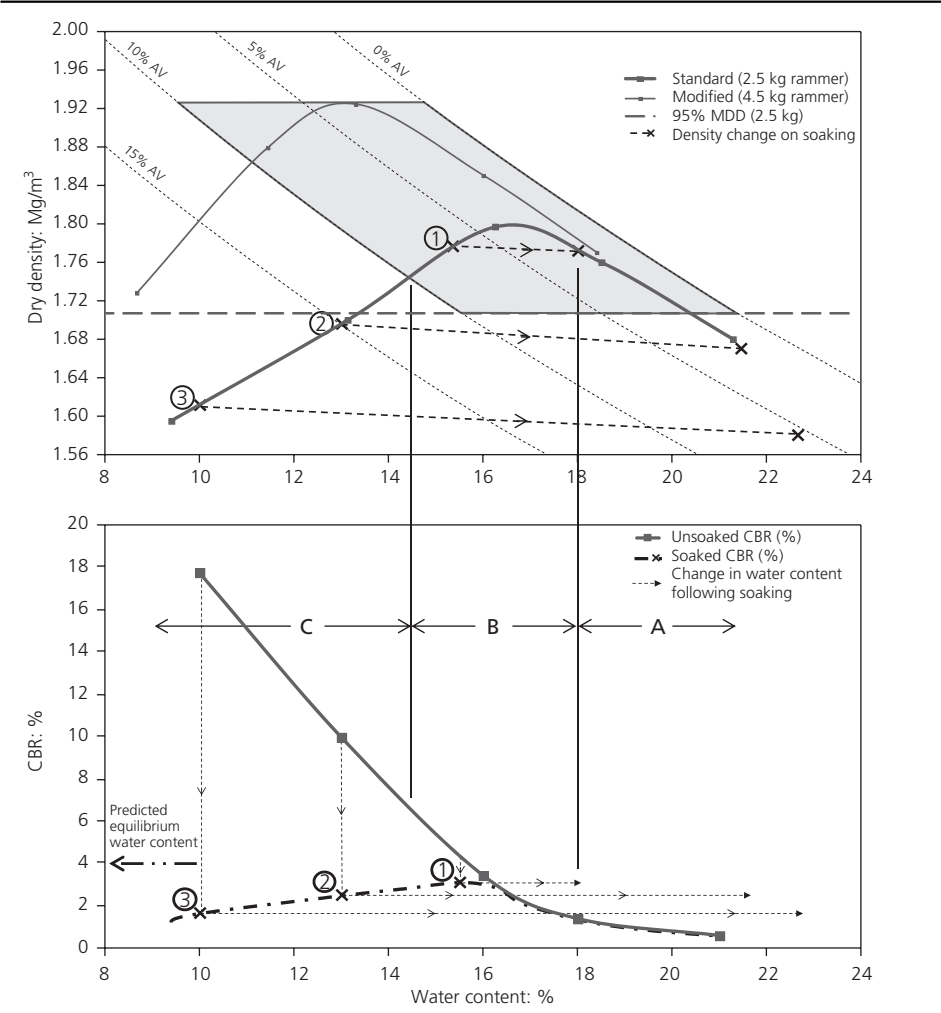
The conclusion reached for this fill material was that it would need to be placed at water contents in the range shown by zone B for a standard compactive effort, and that long-term CBR values of 2-4% could be expected.

post-construction, rather than the magnitude of the overburden/confining pressure (see Figure 3.8(a)). For settlement-sensitive structures and where there is a chance that cohesive fill will be inundated during operation, an initial water content slightly wet of optimum is suggested. Together with a relatively high compactive effort, the risk of collapse settlement should then be reduced or avoided. Situations in which this approach may be appropriate include the following:

- deep backfilled excavations in which a water table may rise up into the base of the fill (see following commentary)
- the basal layers of high embankments in low-lying terrains, where flooding is possible
- high embankments on side-long ground, where there is a possibility of inundation by ponding on the uphill side
- where failure of drainage installations could flood the fill.

The case examples where greatest magnitude of collapse settlements have been recorded (in excess of 1 m) are for deep backfilled quarries (Charles and Skinner, 2001; Charles, 2005). In these cases the compactive effort has often been very low, the thickness of fill is considerable and the resulting overburden (confining) pressure to drive the collapse mechanism is substantial. The collapse normally occurs some time after the infilling because the natural groundwater table will have been artificially lowered to enable the quarrying and backfilling, and the true groundwater re-establishes some time after

Figure 8.6 Example of relationship test results illustrating the tendency for dramatic loss of strength of fine soils compacted dry of OWC (see Figure 8.5 for details)



Note: The shaded area identifies the zone of water content/dry density where fill material tends to remain stable post-compaction.

Compaction condition	Fill behaviour at compacted water content and following saturation for samples compacted by 2.5 kg rammer method
Zone A	Fill compacted wet of OWC: low air voids but low CBR. For this range of water content the fill will be too soft to compact ($c_u < 40$ kPa), and the required density of 95% MDD will not be consistently achieved on site.
Zone B	Fill compacted at or close to OWC: acceptable air voids, moderate CBR and a high density, provides a stable condition as shown by test 1, which showed minimal change in water content/strength upon soaking.
Zone C	Fill compacted dry of OWC: very high initial CBR, but high air voids (and low density) result in a significant increase in water content on soaking with corresponding dramatic loss of CBR as shown by tests 2 and 3.

completion of the backfilling. There have been cases when poorly compacted fill has remained unchanged for many years and then collapse settlement occurred following a significant inundation of the fill; for example, as a result of flooding.

A review of the Spanish approach to working with dry clay fills is of interest because they are very experienced in working under these conditions. The general approach in Spain is to control the works using a modified Proctor test, add a limited amount of water during placement and utilise a high compactive effort, to achieve a defined target zone for compaction that is similar to zone PRSU on Figure 8.3. Although it should be recorded that the Spanish practice is to use saturation curves instead of air voids lines to manage earthworks (the authors' opinion is that air voids are more effective). Tests are undertaken to check the collapse potential of the soil. Less consideration is given to heave, which is probably due to the dry climate in Spain.

One naturally occurring soil type that is particularly susceptible to collapse settlement is 'loess', which is primarily wind-blown deposits of fine soils that have a meta-stable soil structure that will collapse on saturation. Some Colluvium ('hillwash') deposits can show similar behaviours. These collapsible soils are found in various different forms and in many different regions around the world, ranging from the relatively thin 'brick earth' deposits of the south east of England, which have a slight potential for collapse (although generally the collapse has already occurred since deposition), to extremely unstable silt and clay loess deposits present at various locations (particularly eastern Europe, Asia and North America). Significant collapse settlements can easily occur within loess soils or loess fills due to changes in surface water runoff conditions, or rising groundwater conditions as a result of infrastructure and building construction. Excavation and recompaction can be used as a solution to this natural collapse settlement; however, in planning such works it is important to realise that some loess soils have a tendency to retain this collapse potential even after recompaction. Roohnavaz *et al.* (2011) explored the use of unsaturated loess fill for earthworks, and identified that standard Proctor compaction may not be sufficient to break down the bonding between loess particles, and that benefits could be gained by repeated reworking and recompaction, use of greater compactive effort, or compacting fill wet of optimum. Special attention should be given to managing the collapse potential when working with loess soils, and the selection of earthworks specifications and plant should be based on compaction trials (see Jefferson and Rogers (2012) and Roohnavaz *et al.* (2011) for further details).

8.3.4.4 Heave

Heave may occur with cohesive fill that has been heavily compacted dry of optimum and which remains at a low confining pressure after placement (Figure 3.8(b)). If heave is to be reduced or avoided, an initial water content slightly wet of optimum is suggested but with only moderate compactive effort. Situations in which this approach could be adopted include those in which relatively low overburden (confining) pressures exist; they include:

- fill supporting lightly loaded minor structures, such as garages
- fill placed beneath floating floor slabs
- fill placed around the outside of structures to form pathways

- top layers of fill behind pre-cast concrete elements, gabions or cribbed walling
- where failure of drainage installations could flood the top layers of the fill.

Over-consolidated clays and weak mudstones of high plasticity, won from deep cuttings or excavations, are likely to be at a water content significantly below the OWC and plastic limit. If these fills are placed with a very high compactive effort to achieve a low overall air voids ratio within the fill then there will be a high risk of heave in humid climates. In these conditions heave will occur if the final overburden/confining pressure is insufficient to resist long-term suction forces that will establish due to the clay soil's natural tendency to achieve a water content nearer to the plastic limit (see cases 1 and 2 in Figure 8.4). The advisable approach to manage these fill materials is to allow a mellowing period between excavation and placement of the fill. The length of the mellowing period can be reduced by dissecting the clay into small lumps and making moisture available to enable the expansion/softening process (see Section 10.5.3).

An alternative approach to allowing the swelling process to take place is to treat the soil with lime. This approach is commonly used in Spain where swelling clays often must be used in earthworks. However, before this option can be considered the clay or weak mudstone must first be checked to ensure that the chemistry will not result in an adverse chemical reaction (i.e. due to an unacceptable sulfate or sulfide mineralogy), because this would create a far greater problem than the natural swelling potential of the soil. If the chemistry is appropriate then lime treatment can be used to reduce the soil's plasticity, reducing the potential for swelling (see Sections 3.5.1 and 10.5.2). There is an added benefit of this approach because the equipment used for the lime treatment process provides the opportunity to break up the soil into very small lumps and to add a controlled amount of water evenly through the soil, and this can be used to treat the soil to reduce further the tendency for post-compaction changes. Once treated the lime reaction can modify the state of the soil to create a more solid form with the inter-particle bonds created resisting any remaining swelling pressures.

8.3.4.5 Consolidation of very soft fine fills

Designers should also consider whether very soft clay or silt fills will lose water content subsequent to placement. Under significant overburden pressure the loss in water content will tend to result in a reduction in volume by consolidation (without an increase in air voids, see case 6 in Figure 8.4). This will necessitate a hold period prior to the earthwork being brought into service. In Sweden this consolidation process is used as a formal method of soil improvement by the inclusion of granular layers to speed the consolidation and limit the hold period to one or two years. However, the changes to very wet fine fill material under low overburden pressure could simply be desiccation, which is a reversible change of state (see case 5 in Figure 8.4).

8.3.4.6 Laboratory testing to assess collapse and heave potential

When fine soils are to be placed dry of optimum and a risk of volume change post-compaction has been identified, then collapse settlement and swell tests using the oedometer (BS 1377:1990: Part 5: Test 4 (BSI, 1990)) should be conducted on representative samples. This method can only be used for fine grained soils due to the small size of

oedometer test samples. For soils considered at risk of collapse settlement the double oedometer test is commonly used. Two near-identical specimens of soil are incrementally stressed within oedometer cells: one specimen in its natural state, the other having been saturated under a seating stress of typically 5 kPa. The results are compared to provide an indication of the collapse potential (Jefferson and Rogers, 2012).

An alternative approach to give an indication of volume changes due to saturation of dry fill, which can be employed as part of the fill material relationship testing for fine soils, sands and composite soils, is to compact the samples within CBR moulds and then record both the initial and saturated CBR values. The swelling volume change during saturation will be recorded (BS 1377:1990: Part 4: Test 7, an example of this test approach is shown in Figures 8.5 and 8.6). This test method is undertaken at a low confining pressure (typically being equivalent to 70 mm of overlying fill), which is unlikely to generate collapse settlement. For composite soils the designer may only be able to test samples of the material finer than 20 mm particle size, and use the results coupled with any available published data for similar soil types, to make an engineering judgement of risk of collapse settlement.

8.3.5 Fill modification and stabilisation processes

Whether or not treatment is adopted depends on the initial water content with respect to optimum, because it determines the strength and compressibility characteristics that are important to the SLS. For particularly wet cohesive fills, the addition of lime may be necessary to permit trafficking during construction and the avoidance of the pitfalls referred to in Section 3.4.

When hydraulic binders are added to a fill to achieve a permanent increase in the strength, compared to what can be achieved with the same fill without the use of binders, then this is termed ‘stabilisation’. When lime is used to adjust the soil’s water content to within an acceptable range for use as general fill material then the processed is referred to as ‘modification’ because there has been no permanent increase in strength above the natural state of that soil. Both processes rely on achieving a good standard of compaction and low air voids to avoid any subsequent loss of strength.

A commentary on the lime modification/stabilisation process is provided in Section 10.5.2.

8.4. Design and control testing

Having identified the design requirements that need to be satisfied by the earthworks to be constructed on a project (as described in Section 8.3), the designer must next set the acceptable range of fill material properties in order to achieve those requirements. This process can be considered as the earthworks fill design (see Figure 8.1) and is described within the following section. This aspect normally includes defining the project’s earthworks specification.

8.4.1 General

For the purpose of design and control testing of fill materials it is appropriate to consider two distinctly different groups of soil and rock properties.

- ‘Intrinsic soil properties’ such as: particle size distribution, Atterberg limits, organic content, particle density and grain strength; these will not change without physical intervention to modify the basic nature of the fill material.
- ‘State properties’ such as: water content, strength, density and swelling potential; these represent ‘states’ that the fill can readily achieve with changes in water content or compactive effort during normal earthworks processes.

For a material of a given set of ‘intrinsic properties’ there are five common laboratory test methods adopted in the design of engineered fills to explore the different conditions (‘states’) that the fill is likely to reach at different water contents:

- 1 the compaction test (relationship between water content and dry density)
- 2 the undrained shear strength or laboratory vane test
- 3 the MCV
- 4 the CBR
- 5 the permeability test.

Each method provides engineering properties that permit predictions of the likely engineering performance of the fill in the permanent works and may therefore be used for design. The tests can also be employed to set specification acceptability limits on the property, or range of properties, for the given fill and the engineering problem in hand.

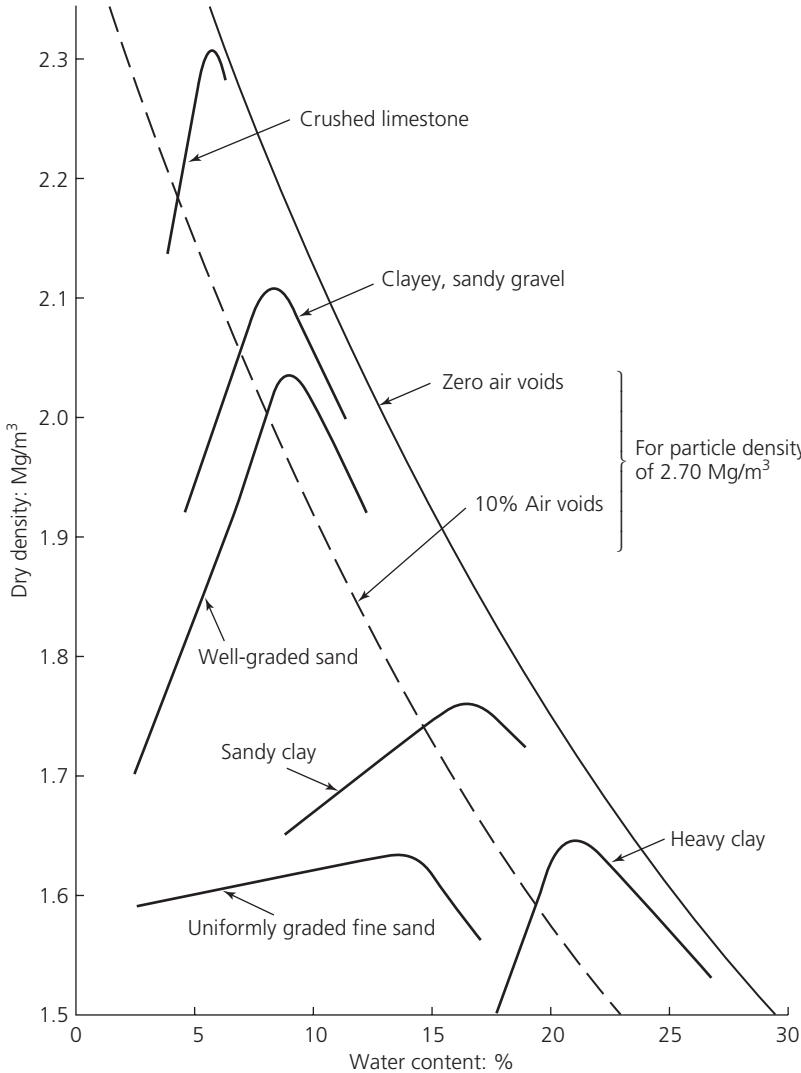
The compaction test (test 1 above) is the fundamental test on which our understanding of earthworks has been established (see Chapter 2). The earthworks Engineer needs to be aware of the differences in response to compaction of common types of fill material, as the soil properties make a big difference to: the shape of the compaction curve, MDD, OWC and air voids achieved by a given compactive effort. A full record of the extensive work undertaken on compaction testing was captured by Parsons (1992). Figure 8.7 is reproduced from that work (note the terms ‘heavy clay’ and ‘sandy clay’, respectively, indicate high and low plasticity).

Test methods 2, 3 and 4 are all providing an indication of the strength/stiffness of the fill. Unfortunately there is no true stiffness test that can be readily and affordably undertaken in the laboratory as part of the compaction-based relationship test process to assess a fill’s properties at different water contents. Consequently when the design requires consideration of stiffness the earthworks designer has to rely on correlations between properties, which is generally a workable (even though not ideal) compromise.

Control testing is of equal importance. It requires that the fill, when placed, to be tested in situ to check that the properties selected during design are being obtained on site. For all but the compaction test, control testing is performed using the same type of test as adopted during design. For the compaction test, control uses in situ dry density and water content determinations.

For some projects, when heavy loads are expected or when substantial vibrations may occur in service, such as earthworks for high-speed rail, design and construction control

Figure 8.7 Relations between dry density and water content for various soils and aggregates when compacted in the BS 2.5 kg rammer test (reproduced from Parsons, 1992)



may be appropriate using the deformation modulus; this could apply both to the subgrade or to the overlying formation. In such cases, construction control could be performed using in situ plate loading tests. An alternative is to use zone tests over a large loaded area (such as skip load tests) that will be more time consuming but have the benefit of testing a greater depth of the fill. It is important to note that these in situ tests determine a subgrade modulus, not a true stiffness value, because of the spread of load with depth beyond the loaded area due to the pressure bulb effect.

Guidance provided for high-speed railways across Europe is based on plate load testing to determine the modulus on the second load cycle (to obtain parameter E_{V2}) (UIC, 2008). In the UK the use of plate load tests to control earthworks is relatively uncommon, part of the problem being that there are many variables that can influence the results. At first reading the E_{V2} approach appears to provide a means of achieving a unified method of measuring a stiffness modulus for use in earthworks design. However, exploration of this topic has shown that there are wide differences between countries in how E_{V2} is determined, which makes comparison of experience between countries extremely difficult. Furthermore, many practitioners in other countries who have more experience in the use of plate load testing for earthworks have concluded that it is not a reliable measure for fine fill materials, so they control the earthworks using conventional testing and only use the plate load test as a final check (effectively to satisfy the specification requirement for that test).

A description of design and control tests is given in Sections 8.4.3 to 8.4.8. Trenter summarised the advantages and disadvantages of these tests in Table 8.1 (where the BS 1377:1990 part and test numbers are also given). It is relevant to note that this has a bias towards landfill engineering, and engineers working in other fields of earthworks will have some different views on the relative advantages of these tests, but Table 8.1 remains a useful guide. A practical point often overlooked is the speed at which tests can be done and the results reported. This is of major importance with control testing because methods that necessitate more than 24 hours to obtain results may not assist the control function as further layers of fill may have been placed above that to which the control testing applies. This should be borne in mind in selecting the test type and also the size of test laboratory, because they will both govern the throughput of results. A well run site laboratory will be an essential part of a major earthworks project to deal with the large amount of testing required.

8.4.2 Earthworks specifications: setting design and control limits

Design and construction control limits have to be set for the soil property and test methods selected for the control of the earthworks operation. The most common approach for earthworks is to define the limits based on compaction test data (see Section 8.4.3). Acceptable limits are established relative to the water content of the fill, although another property (e.g. undrained shear strength or MCV) may then be used by means of relationship testing as a proxy for water content.

The earthworks control limits are normally defined within a project specification, usually within a tabular format of the type used within the SHW, with one or two parameters set for each fill material property that will determine a fill material's acceptability. BS 6031:2009 Clause 7.6.4 (BSI, 2009) defines the acceptability limits relative to water content, with the defined terms being:

- lower acceptability limit (LAL) – sets the minimum water content
- upper acceptability limit (UAL) – sets the maximum water content.

In some cases the designer may opt to use different properties to define the range of acceptable water content, for example for wet cohesive fill: water content relative to

Table 8.1 Advantages and disadvantages of various engineering properties for design, acceptability and control testing of fills (adapted from Trenter, 1999), reproduced by kind permission from CIRIA

Property (see note 1)	Design and acceptability		Control	
	For	Against	For	Against
Particle size (2 : 9)	Governing factor in performance of fills		As design and acceptability	Test lengthy to perform and equipment cumbersome
Water content with respect to plastic limit (2 : 3.2, 2 : 5.3)	Properties easily measured	Precision ^a of plastic limit test low	As design and acceptability	As design and acceptability
Optimum water content and maximum dry density (4 : 3.3 to 4 : 3.7)	Properties fundamental to engineering performance Particle size up to 20 mm tested ^b	Time taken for compaction test performance (usually much in excess of 24 hours)	Equipment for control tests accommodated in mobile site laboratory	Accuracy of density values from sand replacement test questioned (Parsons, 1992)
Moisture condition value (4 : 5.4 to 4 : 5.6)	Particle size up to 20 mm tested	Not readily related to fundamental engineering properties	Equipment accommodated in mobile site laboratory; speed of use	As design and acceptability
CBR value (4 : 7)	Property frequently used for pavement design Particle size up to 20 mm tested ^b	Not readily related to fundamental engineering properties apart from empirical correlations	As design and acceptability	As design and acceptability
Undrained triaxial shear strength (7 : 8)	Fundamental to trafficability and to bearing capacity of fills Water content and bulk density obtained from test specimen	Not readily related to OWC and MDD	As design and acceptability	38 mm test specimens too small for many fills containing gravel; 100 mm test specimens require expensive equipment for testing and sample extraction

Unconfined compressive strength (7 : 7)	As for undrained triaxial shear strength	As for undrained triaxial shear strength	Much lighter and more portable than the undrained triaxial device	38 mm test specimen only: too small for many fills containing gravel
Hand vane (7 : 3) as an alternative undrained shear strength test (see text)	As for undrained triaxial shear strength	As for undrained triaxial shear strength	Light weight, small and easily transportable	Only for clay fill containing little or no coarse sand or gravel
Permeability tests (5 : 5, 6 : 4, 6 : 6)	Fundamental to the design of landfill liners, dam cores and other impermeable structures; wide variety of test types available including some in situ (see text)	Wide range of applications (see Table 8.6)	As for undrained triaxial shear strength	Tests too complex and time consuming for routine site use Best to correlate with results of other test types at the design stage (see text)

^a Precision is defined in ISO 5725 Part 1 (BSI, 1994)

^b Up to 37.5 mm particle size can be catered for in some circumstances (see Section 2.6.1)

1. Figures in brackets refer to the part and test numbers of BS 1377:1990

2. For water content and plastic limit determinations, measurements are made on the fraction of the sample finer than the 425 μ m sieve and the results may not be representative of the whole sample

optimum to define the dry side (LAL), and minimum undrained shear strength to define the wet side (UAL).

The designer may decide to allow some tolerance to variance from the defined acceptability limit. This can be achieved by permitting one test result in every x number of results not to reach the defined limit, with a permitted percentage difference being set. Another approach that can be used for clay barriers has been to define a mean shear strength value together with a range about the mean (see Section 8.4.4).

In setting the acceptability limits, the Engineer must consider:

- the design requirements, as discussed in Section 8.3, to ensure the earthworks will be stable during and after construction
- the potential for changes in material properties post-compaction, to avoid excessive settlement, heave or reduction in strength/stiffness
- the relevant geotechnical properties of the fill at the cutting or borrow sites. There is no point in selecting design water contents if the potential fill cannot realistically be expected to meet them, even with extensive (and expensive) conditioning
- the likely weather conditions when the fill is to be placed, and construction aspects such as under- and over-compaction, matting and the like (see Section 3.4)
- the most appropriate test method to achieve acceptable quality control without unreasonable delay of the works.

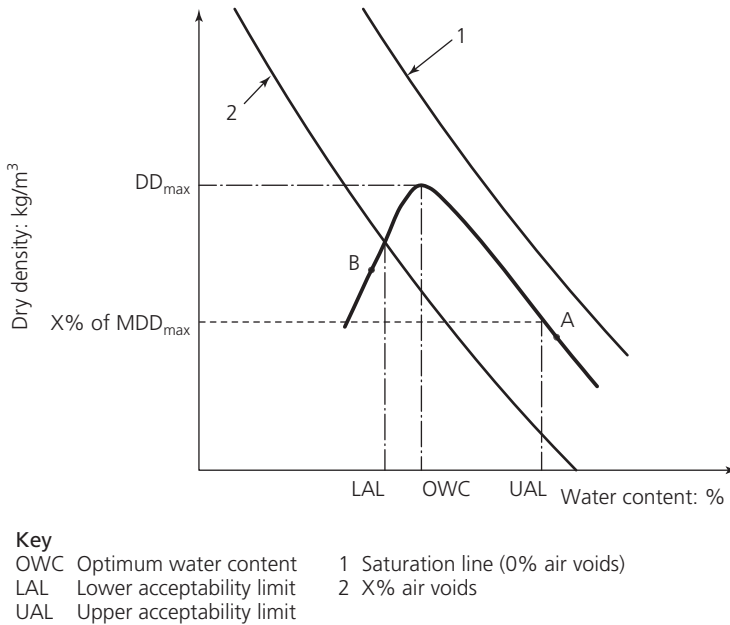
If the approach illustrated in Figure 8.3 is employed, then the design requirement will have been set by reference to the properties defined by the area of the diagram selected. It is then necessary to check that fill having these properties is suitable for trafficking; which is best done by relationship testing (see Section 8.4.8). An example of determining the LALs and UALs is illustrated in Figure 8.8, which is based on the approach set out in Clause 7.6.4 of BS 6031:2009 (BSI, 2009). In this example fill at point 'A' is identified as being below the required undrained shear strength (or target dry density), and fill at point 'B' exceeds the acceptable air voids. Based on the relationship testing data the specification value of LAL is set based on water content, and UAL either on water content or undrained shear strength.

Where there are specific requirements to limit the internal settlement within large bodies of fill that will carry structures the selection of design parameters will need to go beyond the normal SHW approach. For this case BRE Digest 427 Part 3 (BRE, 1998) provides appropriate guidance. Specific guidance on setting end product control limits for engineered fill supporting low rise structures is given in the example specification included in Appendix 1.

8.4.3 Compaction test

The traditional approach to the design of engineered fill is the compaction test. BS 1377:1990 includes three different levels of compactive effort: the BS 2.5 kg rammer method is the lightest (and represents the original Proctor test), the 4.5 kg rammer method (often referred to as the 'modified Proctor' test), or the vibrating hammer method for granular fills is the greatest compactive effort. Compaction tests may be employed for

Figure 8.8 Determination of acceptability limits for fine soils using relationship testing data, see text for explanation (based on BS 6031:2009 (BSI, 2009))



all types of earthworks except very coarse fills. For the given compactive effort and fill specific gravity, the test provides the following information:

- the air voids content of the compacted soil at any dry density or water content
- the variation of dry density with water content for the soil being tested.

The original test methods proposed by Proctor used a 1 litre mould, which is suitable for particles up to medium gravel size. BS 1377 also includes procedures for the larger CBR mould, which suits coarser fill materials; the sample must have at least 70% passing the 20 mm sieve, the fraction retained on the 20 mm sieve can include a limited amount of particles coarser than 37.5 mm but these must be removed and replaced by smaller particles of a similar nature. If these grading requirements are not satisfied then the sample is classified as 'grading zone X' and none of the compaction test methods are deemed appropriate.

It is important to realise that the limitation on particle size for the standard compaction tests are as a result of the size of the mould and the requirement to achieve repeatable results. The aggregates industry does include larger tests suitable for coarser fills, but these are not used in the earthworks industry, presumably due to the high cost of the tests and because clean granular soils are generally good fills and can be assessed based on experience and site trials.

The problem with grading zone X is that if the criteria are strictly applied then many composite soils that are likely to perform well as fill are classified as untestable by virtue of a

relatively low granular content (e.g. well-graded Glacial Till). This is actually detrimental to the management of the earthworks project. Section 2.6 provides methods for adjustment of the results to allow for the influence of coarse fraction. An experience-based approach is recommended for these coarse soils to determine the most appropriate method for testing so that control limits can be set to enable utilisation of the fill. For gap graded or well-graded fills (granular or cohesive) the earthworks Engineer may judge that there is a matrix of testable material that will strongly influence the performance of the fill. In many cases it may prove adequate to remove coarse gravel/cobbles, and replace these with a similar proportion of finer gravel of the same material, to facilitate laboratory testing and make a judgement of the acceptability criteria based on the finer fraction of material; while not ideal this approach is considerably better than progressing with no data (BSI, 2009).

The designer should consider the most appropriate compaction test for the particular fill material and anticipated construction plant. A summary of the different compaction test methods available within BS 1377:1990: Part 4 (BSI, 1990) is set out as Table 8.2. This range of compaction tests for use in earthworks is as extensive as any other national practice. When comparing compaction test results obtained by different national standards it is important to realise that all of the factors listed in Table 8.2 will have an influence on the results obtained.

8.4.3.1 Air voids and specific gravity

The air voids lines are critical to understanding a fill material and should always be plotted onto compaction test results. A specific gravity value (otherwise referred to as particle density) needs to be assumed for the fill material in order to calculate air voids.

As noted elsewhere, a low air voids content (high degree of saturation) usually secures a stable fill. However, air voids content cannot be employed on its own for design and control purposes; a water content limit must also be specified, otherwise unsatisfactorily wet (or dry) fills could result.

When the design air voids content has been selected, it can be controlled during construction together with the water content range using dry densities and water contents measured in situ. A specific gravity of 2.65 or 2.7 is often assumed as a typical value for British soils and rocks, but this can lead to problems, and values determined by laboratory testing should be obtained (see below). When a specific gravity value of 2.7 is assumed, Equation 2.5 given in Section 2.2 becomes

$$A_v = \left(1 - \frac{\rho_d}{2.7} [1 + 2.7w]\right) \quad (8.1)$$

where ρ_d is the measured dry density (Mg/m^3) and w is the water content. Air voids measured in this way can be used with the suggested levels of compactive effort illustrated in Figure 8.3.

In practice, the air voids content can be a difficult property to measure because of the variation in specific gravity exhibited by some materials (e.g. colliery discards, chalk and

Table 8.2 Summary of BS 1377:1990 compaction test methods

Test Type	Soil fraction	Rammer		Mould		Layers		Maximum particle size: mm				
		BS 1377-4 method	Mass: g	Diameter: mm	Drop height: mm	Energy imparted (see note 1): kJ/m ³	Diameter: mm		Volume: l	Number	Blows per layer	Approx. thickness: mm
BS light compaction	<Medium gravel size particles	3.3	2500	50	300	595	105	1.00	3	27	40	20
BS heavy compaction		3.5	4500	50	450	2675	105	1.00	5	27	25	20
CBR (light compaction)	Some coarse gravel size particles	3.4	2500	50	300	595	152	2.305	3	62	42.5	37.5
CBR (heavy compaction)		3.6	4500	50	450	2675	152	2.305	5	62	25	37.5
Vibrating hammer		3.7	(see note 2)	145	–	11740	152	2.305	3	1 min	42.5	37.5

1. The compactive effort of the different test methods is best compared using the energy imparted during the test

2. For the vibrating hammer test a steady downward force is applied to the hammer so that the total force exerted on the sample is between 300 to 400 N. This cannot be directly compared to the mass of a falling rammer

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Table 8.3 Specific gravity values for a range of common minerals and rocks

Minerals	Specific gravity	Rocks	Specific gravity
Augite	3.2–3.6	Andesite	2.5–2.8
Biotite	2.8–3.3	Basalt	2.8–3.0
Calcite	2.7	Chalk	2.5–2.7
Chlorite	2.6–3.3	Coal	1.1–1.5
Dolomite	2.8–2.9	Diabase	2.6–3.0
Fluorite	3.1–3.2	Diorite	2.8–3.0
Garnet	3.6–4.3	Gabbro	2.7–3.3
Gypsum	2.3	Gneiss	2.6–2.9
Halite	2.1–2.2	Granite	2.6–2.7
Hematite	5.2–5.3	Limestone	2.3–2.8
Hornblende	3.0–3.5	Marble	2.5–2.7
Illite	2.6–2.9	Mica Schist	2.5–2.9
Kaolinite	2.6–2.7	Quartzite	2.6–2.8
Montmorillonite	2.0–2.8	Rhyolite	2.4–2.6
Muscovite	2.8–3.0	Sandstone	2.2–2.8
Olivine	3.2–4.4	Shale	2.3–2.8
Orthoclase (K-Feldspars)	2.5–2.6	Slate	2.6–2.8
Plagioclase (Na–Ca-Feldspars)	2.6–2.8		
Pyrite	4.9–5.2		
Quartz	2.65		

This table has been compiled based on data contained in the following references: Bell (1983); Bishop *et al.* (2001); Lagomarsino (2008); Lambe and Whitman (1979); Waltham (2002)

recycled materials). Great care should be taken to obtain representative results. Tests on several specimens from each sample may be needed.

For most fill materials the leg of the compaction curve significantly wet of OWC will plot almost parallel to, and slightly to the left of, the 0% air voids line (see Figure 8.3 for an idealised case, and Figures 8.6 and 8.7 for examples). If results do not plot this way then more careful consideration should be given to whether the specific gravity is appropriate for the nature of the fill. A curve fitting approach can be employed as a simple check; replotting the compaction test results with different assumed specific gravity values to see which value plots closest to the expected norm, and then compare that value to published data to see how likely that result would be given the mineralogy of the fill. For reference, typical specific gravity values for a selection of common minerals and rock types are included within Table 8.3.

The curve fitting approach described above should not be used simply to set a specific gravity without undertaking any laboratory testing because some fills do not follow normal behaviour patterns. Furthermore, with single size fills (especially sands) the compaction curves are very flat, making it difficult to assess the relative positions of the

compaction curve and the saturation line, and obtaining reliable specific gravity values is particularly important.

8.4.3.2 Dry density and water content

The most common approach for specifying the requirements for coarse fills (<15% fines) is to define a required range of dry density to be achieved, the approach taken depends on the form of specification employed.

- An end product specification may define the minimum dry density required as a percentage of MDD achieved by a particular test method (e.g. in situ dry density to be at least 95% of MDD by 2.5 kg rammer method). Although commonly quoted, this simple approach includes no control on air voids. A better approach can be to define a zone of acceptable dry density/water content to satisfy the requirements described in Section 8.3.3.
- A method compaction specification will define the acceptable intrinsic properties (e.g. grading) and state properties of the fill material (e.g. water content to be in the range OWC –2% to OWC +3%), and define the method of compaction to be employed. The required number of passes and layer thickness to achieve the desired compaction with the type of compaction plant will have been determined by extensive trials. Sufficient compaction test results are required to assess the particular fill material, and the LAL is determined to minimise air voids (see Section 8.3.3).

A disadvantage of the compaction test when used as part of the compliance testing to control the earthworks operations is the time taken for its performance. Similarly the water content test to meet BS 1377 (BSI, 1990) will result in a minimum 24 hour delay from the time of sampling. Depending on the resources of the testing laboratory, throughput can be slow. This may not be a problem during the design stage, but could be serious during construction. An example could be when the soil types at the borrow were variable, requiring back-up testing during fill placement, or when conditions at the borrow proved to be significantly different from those expected on the basis of site investigation information. This should be taken into consideration when deciding on the design and control approach.

Control requires the use of in situ dry density methods, such as the sand-replacement test for fine to medium grained soil (BS 1377:1990: Part 9: Test 2.2); water replacement test suitable for coarse grained soil (BS 1377:1990: Part 9: Test 2.3); or the core-cutter method for clay soil (BS 1377:1990: Part 9: Test 2.4). An account of the various tests is given in *Soil Mechanics for Road Engineers* (Road Research Laboratory, 1952), which although very old continues to be a good reference due to the amount of data presented. It should be noted that tests conducted on the current fill layer may give unrepresentatively low results because of disturbance by the compaction equipment, or the tendency of sand fills only to hold the compactive effort when the next layer is compacted above to provide constraint. For these reasons, some engineers favour testing the previous layer that will have been recompacted during placing and compacting the current layer.

For rapid in situ density control, use may also be made of a properly calibrated nuclear density gauge (BS 1377:1990: Part 9: Test 2.5 (BSI, 1990); Parsons, 1992, Highways Agency

Advice Note HA 70/94 (HA, 1994a)). If a nuclear density device is to be employed with colliery discard, the variation in coal content may affect its calibration (Hird *et al.*, 1998). Similar variations may be expected with chalk. To support the laboratory calibration process further calibration checks during compaction trials are advisable (see Section 8.5). Experience shows that for many fills the accuracy and reliability of a calibrated nuclear density gauge is very good, and the advantages of being able to undertake a large number of test results and obtain instant results makes it the best method available.

An alternative to the nuclear density test has been developed to avoid the problems associated with management of the operatives' exposure to radiation. This test is the electromagnetic soil density gauge but at present only an American Society for Testing and Materials (ASTM) standard exists for this test method (ASTM D7830/D7830M-13 (ASTM, 2013)).

In situ density tests also permit water content determinations on the fill material; consideration may be given to using a microwave oven for quick water content determinations, although calibration with properly conducted oven water content tests is essential. When a nuclear density gauge is used to measure water content it is advisable to recover samples for occasional checks by laboratory testing.

8.4.3.3 Particular requirements for uniformly graded granular fills

Section 8.3.3 described the use of air voids to help define the compaction requirements for fill materials, but commented on the limitation of this approach for some granular fills. Particular problems are experienced with uniformly graded fills as in some cases it will not be possible to achieve a low air voids content regardless of the compactive effort applied (see Section 4.3.1). The SHW sets the upper limit of uniformity coefficient (C_u) at 10 for the classification of uniformly graded granular fills, which is a good general indicator for earthworks (see Section 4.2.3). It is relevant to note that within Eurocode 7 a slightly different system is used, with soils that have a uniformity coefficient of less than 6 being described as 'evenly graded' (see Clause 3.1 of BS 6031:2009 (BSI, 2009)).

This situation is reasonably common for sand fill because natural sands can often have a low fines content and a very low uniformity coefficient (sometimes significantly below C_u of 10). An example to illustrate this behaviour is given in Figures 8.9 and 8.10, which describe the case study and present the results respectively.

The behaviour shown by the samples illustrated in Figure 8.10 and described in Figure 8.9 is reasonably typical of many sands. Clearly, it is the fines content of this sand that is determining the behaviour, and a simplistic classification based on the normal definition of uniformly graded fill (C_u of 10) would be misleading.

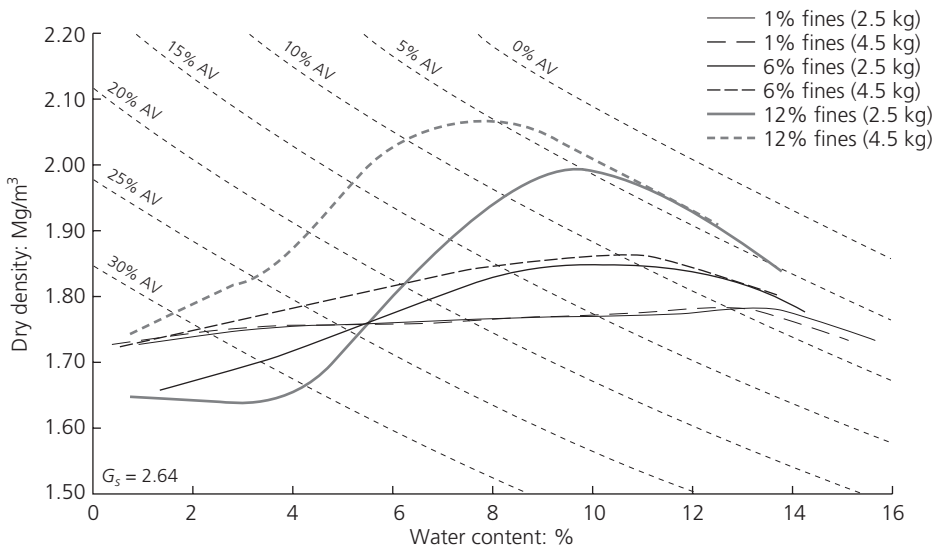
One other feature of sand fills can be experienced when working in arid desert conditions with fills at extremely low water content, a phenomenon of a minimum dry density often occurs just above zero water content. This behaviour is shown by the 12% fines 2.5 kg compaction curve in Figure 8.10. The zone between zero and the water content at the minimum dry density is referred to as the 'arid zone' within the French earthworks standard (LCPC, 2003: Chapter 7).

Figure 8.9 Case study: compaction characteristics of sand fills

Three samples of the same natural silty fine to medium grained sand were tested at both 2.5 kg and 4.5 kg rammer compactive effort. The difference between the samples was a progressively increasing fines content. The response to compaction is shown in Figure 8.10 and can be summarised as follows.

- Sample with 1% fines (C_u of 2.0) responded as a uniformly graded granular fill with almost flat compaction curves showing no increase in density with increased compactive effort, and only achieving just under 10% air voids at very high water content.
- Sample with 6% fines (C_u of 2.8) showed intermediate behaviour.
- Sample with 12% fines (C_u of 4.2) responded as a well-graded granular fill with well developed compaction curves revealing clear OWC/MDD values, with 10% to 5% air voids being achieved either side of OWC, and the classic positive response to the increased compactive effort of the 4.5 kg rammer test.

Samples compacted at 2.5 kg compactive effort were tested for both unsoaked and soaked CBR. The results showed that 10% CBR or higher could be relied on at OWC, but reduced rapidly above that water content. Dry of OWC -3% for the well-graded sand and OWC -6% for the uniform sand it was shown that the initially high CBR values reduced significantly following soaking. The design concluded that a CBR of 10% could be relied on for all of these fills if placed under the right conditions, with the uniform sand being the least sensitive to water content.

Figure 8.10 Compaction test results for silty fine to medium sand showing the change in behaviour with increasing silt content from uniformly graded to well-graded granular fill (see Figure 8.9 for details)

8.4.4 Undrained shear strength test

8.4.4.1 Stability of embankment fill

Factors affecting the undrained shear strength of clay fill were described in Section 3.3.2. Use of the total stress approach (with undrained shear strengths) in embankment stability analysis is commented on in Section 12.4.

8.4.4.2 Placement control

The importance of undrained shear strength as a means of understanding a fill material's changes in state are described in Section 8.3.4. Undrained shear strength is commonly employed to control fill placement. The undrained strength can be used in the earthworks specifications as an alternative to water content as the acceptability limit for fine soils as follows.

- A lower limit of c_u in place of an upper limit of acceptable water content: this is common practice for general earthworks; HA 44/91 (HA, 1995) suggests a typical UAL range of 30–50 kN/m² with the actual value selected depending on factors such as trafficability or stability.
- An upper limit of c_u in place of a lower limit of acceptable water content: this is less common for general fill (HA 44/91 (HA, 1995) suggests a typical LAL range of 150–200 kN/m²), but is reasonably common for low permeability clay barriers (see embankment dams example below).

When undrained shear strength is used as a proxy for defining the LAL of water content, the designer must assess relationship testing data for the fill to determine the appropriate c_u value to achieve a required dry density and air voids ratio for the chosen compactive effort. This approach is logical because clays above a certain undrained shear strength become too difficult to compact with normal earthworks plant.

The same approach can be taken to use undrained shear strength for defining the UAL. If the designer is not concerned about the dry density of the fill then the designer can use strength directly as a means of defining the condition of the fill at which normal earthworks plant will be able to operate. For this purpose the designer can draw on published data on minimum undrained shear strength for operation of different types of plant, some of which is summarised in Table 8.4. Work on the efficiency of plant operation undertaken in the 1970s assumed that the ruts created by construction plant must be limited in depth to the layer of fill that was being placed (rather than extend into the layer below), which was taken as being around 250 mm (Farrar and Darley, 1975; Dennehy, 1978). Such a depth of rutting would not be favoured now; however, the dump trucks that are used for modern earthworks operate at a lower tyre pressure than the plant of the 1970s and consequently Dennehy's guidance is still considered appropriate. The experience of the authors for modern plant used on projects since 2000 is included in Table 8.4 as general guidance.

The undrained shear strength has the advantage over other controls (such as water content or permeability) of being a quick and low cost test because shear vanes can be used on compaction test samples and in the field. However, it is very easy to overestimate c_u by turning the shear vane too quickly. Therefore, the shear vane can only be used as a secondary check to assess the works as they progress, with the main acceptability criteria being laboratory testing on undisturbed samples recovered from the earthworks as construction proceeds.

Undrained shear strength is used for the control of earthworks constructing the clay fill zones within earth dams. For this end use the primary focus is to ensure the clay is

Table 8.4 Undrained shear strength required for efficient plant operation

Plant	Minimum c_u recommended: kN/m^2	Reference	Comment
Caterpillar tractors and scrapers	35	Arrowsmith (1978)	This plant is no longer commonly used.
Large rubber tyred scrapers	50	Arrowsmith (1978) and Farrar and Darley (1975)	TRRL defined efficient working as enabling an average speed of 15 km/h.
Low tyre pressure plant	40–60	Dennehy (1978)	Dennehy related remoulded c_u to tyre pressure of earthworks plant to limit rut depth for efficient plant working.
High tyre pressure plant	60–80	Dennehy (1978)	
Modern dump trucks for general fill	40–50	Unpublished	These values are based on the authors' experience of plant available since 2000. Use of the lower value quoted would require the contractor's acceptance.
Modern dump trucks for landscape fill	30–40	Unpublished	

workable enough to ensure that the earthworks plant can fully remould the fill, breaking down all large lumps of soil and removing air voids. In this situation undrained shear strength can be used as both the LALs and UALs. In some cases an additional requirement has been set based on the average of the tested values. Control strengths selected for six embankment dams are given in Table 8.5, where other relevant information is also provided. The dams included in Table 8.5 were constructed using motor scrapers; consequently the minimum undrained shear strength requirements are higher than might be expected now if dump trucks were used to construct a dam.

For the Bewl Bridge dam, the running mean was to fall within the range from 70 to 100 kN/m^2 and for Ardingley, the running average of five consecutive tests was to lie within the range from 70 to 90 kN/m^2 . Triaxial testing at Kielder was supplemented by 250 mm dia. plate tests using plant as reaction; checks were also made using hand vane and pocket penetrometers, together with checks on rutting depths.

In all cases, use of the undrained triaxial test as a control was reported as being successful, with the stated strength ranges (and running averages where quoted) being met. It is relevant to note that, for the materials tested, the strength ranges quoted corresponded to between 95% and 100% of MDD as measured by the BS 2.5 kg rammer compaction test.

Table 8.5 Undrained triaxial strengths selected for fill control purposes

Embankment dam (fill type)	Undrained shear strength range: kN/m ²	Other requirements
Cow Green (1) (clay till)	48–110	66% of results to fall within stated strength range
Bewl Bridge (2) (Wadhurst clay)	55–110	80% of results to fall within stated strength range (dry density >95% of MDD by BS 2.5 kg rammer method)
Ardingley (2) (Wadhurst clay)	60–100	70% of results to fall within stated strength range (dry density >98% of MDD by BS 2.5 kg rammer method)
Cadney Carrs (2) (Oxford clay)	45–110	66% of results obtained fell within the stated strength range (dry density >97% of MDD by BS 2.5 kg rammer method)
Kielder (3) (clay till core)	60–140	Mean measured strength was 105 kN/m ² and measured dry density was 100% of MDD by BS 2.5 kg rammer method
Empingham (4) Upper Lias clay core	60–90	45% of results obtained fell within the stated range; mean strength obtained was 75 kN/m ²

From (1) Vaughan *et al.* (1975); (2) Kennard *et al.* (1978); (3) Millmore and McNicol (1983); (4) Vaughan (1978)

8.4.4.3 Clay fill foundations

A test measuring the fill's undrained shear strength appears an obvious choice when designing and controlling clay fill to support structures. A design shear strength is selected that is considered appropriate for the structure in question; for example, if a foundation is being designed to an allowable bearing pressure of 100 kN/m², the required undrained shear strength may be calculated for the foundation depth and geometry. Laboratory tests on remoulded samples of the selected fill are conducted to determine the water content and density to which the fill should be placed to obtain the necessary shear strength. However, this could only be used as a guide for construction because the final foundation design would normally be undertaken based on ground investigation post-construction to determine the actual properties of the bearing strata.

In accordance with convention, bearing capacity failure is assessed primarily on undrained loading conditions, and so unconsolidated undrained shear strength tests are commonly used. In British conditions, the fill may become saturated post-construction so consideration should be given to the undrained shear at the water content that could be anticipated in the long term.

8.4.4.4 Methods of testing and sampling

Use of the undrained triaxial test machine requires access either to a commercial testing laboratory or to a reasonably sophisticated laboratory on site, equipped with electric

power facilities. For clay fill containing little or no coarse sand or gravel, 38 mm dia. tube samples may be extracted using man-handled equipment. For clay fill with a significant granular content, including coarse gravel, a sampling rig capable of extracting 100 mm dia. tube samples will be necessary. All of these factors increase expense. The methods set out in EC7 Part 2 (BSI, 2007) for ground investigation require high quality undisturbed samples for triaxial testing, but the authors consider that this would be excessive when the testing is simply used as part of routine earthworks compliance testing.

Substantially quicker and cheaper than the undrained triaxial test is the laboratory vane. This test measures the undrained shear strength by means of a small (typically 12.7 by 12.7 mm) cruciform vane, which is inserted into the remoulded test specimen. The equipment comprises a stout frame that supports the rod carrying the cruciform vane assembly, together with a base plate onto which the specimen container is fixed. Although commonly referred to as the 'laboratory' vane test, it is compact and reasonably light, is quick to perform, and may be used in the back of an all-terrain vehicle because no electric power is needed in its operation.

The hand shear vane can be used as a guide to shear strength of the fill in situ. However, this test method should not be used for quality assurance purposes because unrepresentatively high results will be obtained if the shear vane is turned too quickly (see Section 10.7).

The use of shear vanes is limited to cohesive soils with limited gravel content, and some amount of correlation of results with laboratory undrained triaxial tests is advisable.

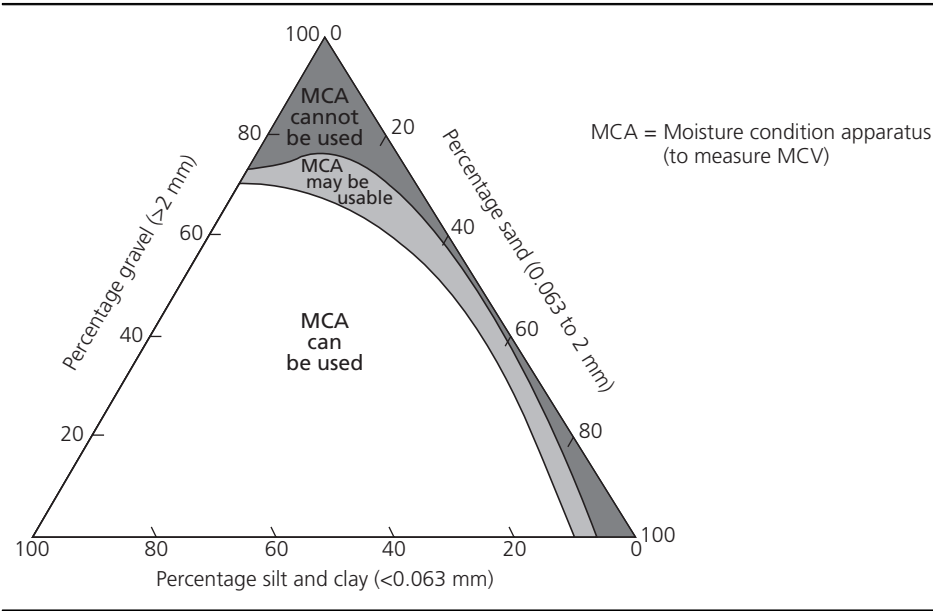
8.4.5 Moisture condition value (MCV) test

The MCV test equipment is reasonably light and may be operated from the back of an all-terrain vehicle. The MCV test can be performed on a wide range of materials illustrated in Figure 8.11. This, together with its portability and provision of immediate results, makes it an ideal test for construction control purposes.

There is no direct correlation between MCV value and the other commonly adopted engineering properties like water content, dry density and shear strength. Such correlations should be assessed for the particular soil type with the results of compaction, strength or CBR and MCV tests using relationship testing (see Section 8.4.8).

The MCV test tends to become very popular with earthworks Engineers once they have worked on a project using the test because it gives reliable results immediately for fine fills, and is easy to correlate to other soil properties. It also gives consistent results, which enables Engineers to draw on data from other projects with similar soil types. Experience shows that for many cohesive fill materials an MCV value of 8 generally corresponds to an undrained shear strength of around 50 kN/m², and consequently is often set within the Specification to define the wettest acceptable state for fine fills. Lower MCV values can prove acceptable for gravelly fine soils (e.g. Class 2C stony cohesive fill of the SHW classification); an MCV value as low as 7 or even 6.5 can prove appropriate for the Glacial Tills found across large parts of Ireland and Scotland, which are low to medium plasticity, very well-graded, composite soils. Using relationship testing to define the acceptability

Figure 8.11 Applicability of moisture condition value (MCV) test (based on Winter *et al.*, 1998)



limit on the wet side of optimum is particularly important for these Irish and Scottish Glacial Till soils because the soils, which tend to have 15–35% fines, are very moisture susceptible with the strength falling away dramatically above a certain water content that needs to be determined for the particular soil.

8.4.6 California bearing ratio (CBR) test

The CBR test has been widely used in the past and often forms the basis of pavement design. For this reason, it finds favour with some as a design and control method. Design testing is usually undertaken in the laboratory where the CBR can be measured for a variety of dry densities and also ‘soaked’ or ‘unsoaked’, corresponding to the groundwater or drainage conditions foreseen. Surcharges can be added to simulate the effect of construction thicknesses. Once a design CBR has been established, it can be controlled in the field either by sampling the placed fill and testing at the site laboratory using CBR equipment, or by adopting in situ CBR procedures. In the latter case, the absence of containment during testing can affect the outcome of tests on granular soils. A useful discussion on the CBR test (with respect to pavement design) is given in the Highways Agency Advisory Note HA 44/91 (HA, 1995).

Like the MCV, the CBR test is empirical but can be approximated to undrained shear strength (c_u) using (Black and Lister, 1978).

$$c_u = 23 \times \text{CBR} \text{ (kN/m}^2\text{)} \quad (8.2)$$

Direct relationships may be established with shear strength and dry density by means of relationship testing, should this be desired (see Section 8.4.8).

It is important to realise that CBR is neither a true measure of shear strength or stiffness; it is instead a ratio of performance relative to a base value established on the Californian limestone. Pavement engineering in the UK has moved away from the use of CBR testing of soils, and instead reflects long-term equilibrium moisture content CBR values determined based on soil plasticity (HD25/94 (HA, 1994b), LR 1132 (Powell *et al.*, 1984; Coney *et al.*, 2012). However, the CBR test continues to be very valuable as part of an earthworks assessment because compaction tests can be undertaken in CBR moulds, which enables the prepared samples to be tested to determine the CBR value of each of the compacted samples, thus being a very useful practical solution (as mentioned at Section 8.1).

8.4.7 Permeability test

Landfill liners should reduce the uncontrolled egress of leachate formed within the landfill to the smallest possible quantity. Flow of leachate through the liner is one of the predominant processes of transfer, so the design and construction control of clay liners using permeability criteria is an obvious choice. There are two principal approaches:

- use of in situ test methods such as shallow ponds and the sealed double ring infiltrometer
- use of laboratory test methods, including the permeameter, the triaxial cell, the oedometer and the hydraulic or Rowe cell.

Shallow ponds have been used for measuring the in situ permeability of clay liners (e.g. Elsbury *et al.*, 1990). The liner is constructed using methods appropriate to full-scale work and an area is bunded off where under-drains were installed prior to liner construction (the test pad). Water is introduced under controlled conditions, usually for a period of days or weeks depending on clay liner permeability, and the permeability $k(m/s)$ is calculated using the expression

$$Q = kiA \text{ (m}^3\text{/s)} \quad (8.3)$$

where Q is the total quantity of water passing through the liner per unit of time; i is the hydraulic gradient, and $A \text{ (m}^2\text{)}$ is the area of the test section. For low permeability materials, evapotranspiration effects could be significant and suitable protection for the ponded area may have to be provided.

The sealed double ring infiltrometer is an ASTM test (D5093-02 (ASTM, 2008)) to which reference should be made for full details. Essentially, the infiltrometer comprises a sealed inner and an outer ring grouted into the material to be measured. Normally, this is part of the liner (the test pad), which has been placed and compacted to the intended specification. The sealed inner ring is connected to a flexible water bag that allows water to flow downwards into the test pad. The water bag is weighed before and after each test and the flow Q per unit time calculated. Permeability may then be determined using Equation 8.3, where A is the area inside the inner ring. The sealed outer ring's function is to provide a reservoir of water to flow downwards into the test pad at the same time as that from the sealed inner ring, thus ensuring one-dimensional flow for the latter. According to the

ASTM, the test is suitable for materials having infiltration rates in the range from 10^{-7} to 10^{-9} m/s.

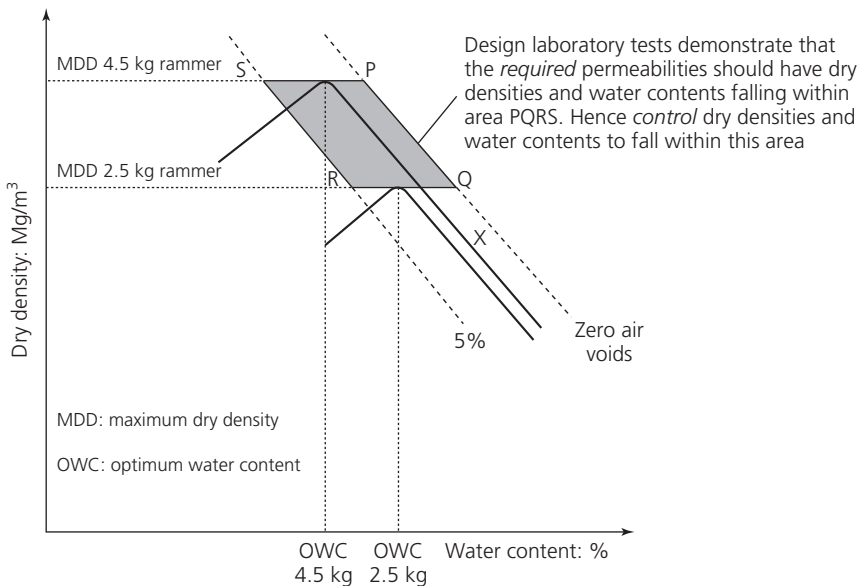
The alternative approach to in situ test methods is the laboratory tests quoted above, of which the fixed wall permeameter (falling head) and the flexible wall triaxial cell (constant or falling head) are the most popular (Murray, 1998). The former equipment is cheap, relatively simple and should be suitable for testing remoulded samples at the design stage. However, there may be leakage of permeant around the sides of a test specimen extracted from the liner during construction control, with inaccurate results as a consequence. This problem is avoided in the flexible wall triaxial cell, where the test specimen is enclosed in a membrane, and the specimen may be consolidated to an effective stress appropriate to that in the field. If constant head procedures are employed, the test duration is far longer than for the fixed wall permeameter. It also requires much more sophisticated laboratory facilities, which, although presenting no problems to most commercial laboratories for testing at the design stage, would seldom be found on the majority of construction sites.

The advantages and disadvantages of the commonly used permeability test methods are summarised in Table 8.6.

An issue in permeability testing for landfill liners and other geotechnical structures, not yet resolved, is the question of hydraulic gradient. Often in laboratory tests it can be greater than 30 but in the field less than 2 (Edelmann *et al.*, 1999). The effect that this

Table 8.6 Advantages and disadvantages of commonly used permeability tests (figures in brackets refer to the part and test numbers of BS 1377:1990)

Type of test	Advantages	Disadvantages
Falling head fixed wall permeameter (test specimens remoulded in the permeameter) (5 : 5)	Equipment cheap and relatively easy to employ. Suitable for site and test duration relatively short. Can be used as a constant head test although test duration lengthened	Test specimen cannot be consolidated to expected in situ effective stress
Falling head fixed wall permeameter (test specimen cut from a tube sample or directly from the lining) (5 : 5)	As above	As above. Also permeant may leak between the sides of the specimen and the sides of the permeameter
Constant head flexible wall (undisturbed or remoulded test specimen) (6 : 6)	Test specimen can be consolidated to expected in situ effective stress. Test specimen may be back-pressured to induce saturation	Equipment expensive, complicated to set up and to use. Not suitable for most sites. Test duration long

Figure 8.12 Design and control approach for landfill liners

difference has on predictions of permeability and erosion made on the basis of laboratory tests is not clear but experience to date suggests that it may not be critical. Nevertheless, it is a strong argument for the use of in situ test methods whenever possible.

An approach to the whole issue of design and control for landfill liners would be to employ the more complex test procedures (including field tests) during design and to correlate the results with simpler and more rapid test methods for construction control. Thus comprehensive laboratory studies, preferably using the flexible wall triaxial equipment, could be conducted on various remoulded samples to determine the water content and the dry density at which the desired permeability (usually less than 10^{-9} m/s in the UK) is obtained. These values would be plotted (area PQRS in Figure 8.12). Dry density and water content determinations may then be employed as a control in the field, to show that the results fall within the desired area. This approach is similar in principle to the one given in Figure 8.3.

Note that the upper limit for the water content is likely to correspond to that controlling trafficability and, for sloping landfill liners, stability (see Sections 3.4 and 3.6). A good discussion on testing for clay liners is given by Murray (1998).

8.4.8 Relationship testing

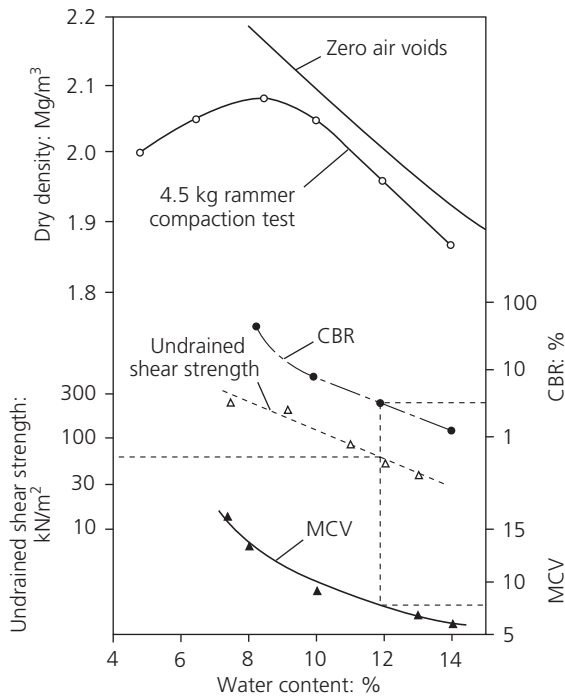
Relationship testing is the performance of several different tests on the same fill to measure several different engineering properties, with each test conducted at a number of different water contents. Using such a test programme, it is possible to correlate the results of the various different tests and to determine directly what, for example, a

particular MCV value means in density, strength or CBR terms. It is particularly useful in determining how responsive a certain test type is for a given soil. For example, the MCV test is not generally used for granular soils (Figure 8.11) and relationship testing may help to disclose the granular content at which the test loses efficacy.

Relationship testing is popular for new fills: compaction tests are performed to measure the variation of dry density with water content and the air voids relationships; undrained shear strength tests are made on the same fill to measure the variation of strength with water content; and MCV and CBR determinations may also be made on the same fill for varying water contents. The results, plotted on the same graph, permit the designer to correlate the various engineering properties and to select a design value. In selecting the test range, the Engineer will be guided by the initial water content at the borrow and the likely site weather conditions. In upland areas, a concentration on soil wetter than initial water content would be reasonable. Testing should always explore the extremes: say between about 40 and 150 kN/m² in undrained shear strength terms. These strengths could constitute the approximate limits at which clay fill may be trafficked and compacted, respectively. It is also important to extend the relationship testing past the anticipated acceptable range of water content, particularly on the wet side in order to explore how quickly the fill loses strength when placed at increasing water content.

As described at Section 8.1 relationship testing should be fundamental to the earthworks designer gaining an understanding of the behaviour of a fill material. Consequently, other references to relationship testing have been incorporated within the preceding part of this chapter to illustrate certain design topics, with an example included as Figure 8.6. The compaction test is always undertaken on proposed fill materials, and the designer should schedule other forms of testing on compacted samples to complement the data. The complementary tests can either be undertaken on all or just some of the compacted samples depending on the issue being investigated. The method is particularly important for the assessment of a fill material that the designer has not worked with before, or when the intended end use is more onerous than is covered by previous experience.

At the start of this chapter a comment was made on the lack of laboratory tests well suited to measuring the stiffness of re-compacted partially saturated soils as part of the relationship testing process (see Section 8.1). On a project of particular complexity when the designer needs to attempt to correlate undrained strength to stiffness then one option could be to prepare a sample of fine soil compacted at a water content slightly above optimum so that it has low air voids, and can then be used for effective stress triaxial cell testing. If the process of sample preparation is successful, then a back pressure could be applied prior to testing (hence the need to start with a sample of low air voids). This testing would give an indication of the stiffness that could be anticipated for a fill placed and compacted in a condition that would be stable in the long term. Such an approach might be worth exploring if the intention is to control the construction works by plate load testing. Effective stress testing on compacted samples was successfully employed by Loach (1987) to enable repeat load triaxial testing (of Mercia Mudstone, London Clay and Gault Clay) with suction measurements to develop correlations between resilient modulus and CBR.

Figure 8.13 Example of relationship testing of Glacial Till (from Parsons, 1992)

A classic example of this approach is given in Figure 8.13. A useful discussion on relationship testing, including a sample test package, is given in the Highways Agency Advice Note HA 44/91 (HA, 1995). The approach forms an underlying principle of BS 6031:2009 (BSI, 2009) for the setting of acceptability limits as described in Section 8.4.2.

The cost of relationship testing is relatively high because a large number of laboratory tests are undertaken on a single soil type. The benefit of this investment, however, is considerable due to the great understanding of the likely behaviour of the fill that the results will provide. The approach provides an opportunity to observe the likely behaviour of the fill under a range of conditions, and to explore potential difficulties with that fill. The required scope of work for relationship testing should therefore be carefully considered by the designer to maximise the chances of gaining the information required, see example included in Figure 8.14.

8.4.9 Plastic limit and plasticity index

In the first edition of this book Trenter did not include reference to the use of Atterberg limits as a means of controlling earthworks fill acceptability. Yet a designer should always consider these properties when reviewing a fine fill material's likely behaviour. Therefore, additional text has been added here to capture issues to consider, and some experience that is still useful even if only to enable an initial assessment of a fill.

Figure 8.14 Example scope of work for relationship testing on dry fine soils to assess risk of volume and strength changes post-compaction

Soil sampling

A large number of bulk samples are required for each borrow pit fill proposed to enable the required suite of soils laboratory tests to be undertaken on a single fill material type so that the different test results can be directly compared for that fill.

In addition, representative samples of the fill shall be recovered at the borrow pit to record the range of natural water content that can be expected during borrow pit operation.

Material properties testing

1. Natural water content (material sampled in situ at the borrow pit) (BS 1377:1990: Part 2: Test 3)
2. Particle size distribution by wet sieving (BS 1377:1990: Part 2: Test 9.2)
3. Particle size distribution by hydrometer analysis for fine particles (BS 1377:1990: Part 2: Test 9.5)
4. Minimum density of sands (BS 1377:1990: Part 4: Test 4) undertaken at natural water content
5. Particle specific gravity (BS 1377:1990: Part 2: Test 8)
6. Atterberg limits (plastic limit and liquid limit) if the soil type has >15% passing 63 µm sieve (BS 1377:1990: Part 2: Test 4.3 and Test 5)

Compaction tests

7. BS standard Proctor test 2.5 kg method utilising CBR mould (BS 1377:1990: Part 4: Test 3.4)
8. BS modified Proctor test 4.5 kg method utilising CBR mould (BS 1377:1990: Part 4: Test 3.6)

Note: undertake compaction tests at six (rather than the standard five) water contents, centred around the anticipated optimum water content, in order to improve the definition of the compaction curve.

Strength testing of compacted samples

9. CBR on unsoaked sample: testing each sample from the 2.5 kg compaction test (BS 1377:1990: Part 4: Test 7)
10. CBR on soaked sample: testing the samples from the 2.5 kg compaction test that are in the range OWC to OWC –4% (BS 1377:1990: Part 4: Test 7)
11. Collapse testing of compacted fill material (BS 1377:1990: Part 5: Test 4). Testing of samples compacted to >95% maximum dry density, determined in accordance with item 7 above. Testing of samples at a range of inundation pressures. Loading stages and inundation pressures to be established based on overburden pressure and the results of items 1–8 above.

Presentation of results

The set of test data shall be presented as a 'relationship test' in a similar manner to that described in Clauses 7.6.4 and 7.6.5 of BS 6031:2009. The data shall be presented as follows

- 2.5 kg compaction test results shall be presented as dry density versus water content at time of compaction in the normal manner.
- The air voids lines shall be shown on these results determined based on the specific gravity of the soil that is used for the compaction test programme (air voids 0%, 5%, 10% and 15% to be plotted).
- The minimum dry density for that soil type shall also be shown on the plot.
- Unsoaked CBR results shall be presented beneath the compaction curve and so that the CBR and density at the same water content can be directly read.
- The reduction in CBR on soaking shall be indicated.
- If possible the 4.5 kg compaction curve for the same soil type shall be indicated (but shall be as a thin line given that the CBR tests are for the 2.5 kg compaction samples).

In the UK the SHW is so widely used that it is now set as the default specification within BS 6031:2009 (BSI, 2009). It utilises a relatively broad classification system for fill materials that does not subdivide fine soils into classes based on plasticity index. However, many other systems of fill classification do subdivide fine soils on this basis because the behaviour of a low plasticity clay will be noticeably different to that of a high plasticity clay. Even when working to the SHW designers should always give consideration to a fine soil's plastic limit and plasticity index to help understand the material's behaviour, enable comparison to other similar fills and aid the use of published data.

As described at Section 8.3.1, the SHW classifies clay soils as unacceptable for use as fill material if the liquid limit exceeds 90% or plasticity index exceeds 65%. This is to avoid utilising the most highly plastic clays that are particularly prone to heave.

The relationship between a fine soil's water content and plastic limit provides a useful reference framework to help understand undrained strength and also compaction of that fill. A cohesive fill material placed at a water content around the plastic limit tends to produce an acceptable compacted density and also 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. Section 8.3.4 illustrated common problems for fine fills compacted significantly dry of plastic limit.

Within the SHW the water content is one option for setting a project's acceptability limits for general cohesive fills. The LAL, defined in Figure 8.8, for wet cohesive fill (Class 2A) and UAL for dry cohesive fill (Class 2B) are set as a water content of plastic limit (PL) – 4%; that is, this value is used to define the difference between wet and dry cohesive fill. This reflects experience that for many clay fills the OWC from the 2.5 kg rammer test is often at or slightly below the plastic limit.

Up until about the 1990s, the UAL for water content of wet cohesive fill was commonly set within the project specification based on a multiplier of plastic limit. The range of multiplier on plastic limit for upper bound acceptable water content varied with material, and usually plasticity index, but accepted limits were in the range $\text{PL} \times 1.1$ and $\text{PL} \times 1.3$. Arrowsmith (1978) presents a range between $\text{PL} \times 1.1$ and $\text{PL} \times 1.2$ for Glacial Clays in the north west of England, from which a value of $\text{PL} \times 1.2$ was chosen to produce a minimum compacted remoulded shear strength of 70 kN/m^2 .

8.5. Compaction trials

8.5.1 General

A compaction trial is undertaken by constructing a small section of earthworks, which is then tested to check that the quality of earthworks achieved is acceptable. The trial section is constructed under representative site conditions, using the fill material and method of working that is proposed for the main works.

Compaction trials are advisable for all earthworks operations, and are essential where sensitive or heavily loaded structures are to be placed on the fill or a landfill liner is to

be constructed. They apply to all methods of design and control; indeed, they should be considered an extension to design in that they enable details of layer thickness, placement water content, plant type and number of passes to be finalised for the fill(s) under consideration. When working to a method specification the compaction trial should be used to check that the intended density and air voids will be achieved for the particular fill material and that the acceptability limits set within the specification are appropriate.

Compaction trials also provide information on the suitability of the testing procedures selected for quality control of the earthworks. For this reason, contract documents should be written to permit the compaction trials to proceed before the main earthworks operations commence, and to permit changes in procedure (if necessary) on the basis of the trial results.

For most earthworks activities the first section of earthworks to be constructed with a particular fill material, or a different method of working, can be designated as the compaction trial. It is advisable for this section to be observed by the designer and tested at a rate of testing greater than that required for the main works. The results are then assessed to confirm that the designer's assumptions are satisfied, and approval is given for the trial section to be incorporated into the permanent works (or to be excavated and replaced), and for the commencement of the main earthworks. This approach minimises the additional costs or delays associated with undertaking trials.

It is often thought that compaction trials are designed to investigate the performance of compaction plant only, but the performance of the construction plant placing, spreading and dozing the fill can also be investigated. This is particularly important with weak rocks, such as mudstones, poorly cemented sandstones, the weaker limestones and chalks, and the stiffer over-consolidated clays. Compaction trials provide an opportunity to determine the extent to which different construction plant breaks down the fill to produce a well-graded material with good compaction properties.

Compaction trials also provide an excellent opportunity to test methods of conditioning soils and weak rocks (see Section 10.5). Chalk, for example, when placed wet, may require time to harden before compaction and before the next layer can be placed (see Section 5.3.3). This essential information can be established during the trials and under controlled conditions.

8.5.2 Method of approach

Neil Trenter's recommended method of approach for a compaction trial is set out below; this approach is well suited to assessing a fill material over an anticipated range of acceptable water contents. Trenter (1999) proposed the following stages.

- Two or more strips of ground should be cleared of topsoil, which are wide enough to take the full width of the compaction plant and are sufficiently long to enable the plant to be manoeuvred into position and to reach its operating speed; a length of some 30–40 m is often found suitable, but the precise length selected will depend on the size and speed of the plant and the space available. The width adopted should allow for the fact that during a comprehensive series of trials, at

least eight layers will have been placed raising the fill height to 1.0–1.5 m above ground level; at a side slope of 1 (vertical) in 1 (horizontal); this would increase the width of the strip by at least 1 m either side.

- In some cases, the natural ground may be stiffer and more resilient than a large thickness of fill and a layer of fill placed and compacted directly on it will yield higher density or strength values than if placed on fill alone. Consequently, where these conditions occur, at least three layers of fill should be placed and compacted before testing begins in order to obtain reasonably representative results.
- The first layer to be tested will usually be placed at the water content at the cutting or borrow pit from which the fill is to be sourced. The material should be compacted using the plant selected for the main works construction, placed in different layer thicknesses and control tested after various numbers of passes. At this stage, the results of the control tests could be reviewed and the optimum number of passes selected.
- At the same time as these compaction trials are underway, trials could proceed on the other strip at an increased (or decreased) water content. The compaction trials should provide a good opportunity to determine the feasibility of conditioning by wetting (or drying): as noted elsewhere, adding water in such a way that the whole fill receives water in equal amounts can be difficult, if not impossible, for most stiff over-consolidated clays; likewise, conditioning by most of the methods, apart from the rotovator mentioned in Section 10.5, can be particularly weather dependent.
- After three different layer thicknesses have been tested together with different placement water contents, the performance of the compaction plant can be reviewed and other equipment substituted. If two or more sources of fill are in contention, then more than two strips may be necessary, if the trials are to be completed in good time.

In many cases the compaction trial will not need to be to such a high standard as that described above, especially when working with a fill material that is already well understood. The designer should judge what is required for the particular project and the issues that need to be assessed. In all cases the laboratory relationship testing data should be used in conjunction with the results of the compaction trial to maximise the understanding of the fill material under consideration.

8.6. Control test frequency

A matter of importance to the successful outcome of a project is the frequency of control testing, that is the number of control tests to be performed to ensure that the specification is being met. The topic of quality assurance and quality control is covered in Section 10.7.1, but a comment on control test frequency is needed here because this will be the final aspect of establishing the earthworks specification. Control test frequency depends on several factors:

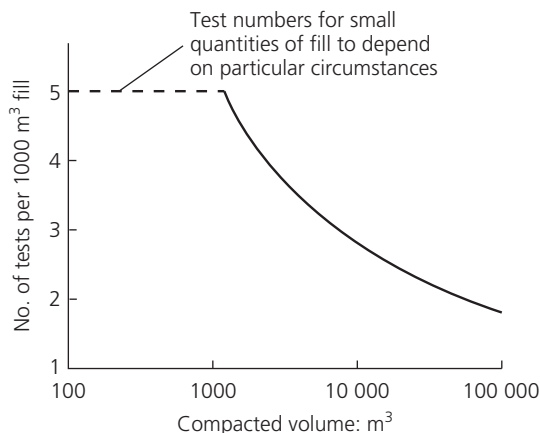
- the volume of fill placed
- the nature of the earthwork and proposed end use
- the uniformity of the fill, for example, whether just one soil (or rock) type or several, and whether the material type(s) are uniform in themselves

- the relative complexity of the fill material, and whether particular potential difficulties have been identified by laboratory relationship testing that will require careful management during the works
- the nature of the particular test; for example, in situ density tests will be undertaken each day but compaction tests on a weekly basis
- the outcome of the compaction trials, that is whether or not generally consistent control test results were achieved; the wider the spread of results during the trials, the more tests should be performed during main works construction
- the progress of the main works compaction itself.

Small volumes of fill often mean small sites, with much manoeuvring of the plant (which can disturb finished work) and difficulty in achieving a production line approach. Relatively more control tests than for large volumes are therefore warranted. Trenter and Charles (1996) suggested guidelines for the minimum control test frequency for fills supporting low rise housing, illustrated in Figure 8.15: for larger sites of say 100 000 m³ fill volume or above, control testing was suggested at the rate of about two per 1000 m³ of fill placed; for smaller volumes, the control testing rate was suggested at about five per 1000 m³.

The guidelines illustrated in Figure 8.15 should be regarded as preliminary and much dependent on the factors described above. Fill for a high-speed railway embankment would demand more control than that for a corresponding road embankment, and fill supporting buildings (with large expanses of glass panelling) might demand more than both. If a particularly heterogeneous fill were being produced at the cutting or at the borrow pit, the control test rate should be significantly increased, as it should if standards of workmanship on site were unimpressive. Likewise, for landfill sites, where maintaining strict permeability criteria were essential, a significantly enhanced control rate might also be necessary.

Figure 8.15 Guidelines for minimum control test frequency (from Trenter and Charles, 1996)



Suggested test frequencies for classification and acceptability testing for earthworks constructed under the SHW (HA, 1998, 2013) is given in Table 3.1 of the Highways Agency Advice Note HA 44/91 (HA, 1995) and in Clause 8 of BS 6031:2009 (BSI, 2009). When working to the SHW the designer must set the test frequencies for the project within Appendix 1/5. In this advice note, some tests are recommended on a daily or weekly basis (i.e. are independent of fill volume), whereas others are recommended per stated volume of fill, this reflects collective experience of the normal variability of these parameters and the level of influence that the particular fill material property will have on the control of the works.

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Chapter 9

Excavation

9.1. Introduction

Excavation is the process by which soil or rock is removed in order to reach the required line and level. If the overall quantities of earthworks material for the scheme are in balance, the material resulting from such excavation will be employed as engineering fill for embankment construction. When the material is unacceptable, or the overall earthworks quantities are in deficit, acceptable fill material may be excavated from specially designated areas either close to the route corridor or from a greater distance, often termed ‘borrow pits’. Even when the earthworks are in balance and the material is predominantly acceptable, on some earthworks contracts it may be more cost effective to excavate borrow pits locally in order to meet the overall construction programme, as haul distances are prohibitive or parts of the scheme are inaccessible. The latter is usually the case where the route corridor is traversed by major rivers, roads or railways, where additional local excavation is more cost effective when compared to the cost and programme time required to construct a temporary crossing.

Material unacceptable for use as engineering embankment fill and/or surplus acceptable material may be run to tip remote from the route corridor. In the UK, however, this activity now accrues a landfill tax per tonne of material disposed. This additional cost, together with the additional cost of off-site disposal, has led to more effort to condition unacceptable material won from earthworks cuttings, usually by drying or the addition of a small percentage of lime prior to placing (refer to Section 10.5.2). In addition, excess material is often incorporated within the route corridor by its use as landscape fill or noise bunds.

Contaminated material is usually disposed of ‘off-site’ as special waste into specially designated disposal sites. These sites are, for some contracts, a considerable distance from the scheme and in these cases ‘soil hospitals’ have been developed during construction where the contaminated material can be treated on site for incorporation into the works.

The nature of the soil and rock to be excavated is normally determined during the site investigation and sections are prepared showing the ground and groundwater conditions in the area concerned. Locations at which excavation will be necessary below the groundwater table should be clearly identified on the sections, as should areas of unsuitable fill. Presumed excavation characteristics of the soils and rocks can be illustrated on these section drawings, together with information relating to fill placement, suitability and control.

Normally, a further investigation will be carried out early in the construction programme to characterise more accurately the nature and likely acceptability of material won from cuttings to be incorporated into the completed works. This usually takes the form of trial pitting at a frequency of some 50 m intervals for highway schemes.

9.2. Soil excavation

Most soil can be excavated readily in normal weather conditions and the choice of plant is based on achieving production targets, taking into account manoeuvrability and trafficability during placement. A brief description of some plant for excavation and haulage purposes is given in Appendix 2.

9.2.1 Impact of groundwater and weather

Water can have a controlling influence on the behaviour of some soils during excavation. Granular soils and clay soils of high plasticity are usually more tolerant of excess water than fine soils of low to intermediate plasticity. Particular care should be taken with low plasticity soils, such as Glacial Tills when wet, because they can turn to a slurry and lose sufficient strength to become virtually unworkable. Tills often contain water-bearing granular layers or lenses, which may assist with introducing groundwater into the cohesive material and facilitate the slurring process.

All earthworks should be undertaken with the possible presence of water in mind, either as run off from rainfall or as groundwater, and steps should be taken to minimise its impact. Clause 602.15 of SHW (HA, 1998, 2013) requires that earthworks should be kept free of water, including rainwater and water 'entering the earthworks from any source', which will include groundwater. The same clause requires that water should be lowered, by appropriate measures, sufficient to allow the permanent works to be constructed. Clause 602.16 lists actions to be taken in carrying out the requirements of Clause 602.15, which are included in Section 10.3.

9.2.2 Misidentification of rockhead in glacial terrains

In glacial terrains it may be difficult to assess rockhead depth accurately due to the presence of large cobbles or boulders in overlying tills. Disruption and delays may occur if what was considered to be a boulder at site investigation stage is shown to be rockhead during excavation for the main works. In such cases, excavation would have to continue in rock, although excavation in soil containing cobbles and boulders had been programmed. The opposite circumstance can be just as significant: where what was considered to be bedrock at the site investigation stage is found during construction to be a boulder. This can be of particular concern for foundations, because deeper penetration may be necessary in order to secure the required load-carrying capacity. Again disruption and delays may occur.

An example of the latter was encountered during the ground investigation for the eastern section of the M65 Blackburn southern bypass. The preliminary ground investigation identified a length of route corridor underlain by some 1.5 m of Coal Measure Sandstone overlying some 4.5 m of heavily fractured Coal Measure Mudstone. The main investigation discovered this sequence actually to comprise very large sandstone boulders

overlying Glacial Till, comprising entirely locally derived Coal Measure Mudstone, with rockhead at some 6.5 m below ground level.

Great care is necessary when undertaking site investigations in such conditions. A minimum penetration of 3 m below presumed rockhead and careful core logging are necessary to establish the true position. This requirement may be greater, depending on the geological relationships identified during the preliminary site investigation. Further investigation during the main works contract may be necessary to clarify the position.

Confusion over rockhead level can also arise because of buried channels; rockhead reliably established at one location may be encountered at a completely different elevation a short distance away because of channelling in glacial times. Geophysical methods have been found advantageous in locating rockhead on many occasions, but accompanying borehole control is always necessary.

9.3. Definition of rock

9.3.1 General

In strength terms, rock can be classified as shown in Table 9.1. Seismic wave velocities are also shown.

Table 9.1 Rock strength classification (after Bell, 1992)

Rock hardness description	Identification criteria	Unconfined compressive strength: MN/m ²	Seismic wave velocity: m/s
Very soft rock	Material crumbles under firm blows with a geological pick; can be peeled with a knife; pieces up to 30 mm thick can be broken by finger pressure	1.7–3.0	450–1200
Soft rock	Can just be scraped with a knife; indentations 1–3 mm produced under firm blows with a geological pick; dull sound under the hammer	3.0–10	1200–1500
Hard rock	Cannot be scraped with a knife; hand specimen can be broken with a single firm blow of a geological pick; rings under the hammer	10–20	1500–1850
Very hard rock	Hand specimen breaks after several blows of a geological pick; rings under the hammer	20–70	1850–2150
Extremely hard rock	Hand specimen breaks after many blows of a geological pick; rings under the hammer	>70	>2150

According to Bell (1992), the very soft and soft rock categories correspond roughly to 'easy ripping' and 'hard ripping', respectively, and therefore correspond to 'weak rock' as defined in Section 4.1. Some engineers believe that seismic velocity (shown in Table 9.1) provides only a very rough indication of rock behaviour on excavation and consequently the data shown should be treated with caution.

9.3.2 'Rock' in contract documents

Although 'rock' can be readily identified by most Engineers and engineering geologists, there can be difficulty in framing an adequate definition of rock for contract purposes. In many contracts in the UK, rock is expected in relatively small quantities and, if so entered in the bill of quantities, attracts high rates from tendering contractors. If the bill is subsequently proved wrong and much larger volumes of rock are encountered, the employer pays disproportionately. Another potential source of difficulty is that the rock is harder or stronger than could have been foreseen at the time of tender. A reasonably correct definition of 'rock' as opposed to 'soil', together with its hardness or strength, is therefore important. It is normally defined in one of the following ways (some disadvantages of the definitions follow in brackets).

- By reference to a named geological stratum or well known rock type, for example, Old Red Sandstone or Granite. (Relating 'rock' to a named stratum or rock type can lead to arguments over correct identification; in any case, boulders in surface drift deposits requiring excavation may be of a completely different kind from those of the underlying solid rock named in the contract; this is especially the case in glacial terrains, where erratics are common.)
- By reference to strength characteristics, for example a minimum unconfined compressive strength or a strength range. (Relating rock to an unconfined compressive strength or strength range may also be flawed because the overall rock mass, which is governed by discontinuities, may be more important in excavation terms rather than the intact rock, on which the unconfined compressive strength is measured.)
- By reference to a means of excavation, for example, by blasting or the use of certain tools. (Defining rock by the tools used for excavation removes from the contractor the responsibility for using his own skill and experience in selecting a means of excavation.)
- By reference to its volume, for example rock fragments to be of a certain size. (Relying on volume, by reference to individual masses could lead to disputes over measurement.)
- By reference to designated deposits on a site plan, identified by site investigation data. (Reliance on designated deposits on a site plan, based on interpretation of rock from borehole data can be unreliable, unless a thorough site investigation is performed.)

The *Method of Measurement for Highway Works* (DfT, 1991), does not refer to rock, but to hard material and uses the following methods to define it:

- material that requires the use of blasting, breakers or splitters for its removal but excluding individual masses less than 0.2 m³ (i.e. by reference to means of excavation and by volume)

- those strata or deposits so designated in the contract (i.e. by reference to a named stratum).

This definition has been removed from the 2001 version of the *Method of Measurement* (DfT, 2001, 2006a; DfT 2001, 2006b) although an extra/over item for excavation in hard material is still included.

Of the various methods discussed, the one that relies on close characterisation of the site investigation data may be the most satisfactory if it produces a reasonable picture of the conditions relating to excavation. Boreholes, trial pits and geophysical methods are adopted to locate and describe the rock and to provide a full description of rock discontinuities. A comprehensive suite of strength determinations (unconfined compressive strength and point load tests) is performed to characterise the intact rock strength. This information may be employed with excavatability schemes such as that proposed by Walton and Wang (1993) or Pettifer and Fookes (1994) and discussed further below. Core samples should be made available to tendering contractors, with the borehole records alongside, so that the approach adopted by the engineering geologist responsible for logging the core can be judged by the contractors' specialists. Photographs, when used on their own, often fail to provide the same level of information as cores themselves.

Another approach might be by reference to productivity (say in m^3/h), using soil excavatability as a comparator. Here, the Engineer or designer would specify rock as being that material whose removal by soil excavation methods is not considered feasible, and which demanded the appropriate plant to maintain productivity at a certain figure (say 50%) of that achieved during soil excavation. This method relates rock excavation to known productivities, rather than physical attributes such as volume, type or strength, and it provides an inducement to the contractor to use its skills and experience to devise appropriate excavation methods, without specifying the tools to be adopted. However, note should be made of the substantial differences in the nature of bucket excavation between soil and rock, and the effect it could have on the 'flowability' of the material into and out of the excavator bucket. The approach also recognises that excavation is not only about the physical break-up of the rock but is also about loading and hauling, both important parts of the excavation cycle.

Excavatability trials are as important as compaction trials (see Section 8.5). Note that contract documents should be written in such a way as to permit changes in procedure on the basis of the trial results.

9.4. Rock excavation

9.4.1 General

Excavation in rock may be accomplished by:

- direct excavation
- pre-treatment to break up or fragment the rock
- blasting.

Under favourable conditions, rock can be excavated directly by bulldozer, back hoe or face shovel. This is especially the case at, or close to, rockhead, where it may be significantly weathered. With decreasing weathering and with increasing rock strengths and unfavourable discontinuity patterns, pre-treatment is necessary. Such pre-treatment usually comprises ripping, using a tine attached to the rear of the bulldozer. This may break up the rock sufficiently for its removal by bulldozer blade. In the case of strong unweathered rock with unfavourable discontinuity patterns, even ripping with powerful plant may prove inadequate as a pre-treatment and the rock may have to be loosened by blasting or ultimately fractured by blasting.

Blasting is a topic with substantial safety and environmental concerns. Specialist publications and/or organisations should be consulted if it is to be employed.

9.4.2 Use of excavation schemes

A chart relating the excavatability (diggability) of rocks to factors such as rock strength and discontinuity spacing was prepared by Franklin *et al.* (1971) and is reproduced in Figure 9.1(a). Excavatability was subdivided into dig, rip, blast to loosen and blast to fracture. Since that time, modern excavation equipment has changed considerably and an updated approach is given by Walton and Wang (1993), reproduced in Figure 9.1(b). This relates excavatability to rock (point load) strength and to discontinuity spacing as before, but the frequently used rock quality designation has also been added. Also incorporated is the 'diggability index rating' of Scoble and Muftuoglu (1984), given in Table 9.2, which takes into account rock weathering, strength and discontinuity spacing.

This index rating may be used as the basis for a diggability classification for excavators, presented in Table 9.3.

Naturally, schemes and classifications such as these can only provide an impression of the actual conditions as far as excavatability is concerned, but their use may be helpful at the preliminary stage and as a guide to setting up and carrying out excavatability trials.

The excavation scheme by Pettifer and Fookes (1994) may also be consulted.

9.4.3 Pre-treatment by ripping

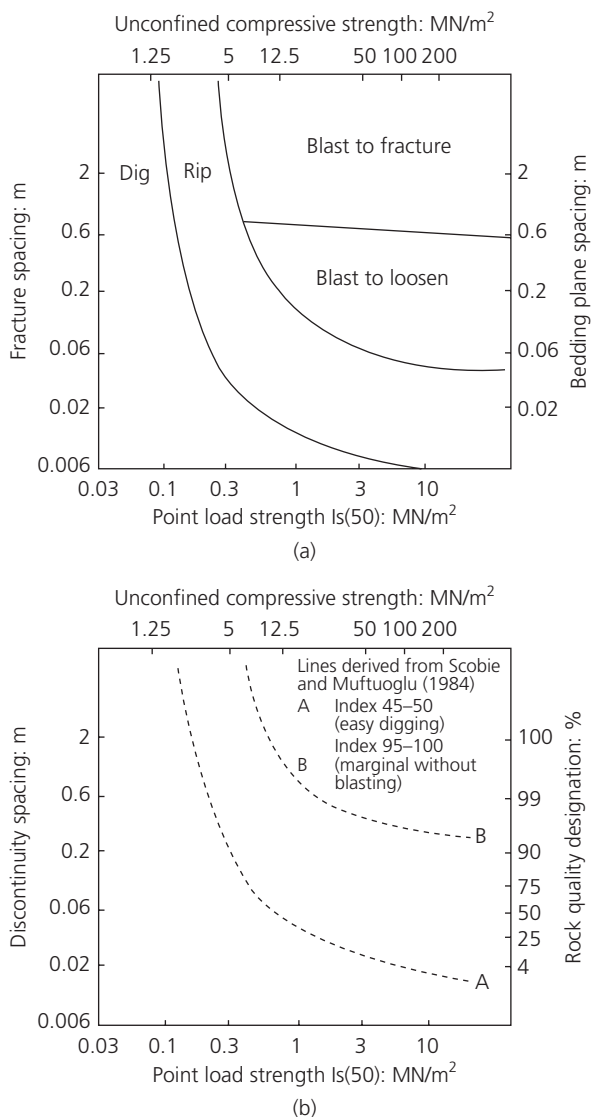
Pre-treatment by ripping is controlled by a number of factors including:

- size and power of plant
- method of working
- nature and orientation of discontinuities
- strength of intact rock.

9.4.3.1 Size and power of plant

It is self-evident that the weight and power of plant will influence its ability to excavate rock, as will the bucket size and geometry, which controls the ability of the fragmented rock to flow into and out of the bucket.

Figure 9.1 Excavatability chart (from Walton and Wang, 1993): (a) original diggability chart (from Franklin *et al.*, 1971); (b) modification of the original diggability chart covering modern hydraulic excavators



9.4.3.2 Method of working

If ripping is necessary, the direction of travel of the bulldozer and the spacing between runs will influence the outcome. The closer the spacing, the more intense the treatment but also the greater the cost and the time taken.

Table 9.2 Diggability index rating (after Scoble and Muftuoglu, 1984)

Parameter		Ranking			
Weathering	Completely	Highly	Moderately	Slightly	Unweathered
Rating: W	0	5	15	20	25
Strength: MN/m ² (UCS)	<20	20–40	40–60	60–100	>100
Is(50): MN/m ²	<0.5	0.5–1.5	1.5–2.0	2–3.5	>3.5
Rating: S	0	10	15	20	25
Joint spacing: m	<0.3	0.3–0.6	0.6–1.5	1.5–2	>2
Rating: J	5	15	30	45	50
Bedding spacing: m	<0.1	0.1–0.3	0.3–0.6	0.6–1.5	>1.5
Rating: B	0	5	10	20	30

Note: UCS = unconfined compressive strength

Run direction may be dictated by the size and the shape of the working area. It may not always be possible to select the run with respect to geological features such as dip and/or discontinuity spacing. In circumstances such as these, the combination of hydraulic breaker and light tractor plus ripper may be more productive than a much heavier bulldozer used on its own.

Assessing an appropriate method of working requires considerable planning, involving the plant engineer and his engineering geologist colleague. It should certainly not be dictated simply by what plant happens to be available at the time.

Table 9.3 Diggability classification for excavators (after Scoble and Muftuoglu, 1984)

Class	Ease of digging	Index (W + S + J + B)	Typical plant that may be used without blasting
I	Very easy	<40	Hydraulic backhoe <3 m ³ , e.g. CAT 235D
II	Easy	40–50	Hydraulic shovel or backhoe <3 m ³ , e.g. CAT 235 FS or 235 ME
III	Moderately easy	50–60	Hydraulic shovel or backhoe >3 m ³ , e.g. CAT 245 FS or 245 ME
IV	Difficult	60–70	Hydraulic shovel or backhoe >3 m ³ , e.g. CAT 245 or O&K RH 40 (short boom if a backhoe)
V	Very difficult	70–95	Hydraulic shovel or backhoe >4 m ³ , e.g. Hitachi EX 1000
VI	Extremely difficult	95–100	Hydraulic shovels and backhoes >7 m ³ , e.g. Hitachi EX 1800 or O&K RH 75

9.4.3.3 Nature and orientation of discontinuities

All rocks contain discontinuities. For a given intact rock strength, the closer the spacing between discontinuities, the easier excavation will be and the greater the productivity. This is demonstrated in the excavatability chart of Walton and Wang illustrated in Figure 9.1(b). However, note must be taken of discontinuity orientation as well as spacing. Ripping is normally most favourable when the run is up-dip or at right angles to a major discontinuity set and where free faces (including previously excavated surfaces) are available.

Weathered discontinuities daylighting into the working surface and those with large aperture (openness) are usually favourable because of the ease with which the tine can penetrate the opening and the blocks can be dislodged, especially if the discontinuities dip at roughly 45° to the direction of the tines.

9.4.3.4 Strength of intact rock

The strength of the intact rock between the discontinuities is also an important determinant of rippability. It may be measured by using the unconfined compression test or the point load test conducted on rock cores. As would be expected, for a given discontinuity set, the higher the rock strength, the greater the resistance offered to ripping.

A good discussion of rock excavation by ripping is given by Pettifer and Fookes (1994).

9.4.3.5 Processing for acceptability

It is likely that rock excavated by ripping or blasting will not be acceptable for use as embankment fill material as Class 1 material to SHW as dug, and will require further processing on site in terms of being run through a crusher and possibly over grading screens. Similar treatment will be required to generate Class 6 specialist granular fill from as dug rock material.

9.5. Bulking and shrinkage

Bulking occurs when soil or rock is excavated: one cubic metre of soil or rock at the borrow does not translate into one cubic metre of fill in the lorry or when placed and compacted on site. A guide to bulking factors is given in Table 9.4.

The factors shown in Table 9.4 should be used when estimating transport haulage capacities, noting that the actual factor will depend on local circumstances and could vary throughout the borrow pit or cutting.

For most soils and rocks, the result of excavation, transport, placement and compaction as fill is a net bulking: this amounts to between 0% and 5% for the majority of soils and weak rocks and to between 5% and 15% for harder rocks. Some materials, particularly chalk and sand, demonstrate a net shrinkage: for chalk, this amounts to between 0% and 10% and for sands to between 0% and 5%. Bulking can have a significant effect on the balance of cut and fill volumes and hence the cost of a scheme.

Table 9.4 Guide to bulking factors for soils and rocks (from Horner, 1988)

Soil	'Bulking factor': %	Rock	'Bulking factor': %
Granular	10–15	Igneous	50–80
Cohesive	20–40	Metamorphic	30–65
Peat	25–45	Sedimentary	40–75
Topsoil	25–45	Chalk	30–40

The above phenomena of bulking should not be confused with wastage, which inevitably occurs on most construction sites. Typical reasons for wastage are:

- construction and patching up of haul roads
- the requirement for additional fill for embankment construction over soft compressible soils where the embankment foundation settles under applied load (this can involve significant quantities of fill and the amount should always be quantified as part of the earthworks balance)
- overbuilding of embankments (see Section 12.7.2)
- otherwise acceptable fill rendered unacceptable by wetting up
- inaccuracy in the calculation of cut and fill volumes
- presence of unforeseen poor ground requiring additional excavation and replacement.

Some 5% of fill volume is often allowed for wastage. Like bulking, wastage can also have a significant effect on haulage for a major project and should be allowed for when estimating transport costs.

Examples of planning and constructing earthworks are given in Appendix 3.

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Chapter 10

Placement and compaction of fill

10.1. Introduction

This chapter deals with the placement and compaction of fill material to construct earthworks. Before starting on site the earthworks should have been designed and a Specification prepared to suit the particular project requirements and the site conditions (see Chapter 8). Once on site the skill and experience of the earthworks foreman will be fundamental in ensuring that that operations progress efficiently and the design is delivered. As with excavation (see Section 9.2.1), weather has a major influence on placement and compaction. In some wetter parts of Britain, earthwork sites can close down over the winter period whatever the fill employed. Note should be taken of the two SHW (HA, 1998, 2013) clauses on handling water at earthworks sites, mentioned in Section 9.2.1.

10.2. Preparation of site prior to filling

Before starting earthworks construction the site should be cleared of all unsuitable materials including those listed in Section 8.3.1. Derelict foundations and underground services may have to be removed and, where excavations have to be made, due consideration should be given to the stability of adjoining structures and to health and safety requirements (Chapter 16).

Particular attention should be given to assessing potential water flow paths and the appropriate drainage requirements to ensure the earthworks can be constructed without problems (requirements for ‘pre-earthworks drainage’ are included within BS 6031:2009 (BSI, 2009)). Drainage grips should be provided on the uphill side of excavations and the provision for sump-pumping made where this is considered necessary.

During site preparation, topsoil is often stripped as one of the first actions to be taken. This is understandable if it is to be re-laid and is usually necessary if the area concerned is to receive fill. However, ground from which the topsoil has been stripped can immediately be turned into a quagmire by construction plant. For site areas providing short-term access without the benefit of haul roads, and not themselves receiving fill, it can be better to leave the topsoil in position to provide a better trafficking surface, although there is a risk of significant rutting within the topsoil.

Consideration should be given to whether removal of all the topsoil and subsoil from areas to be filled is actually necessary. Commonly, more topsoil and subsoil is excavated than is actually required for landscaping of the completed earthworks. The earthworks designer can often demonstrate that there is no geotechnical requirement to remove the

topsoil from beneath embankments of significant height because the load of the embankment will be sufficient to compress the topsoil fully (an embankment height of as little as 3 m is often sufficient provided vegetation is cut down to the ground surface). However, checks must be made as to whether there is any legal or contractual obligation to remove all of the topsoil and subsoil.

Once the pre-earthworks drainage has been constructed, major obstructions removed, and any topsoil/subsoil removal exercise completed, the area that is to receive fill should be regulated to provide a reasonably even formation surface that is ready for the placement of fill. This formation preparation will often include:

- removal of localised irregularities and obstructions to provide a level surface
- grubbing up of hedgerows
- cleaning out minor watercourses and removal of land drainage crossings under the earthwork, backfilling the original route, and diversion of the watercourse into a culvert
- interception and drainage of potential spring lines across the formation
- removal or perforation of areas of hardstanding to ensure that these do not act as a barrier to water beneath the embankment, or create hard spots particularly below shallow embankment construction
- excavation and replacement of existing soils assessed as too soft to be left in place beneath the embankment (see below)
- remediation of contaminated material if this is identified as being necessary
- benching on steeply sloping ground (see below).

When fill is to be placed on steeply sloping ground the construction of benches is necessary in order to provide a stable key into the formation and eliminate the risk of potential failure surfaces developing at the base of the fill. The construction of benches also helps to ensure that effective compaction is achieved in the lower layers of fill and that the finished earthwork is stable. The steepest gradient of slope that fill can be placed on without benching is generally between 1 : 5 and 1 : 8 (vertical : horizontal), depending on the particular circumstances. It is relatively common practice to construct the lower benches, commence filling, and then work up the slope, cutting successive benches ahead of the filling operation. This approach has the dual benefits of allowing the excavation plant to work from the flat surface of the fill that has been placed (rather than having to operate on the sloping face that is to be benched into), and minimising the exposure of the formation ahead of the working front (thus avoiding deterioration in poor weather).

If the geotechnical design of the earthworks requires the removal of soft soils from beneath an embankment then particular attention will be required as to the construction method. In most cases the excavation will be undertaken in dry conditions, with pumping from sumps to draw down the groundwater in the immediate vicinity of the excavation, which is normally sufficient to prevent deterioration of ground conditions. More advanced groundwater lowering systems, such as well-points, may be necessary where excavations extend a significant depth below the groundwater table, or are in difficult

ground conditions (Preene *et al.*, 2000). Alternatively, the excavation can be undertaken beneath water provided that the backfill selected is suitable to be placed underwater (e.g. SHW Class 6A starter layer, see Section 10.6.4).

Particular care should be paid to sites bordering existing properties, because vibration from plant, particularly but not limited to vibrating compactors, could cause cracking. New (1986) reports that human perception of vibration is highly subjective and that some members of the public may be minded to complain as soon as vibrations from an outside (or unusual) source exceed human perception level. The situation is likely to worsen with the duration of the vibration, particularly outside working hours. Photographic surveys of adjoining structures taken prior to the works are recommended in such circumstances, for use as comparisons in case of later claims. Vibration monitoring equipment can be used at sensitive sites to measure the actual ground vibrations caused by construction activities (which will normally show that these are at a relatively low level).

10.3. Fill deposition

Once the site has been prepared, fill material sourced and construction plant is available, the placement of fill can commence. This section describes some of the construction issues to be aware of during deposition of fill material.

Wherever possible, fills of widely different types (e.g. clay and granular soils) should be segregated at the borrow pit or point of excavation, and placed in separate and roughly horizontal layers across the whole width of the compacted area. Mixing suitable clay soils with suitable granular soils does not necessarily lead to a suitable fill (see Section 3.3.2). Random placement of fill types can produce unpredictable settlements at formation level. The final 600 mm or so of fill should be placed and compacted in one continuous operation across the whole fill area.

Steps should be taken to protect construction during adverse weather. Damage occurs not only because of infiltrating rain or uncontrolled groundwater flow but also because of traversing plant. SHW (HA, 1998, 2013) Clause 612.10 (ii) requires that the upper 600 mm of general granular or cohesive fills beneath formation level receives double the compactive effort it would receive at lower levels in order to increase strength and reduce permeability, and hence better to withstand the damaging effects of construction traffic. (However, note the remarks on heavy compaction made in Section 2.5.)

Actions to be taken to avoid softening of fill material during earthworks include:

- not placing any fill until compaction plant is on site to work it
- shaping the surface excavated or filled to falls and sealing the surface with adequate passes of a smooth wheel compaction plant
- excavating grips or drainage ditches to collect run off and/or groundwater and pumping off site if necessary (note the need to trap silt before discharging into permanent drainage systems)
- ensuring that surface water does not pond at low points within the earthworks, the use of pumps with automatic trip switches is the most effective approach

- additional compaction of clay fill to increase its strength and imperviousness and hence its resistance to damage from traversing construction plant (but always avoiding overcompaction).

Additional precautions are necessary to protect the top of the earthworks. In the case of highways, SHW (HA, 1998, 2013) requires protection of the finished sub-formation (or formation), on to which pavement foundation capping (or sub-base) is to be placed, by either:

- promptly covering the surface excavated or filled; or if the programme does not permit this
- leave a 300 mm protective thickness of fill material in place above formation level (that will be removed just prior to placing the capping or sub-base). This should be increased in the case of clay soils, because this material is more easily damaged.

It is just as important to protect the work during excavation if the resulting material is to be employed as fill. Failure to do so could mean that otherwise suitable fill is rendered unsuitable due to weather and construction traffic.

10.4. Factors influencing compaction

10.4.1 General

Effective compaction is a function of plant, fill type and drainage conditions. Plant that achieves specification and productivity targets for one fill may not do so for another. Therefore the engineer or earthworks foreman must draw on experience of the most suitable plant for earthworks conditions (giving consideration to the available site investigation and fill testing data). Unfortunately, many decisions are taken on the basis of what plant is immediately available. The engineer should realise that employing such equipment in the long term may be far more expensive than bringing in more suitable equipment, because the overall costs in terms of loss of productivity and effective compaction could be much higher.

As far as equipment is concerned, achieving adequate compaction depends on:

- the compaction plant, its size, weight and operating speed
- fill layer thickness
- fill water content
- number of passes, and
- drainage conditions of the underlying ground.

10.4.1.1 Compaction plant

The depth of penetration of the compactive effort is related to the contact bearing pressure exerted by the compaction plant. In practical terms this means that for a given width of roller increasing the weight of the plant will result in higher compactive effort and deeper penetration into the fill layer. Similarly, the slower the speed at which the equipment is operated the greater the compactive effort that will be imparted (particularly in the case of vibratory rollers). Note that the contact area between a roller and the underlying fill is restricted to a narrow strip foundation, the size of which depends on the mass of the

roller and the consistency of the fill. The contact area becomes exceedingly small for a light roller on a dense fill. Examples of common compaction plant are included in Appendix 2.

10.4.1.2 Layer thickness (and oversize particles)

Thinner layers promote the uniform penetration of compactive effort throughout the compacted fill. If the layer is too thick then the lower part may not be sufficiently compacted. The method compaction requirements set out within SHW define the maximum compacted layer thickness for different compaction plant based on extensive research by TRL (Parsons, 1992).

A disproportionate amount of compactive effort is absorbed manoeuvring cobbles into position within the compacted layer. Cobbles or boulders in excess of two-thirds the layer thickness should not be permitted because this will limit the effectiveness of the compaction process. Comparison of earthworks standards across Europe has shown that this rule is possibly the only earthworks requirement that is the same in every country.

In well-graded fills that include cobbles and boulders (such as Glacial Till) these very large particles will reduce the amount of compactive effort that is applied to the surrounding fill material, resulting in a variable degree of compaction being achieved through the layer. In this situation either the compaction method will need to be modified to utilise larger compaction plant to deliver a greater compactive effort to allow a greater layer thickness, or the oversize particles will need to be removed. When the oversize particles are infrequent it can be possible to remove them as the fill is spread, but experience shows that some oversize material normally remains in the fill. The most effective solution is formal processing of the fill, to separate the oversize by screening and then crushing those coarse particles to create a granular fill.

10.4.1.3 Water content

Achieving the required low air voids content is strongly dependent on the water content of the fill: the higher the water content, the more rapidly the desired low air voids content is achieved; the lower the water content, the more energy will be needed, in terms of plant size and number of passes. There are construction difficulties in adopting too high a water content, particularly with fine grained fill materials, and the discussion in Section 3.4 should be referred to. The water content of a fill is a property that can be modified in order to manage the effectiveness of compaction and the stiffness of the earthwork; the relationship between these properties is discussed in Chapter 8.

10.4.1.4 Number of passes

For any given combination of compaction plant, layer thickness and water content, increasing the number of passes increases the compactive effort transferred to the fill and results in lower air voids. However, if the layer thickness is too large for the type of plant, additional passes will not succeed in compacting the base of the layer and the only result will be to compact the top of the layer to 'refusal', with the basal part of the layer remaining under-compacted. It is important to remember that once the fill material has been compacted to a condition close to zero air voids, then additional passes will not increase the density (examples of which are illustrated in Figure 10.1); and in the case of fine grained fill

materials will result in problems related to over-compaction (see Section 3.4). The method compaction requirements set within the SHW define the minimum number of passes required for different compaction plant based on extensive research by TRL (Parsons, 1992). It should be recognised that the SHW applies to typical UK practice where earthworks operations are restricted during the winter, thus minimising cold weather working and the options for use of frozen fill (see Figure 6.1). If earthworks are to be undertaken using fill at temperatures below about 5°C, then a greater compactive effort is likely to be necessary to achieve the required dry density (Andersland and Ladanyi, 2003), and the method of working will need to be adjusted to suit the operational conditions (especially for fills with a significant fines content).

10.4.1.5 Subgrade drainage conditions

The most favourable conditions are the presence of free draining subgrade soils because these allow for good drainage and the release of excess porewater pressures generated by the compaction process. By contrast, the localised presence of a confined layer or lenses of granular soil at shallow depth below the working platform can lead to localised liquefaction under construction traffic and compaction plant loading (a problem that can be common in some Glacial Tills).

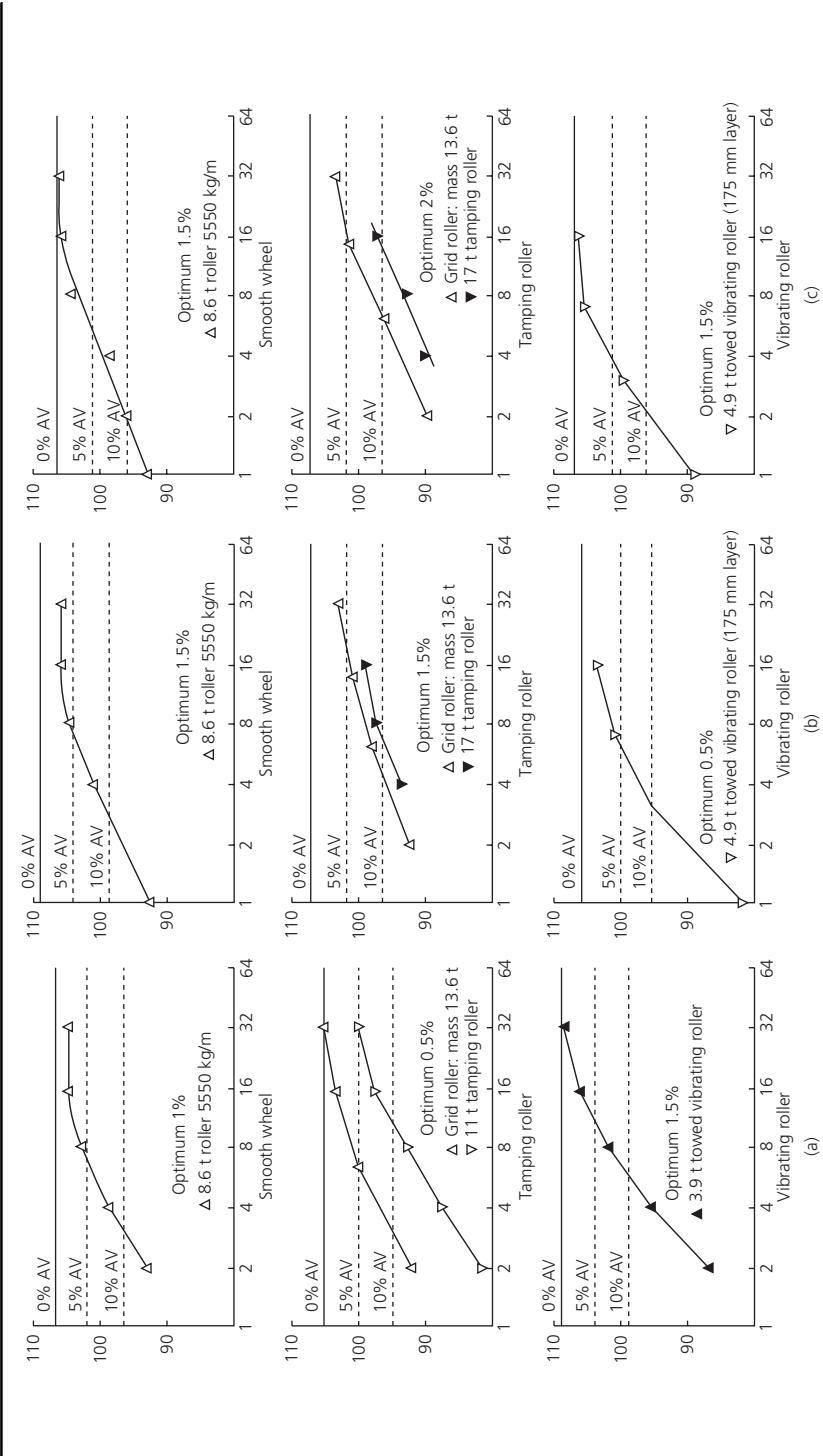
10.4.2 Influence of fill type on choice of compaction plant

Fill type has a significant influence on the effectiveness of different types of compaction plant (see Appendix 2 for details of common plant). This can be seen by comparing the various ‘compaction methods’ set out within SHW: the class of fill determines which method can be used (HA, 1998, 2013 Table 6/1), and for each method the permitted compaction plant options and compaction requirements are listed (HA, 1998, 2013, Table 6/4). The full table is extensive; therefore, it is easier to illustrate the difference between plant types by assuming just one choice of layer thickness, as then the variations in the number of passes required become clear. Table 10.1 has been prepared on this basis for bulk earthworks plant assuming a compacted layer thickness of 250 mm (selected to enable comparison of all methods including those for coarse fills); it is important to note that this simplified summary is purely intended to give a general illustration. Designers should always use the full set of options set out in SHW Table 6/4 to assess what is most appropriate for a site from the numerous options available.

It is apparent from Table 10.1 that the selection of compaction plant for a particular class of fill is not a simple matter. It is important to recognise that some plant is either not acceptable or less efficient for some classes of fill. Consideration should be given to the plant manufacturer’s recommendations and, more importantly, the guidance of an experienced earthworks manager.

A variety of classes of fill materials will be utilised on most sites. Consequently, the earthworks manager’s compaction plant selection will inevitably include some amount of compromise to avoid some plant being left unused for significant amounts of time. Smooth drum vibratory rollers have proved to be the most versatile piece of compaction plant, achieving reasonably good compaction on both cohesive and granular fills, and

Figure 10.1 Performance of compaction plant on different soil types: (a) sandy clay; (b) well-graded sand; (c) clayey, sandy gravel (from Parsons, 1992 with adaptations from Trenter, 1999), reproduced by kind permission from CIRIA



Relationship between relative compaction value (vertical axes) and number of passes (horizontal axes) for three soils and various items of compaction equipment. Relative compaction expressed as percentage of maximum dry density achieved by BS 2.5 kg rammer method (from Parsons, 1992). All layer thicknesses 150 mm unless otherwise stated.

Table 10.1 Typical compaction characteristics of fills used for bulk earthworks construction

SHW compaction method	Fills suitable for suggested method of compaction	Compaction options for compacted layer thickness of 250 mm		Remarks (see Section 10.4.2 for commentary on plant selection)
		Suitable compaction plant	No. of passes	
Method 1	Wet cohesive fills ■ Class 2A, 7C	■ Smooth wheeled roller (or vibrating roller without vibration)	**	** plant requires ≤150 mm layer thickness
		■ Deadweight tamping roller >6000 kg/m width of roll	5	
		■ Pneumatic-tyred roller 4000 kg to 6000 kg mass per wheel	4	
		■ Vibratory tamping roller 4300 kg to 5000 kg/m width of roll	9	
		■ Vibratory roller 4300 kg to 5000 kg/m width of roll	4	
Method 2	Well-graded granular and dry or stony cohesive fills ■ Class 1A, 2B, 2C, 6I, 7D	■ Smooth wheeled roller (or vibrating roller without vibration)	**	** plant requires ≤150 mm layer thickness *** plant requires ≤200 mm layer thickness
		■ Deadweight tamping roller >6000 kg/m width of roll	***	
		■ Vibratory tamping roller >4300 kg/m width of roll	9	
		■ Vibratory roller >4300 kg/m width of roll	4	
Method 3	Uniformly graded granular fills and silty cohesive fills ■ Class 1B, 2D, 6C, 6H, 6J	■ Smooth wheeled roller (or vibrating roller without vibration)	**	** plant requires ≤125 mm layer thickness * SHW stipulates that compaction plant must be towed by track-laying tractors (self-propelled machines are unsuitable because of the tendency to 'dig in')
		■ Deadweight tamping roller 4000 kg to 6000 kg/m width of roll	4	
		■ Vibratory tamping roller 2300 kg to 2900 kg/m width of roll	12*	
		■ Vibratory roller 2300 kg to 2900 kg/m width of roll	10*	

Method 4	Chalk fill ■ Class 3	■ Smooth wheeled roller >5400 kg/m width of roll	4	1. Chalk is sensitive to weight and operation of compacting and spreading plant. Low compactive effort is appropriate 2. SHW notes that all types of vibratory rollers of categories over 1800 kg shall not be used
		■ Grid roller 2700 kg to 5400 kg/m width of roll	4	
		■ Pneumatic-tyred roller 1500 kg to 2000 kg mass per wheel	4	
Method 5	Coarse granular fills ■ Class 1C, 6B	■ Vibratory tamping roller >2300 kg/m width of roll	5	SHW quotes layer thicknesses of ≥400 mm due to coarse nature of the fill
		■ Vibratory roller >2300 kg/m width of roll	5	
Method 6	Granular and stabilised granular capping fills ■ Class 6F1, 6F2, 6F3, 6F4, 6F5, 9A, 9F	■ Smooth wheeled roller (or vibrating roller without vibration)	**	** plant requires ≤150 mm layer thickness Number of passes varies depending on mass of plant used, with heavier plant requiring fewer passes
		■ Deadweight tamping roller >6000 kg/m width of roll	20	
		■ Vibratory tamping roller 4300 kg to >5000 kg/m width of roll	10–12	
		■ Vibratory roller 1800kg to >5000kg/m width of roll	6–12	
Method 7	Stabilised cohesive capping fills ■ Class 9B, 9D, 9E	■ Smooth wheeled roller (or vibrating roller without vibration)	**	** plant requires ≤150 mm layer thickness Number of passes varies depending on mass of plant used, with heavier plant requiring fewer passes
		■ Deadweight tamping roller 4000 kg to >6000 kg/m width of roll	6–8	
		■ Pneumatic-tyred roller 4000 kg to >12 000 kg mass per wheel	4–16	
		■ Vibratory tamping roller 3600 kg to >5000 kg/m width of roll	12–16	
		■ Vibratory roller >5000 kg/m width of roll	12	

Note: This table is based on information provided in Table 6/4 of the SHW, summarising the methods for a compacted layer thickness of 250 mm to enable comparison of different types of compaction plant for bulk earthworks for the different fill material types. Plant options in italics have been included as alternatives that would be commonly employed but require <250 mm layer thickness

consequently are the most commonly seen plant on earthworks projects. In contrast, pneumatic-tired rollers have very specific uses, and are rarely seen. Tamping rollers (often inappropriately referred to as sheepfoot rollers) are particularly effective on cohesive fill materials, but unfortunately are often not brought to a site because tamping rollers are not an ideal option on clean granular fills. A summary of the different types of compaction plant is provided in Appendix 2. Another perspective on this topic is presented by Dumelow (2012).

An important observation from the TRL research is that towed compaction plant is advisable on uniform sands, because self-propelled compaction plant can often dig itself into the loosely placed sand (Parsons, 1992).

When dealing with relatively wet fill there is a tendency for modern heavy plant to overcompact the fill resulting in the problems of matting (described in Chapters 3 and 8). In this situation it is important to realise that vibratory rollers can be operated without the vibration, and used as a deadweight roller. This is recognised in the first and third rows of SHW Table 6/4. In these cases the layer thickness is reduced and the number of passes increased (as indicated in Table 10.1), but the result will often be far more effective than attempting to use the vibration on a thicker layer of fill.

Each earthworks practitioner will have his or her own personal preference of plant type for different situations. Neil Trenter summarised his experience of the effectiveness of different compaction plant on different fill types, in the first edition of this book, as follows.

- Granular Glacial Tills and other coarser granular soils are normally best compacted using smooth wheel or grid rollers to provide the high contact stresses necessary to break down and/or to manoeuvre the larger fragments into a denser packing. For coarse gravels heavy towed equipment should always be employed. A vibratory component almost invariably produces better compaction with granular soils.
- Uniform fine sands are probably best compacted using a single vibrating roller, preferably watering the fill as required. These fills usually need some confinement and it may be found that the current layer may not be achieving specification, although the previous (underlying) layer does.
- Smooth wheel and grid rollers may also be considered for compacting some industrial fills and demolition wastes, and vibrating rollers have been found to perform well with burnt and unburnt colliery discard (Chapter 6). Weaker components in demolition rubble (bricks, mortar, plaster and weaker concrete fragments) may crush into fine particles or powder under the high contact pressures from this equipment.
- Smooth wheel compaction plant is particularly useful for sealing and shaping fill surfaces to prevent infiltration between lifts, but scarification is frequently necessary to help bond the next layers, especially where more plastic clay fill is employed.
- For clays and finer grained soils, the sheepfoot roller is effective. Because of its action, lumps and chunks are broken up and the top of the previous compacted layer is left rough, offering good bonding with the fill of the current layer. It is

reportedly successful in the USA with clay soils used for landfill liners. For maximum effect, the layer thickness should be about equal to the length of the feet, plus 50 mm, according to Privett *et al.* (1996). Based on work in the USA, these authors recommend long thin feet to assist remoulding soil clods and to bond with the previously compacted layer beneath.

- The number of passes of a sheepsfoot roller is related to the foot area covering the drum. If the foot area is 20% of the drum area, then a minimum of five passes would be necessary. In practice, the sheepsfoot begins 'walking out' when maximum compaction has been reached. In the UK sheepsfoot rollers developed into the tamping roller with relatively larger feet, because the contact stresses are lower, which means that it can better operate in the wetter conditions frequently encountered, and to also suit granular fills. One version of this equipment has a dozer blade, which makes for versatility.
- Hard clays, shales, mudstones, siltstones, sandstones and other weak rocks may benefit from some form of heavy vibratory roller. With these materials, vibratory techniques often lead to better performance in dry density terms than do static techniques, although vibratory compactors can dig themselves in if the initial layer thickness is too large. Tamping rollers are also useful in such fills.
- Heavy towed vibrating rollers usually operate at lower frequencies than do the lighter variants, adding an element of impact loading, which is reportedly advantageous for cohesive fill (Parsons, 1987). The effect of vibratory equipment on adjoining structures should be remembered (see Section 10.2).
- Pneumatic-tyred rollers have the advantage of adjustment, not only to weight by adding ballast, but also to tyre width by varying tyre pressures. It is possible to adjust in two ways: (a) increase the tyre pressure at constant wheel load, which reduces the contact area at the surface and increases contact pressure, or (b) increase the wheel load at constant tyre pressure, which increases the contact area at the layer surface. These properties make the pneumatic roller a good piece of plant to experiment with difficult materials.
- Sands, clayey sands and sandy clays are probably the fill types most suited to compaction by pneumatic-tyred equipment. It is frequently employed with pulverised fuel ash and its possible use with some other types of industrial fill has already been noted.

Parsons (1992) gives information on the characteristics of several soils compacted by various pieces of equipment. In Figure 10.1, the performance is illustrated for three soil types typical of British conditions, 'sandy clay', 'well-graded sand' and 'clayey, sandy gravel'. Direct comparison is difficult because of the difference between layer thickness and water content and because different sizes of plant were frequently used. Nevertheless, good performance of the vibrating equipment is evident, even though it was substantially lighter than the other plant investigated. Satisfactory performance is observed for the smooth wheel roller, which has always been popular in Britain.

It should also be noted that compaction of fill material can also be achieved by plant that delivers densification of the soil to significant depth, such as a few metres below the ground surface. This is often considered as a form of ground improvement rather than

conventional earthworks and so will not be covered here in any detail. There are two proprietary methods that are relatively commonly encountered.

- The rapid impact compactor, which achieves densification of the ground in a similar way to the dynamic compaction method of ground improvement, but on a smaller scale. This does not appear at all like earthworks, but it is relevant to note that the research as early as 1954 at the TRL did include a number of options for compaction of soil by dropping-weight compactors, and by pneumatic impact compactors (Parsons, 1992).
- Large non-circular towed rollers that deliver compaction by impact of the ground as a result of their large mass and non-circular shape, in a process known as ‘high energy impact compaction’. Various shapes of roller have been considered, but modern impact rollers are truncated triangular or pentagonal in cross-section (refer to Appendix 2). These work in a combination of vibration and a dynamic ‘cam’ effect as the roller rotates.

Both of these methods of improving the ground can be very useful when the site conditions and project requirements are appropriate. However, it is important to remember that compaction of thick layers of soil will always carry the risk of some proportion of the material not being improved to the required standard. The normal earthworks approach is to place the fill in thin layers and compact each layer fully.

10.5. Fill conditioning

10.5.1 General

When the fill has too low or too high a water content, either because of the weather during excavation or because the fill was excavated from below the groundwater table, it may need wetting or drying before or during placement. The same applies to some clay or weak rock fills, excavated in chunks and that cannot be properly compacted. This process is frequently referred to as fill ‘conditioning’ and may take one of a number of forms described in the following sections.

10.5.2 Treatment of fine soils with lime

Soils can be treated by the addition of a ‘binder’, such as lime, cement, PFA or granulated blast furnace slag. This is normally achieved in situ by pulverising the soil, adding the binder, allowing a mellowing period if appropriate and then compacting the soil. In the case of lime the treatment has the effect of conditioning the fill and so is described briefly in this section.

Treatment with lime is usually applied to cohesive soils, but occasionally it is used to reduce the water content of granular soils. Quicklime (CaO) is a common conditioning medium for wetter fill, but hydrated (slaked) lime may be used for drier material. Special measures are necessary for investigating the suitability of the fill to be treated. The following should be identified if treatment with lime is being considered:

- the presence of sulfate and sulfide-bearing materials
- the presence of soils of adequate plasticity (minimum plasticity index 10%)

- the presence of any organic material
- groundwater and its chemical content.

Treatment of soil with lime can be achieved at two levels:

- 'lime modification', where a limited amount of lime is added to the soil to modify the water content to aid compaction (e.g. $<2\%$ lime by weight), and
- 'lime stabilisation', where sufficient lime is added to change the soil's mechanical properties permanently (normally $>2.5\%$ lime by weight).

Lime modification is achieved by mixing the soil with a limited quantity of binder, which improves short-term performance primarily by a reduction in water content. There may also be an associated enhancement of bearing capacity and reduction in plasticity. Once the binder has been fully mixed with the soil a short 'mellowing period' is allowed to give time for the modification process to take place; this can be from a few minutes up to 2 hours depending on the efficiency of the processing plant used. The benefits of this modification process are that it enables the fill to be

- handled by conventional earthmoving equipment
- satisfactorily compacted in layers
- trafficked and provide a temporary working platform for the superimposed fill layer.

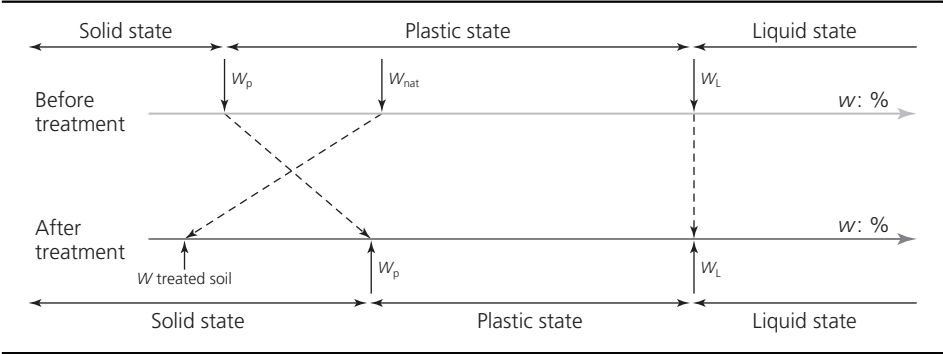
Modification of fill with lime is essentially a temporary change that can be largely reversed by the replacement of the water that is lost from the soil during the treatment. However, the benefits of the modification process can be locked in by avoiding further saturation of the fill during mellowing, and compacting the modified fill immediately after mellowing to achieve less than 10% air voids content.

The process for lime stabilisation is similar to that described above for lime modification, but the amount of binder added is sufficient to achieve the additional benefit of a long-term enhancement of the fill's strength/stiffness (an upper limit for design purposes of 15% CBR is often assumed). In this case the reaction of the lime is sufficient to achieve cementitious bonds between the soil particles, resulting in higher strength and stiffness. For lime stabilisation a mellowing period of at least 24 hours is required.

Rogers *et al.* (1997) state that, in addition to an immediate drying of wet clay soil (with accompanying strength increase), the following changes occur:

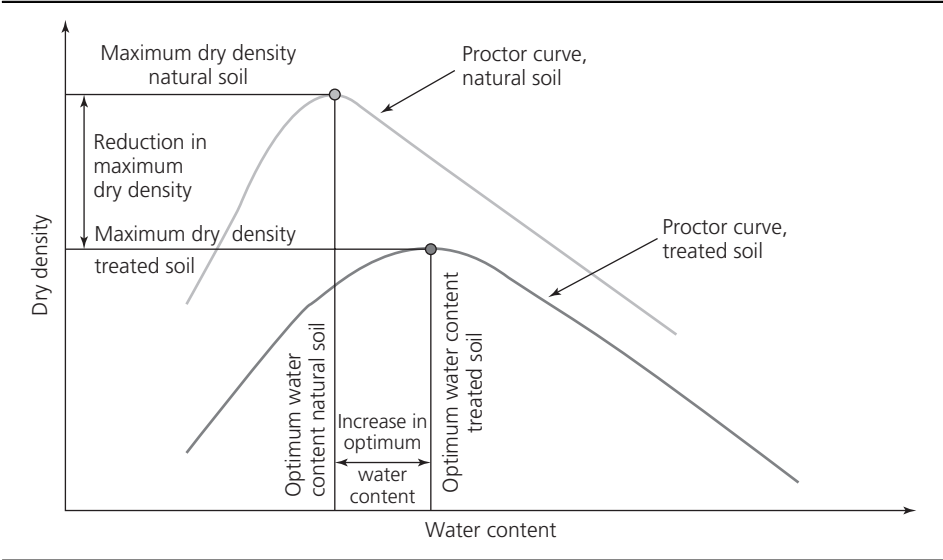
- reduction in clay mineral electrical double layer thickness (the adsorbed water layer)
- flocculation of clay particles
- increased shear resistance between particle agglomerates and greater strength of the agglomerates themselves
- reduction in plasticity index due to an increase in plastic limit (without a significant change in liquid limit).

Figure 10.2 Short-term changes to natural water content, plastic limit and liquid limit through lime modification



The processes of lime modification and lime stabilisation rely on the fundamental effect of lime on the soil's plasticity that is illustrated at Figure 10.2. One of the main binders used for the modification and stabilisation of cohesive clay soils is quicklime. As well as the change in plasticity, there is also an increase in the shear strength of the soil and a modification of the compaction characteristics. As shown in Figures 10.2 and 10.3 the optimum water content for a lime-modified soil lies below and to the right of the corresponding curve for the natural soil. This is always found regardless of the compactive effort used for the test (although as noted above achieving a good standard of compaction is particularly important for fills treated with lime). This increase in optimum water content (OWC) when lime is added to the clay soil is a special feature of the treatment process, because it is the opposite of what is found with natural clays when low

Figure 10.3 Effects of lime on fill materials compaction properties



plasticity clays tend to have lower OWC than high plasticity clays. This change in the OWC and shape of the compaction curve are very beneficial to the ease of handling of a clay fill during earthworks. Lime stabilisation lowers the maximum dry density of the soil and raises the water content at which this density can be achieved, as shown in Figure 10.3.

If sufficient lime is added to increase the pH of the soil to greater than 12 then over time a pozzolanic effect is seen in which a long-term strengthening of the soil is achieved. This is from the growth of a crystal structure similar to that found in a concrete, albeit at a much lower strength. The reaction is between the lime and the clay minerals, silica, alumina and iron in the soil. There is, however, a limit to the strength that lime-only stabilisation can achieve. Results can vary greatly depending on the exact soil type, which is why prior testing is important for soil stabilisation. The following are typical results that could be anticipated for lime stabilised clay fills.

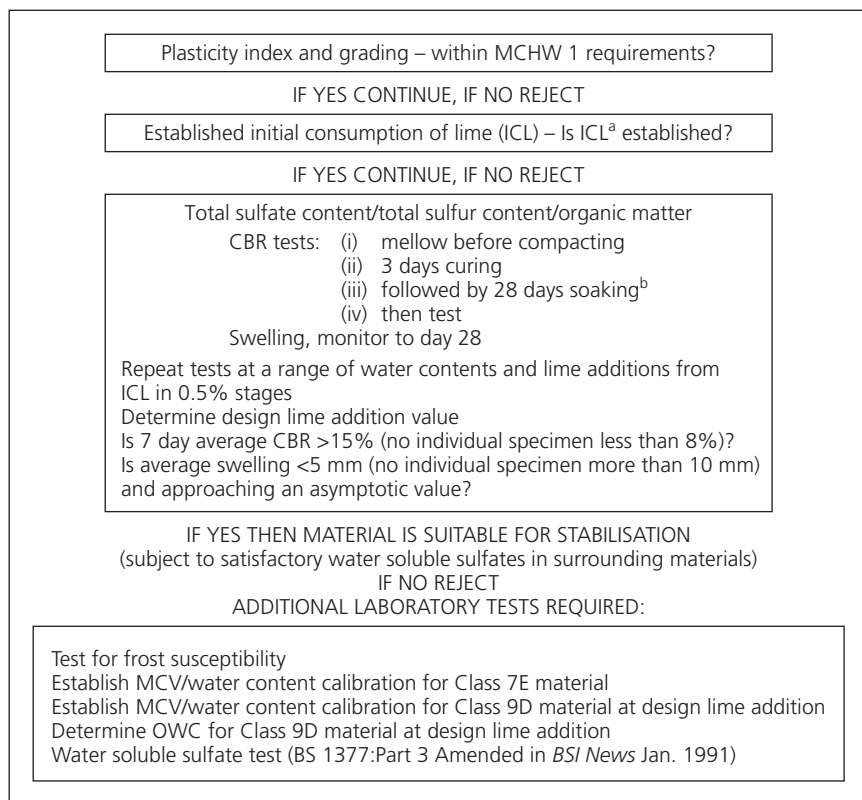
- $\text{CBR} > 15\%$
- $c_u > 150 \text{ kN/m}^2$
- $\phi' > 25^\circ$
- $c' > 10 \text{ kN/m}^2$ (c' from test results should be treated with caution)

If greater strengths with longer term durability are required then a second binder is used in addition to the lime, such as cement, ground granulated blast furnace slag (GGBS) or PFA. These are generally used either as lime and cement, or a combination of lime and other binders. These binders form cementitious bonds within the soil and offer significant improvement to strength properties depending on the soil types, and improved performance and stability, particularly with respect to the action of water and frost. A typical range of figures is not presented for these binders as they are very variable based on soil types: for example, a 1 N/mm^2 compressive strength could be achieved in a cohesive sample, whereas this could be 5 N/mm^2 in a granular soil for the same percentage of binder added.

The process of lime modification tends to work well with most British soils, but failure to achieve the desired stabilisation reaction has been reported with some, perhaps because of their particular clay mineral content or excessive organic content.

A major restriction on the treatment of soils with lime for use in earthworks is the presence of high levels of sulfate-bearing minerals, because these can result in an adverse reaction occurring post-compaction of fill that contains high concentrations of lime, resulting in heave. This is not normally a problem for lime modification because all of the lime is fully consumed as part of the modification process, but it can be a major problem for lime stabilisation as excess lime is available to react with the sulfates. Some research shows that this adverse reaction can be managed in some high sulfate clays by using a combination of lime and PFA or GGBS as the binder. Therefore, the ground investigation must assess the chemical properties of the natural soil, fill materials and groundwater. In addition, laboratory trials should be undertaken with the proposed fill and appropriate proportions of binder to check for adverse reactions.

Figure 10.4 Flow chart for assessing soil suitability for lime stabilisation purposes (from Perry *et al.*, 1995)



^a Initial consumption of lime: BS 1924: 1990: Part 2 (BSI, 1990a)

^b The soaking period was increased from 4 to 28 days in HA 74/07 (HA, 2007)

As pointed out in Section 3.5.2, the concentration of sulfate- and sulfide-bearing materials is likely to be least at shallow depth; furthermore, the most highly weathered (and hence the most plastic) soils also usually occur at shallow depth. Consequently, fill from the top 2 or 3 m of the cut excavation is often the most suitable for lime stabilisation purposes. Should this material prove inadequate, investigation should focus on zones elsewhere within the cut depth or on other borrow pits that demonstrate adequate plasticity, low organic matter and the least sulfur-bearing material content. Perry *et al.* (1995) recommend the procedure given in Figure 10.4 for assessing the suitability of soils for lime treatment purposes. Information on site investigations for lime stabilisation is also given in the Highways Agency, Advice Note HA 74/07 (HA, 2007), where information on chemical testing procedures is also given.

The requirements for chemical testing set out within HA 74 were enhanced between the version HA 74/95 (HA, 1995) and HA 74/07; although this greatly reduces the risk of

problems it has increased the testing costs associated with stabilisation of soils. However, in many circumstances soil modification or stabilisation remains an economic alternative to disposal of unacceptable soils and replacement with imported fill.

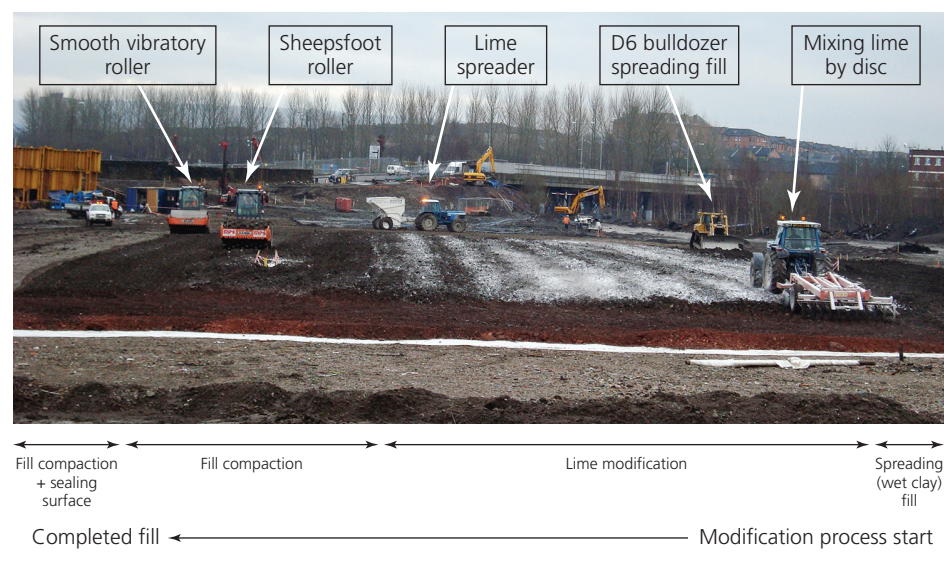
Softening following wetting has been recorded at some sites and may be due to poor compaction or patches of untreated or only partly lime treated fill in among the completed fill. Good compaction and drainage, both during construction and in-service, appear essential. Perry *et al.* (1995) also report that well compacted lime stabilised soils with an air voids content less than 5% should help guard against sulfate-induced heave.

Additional observations relating to soil modification and stabilisation with binders are provided as follows.

- Much of the industry focuses on treatment with lime, which to some extent is the case in this short summary because the focus here is on ‘fill conditioning’ for general earthworks. However, the earthworks engineer should consider the potential benefits of alternative binders. For example, in many low to medium plasticity soils such as the glacial soils that cover large areas of the UK, a two-shot treatment process can be far more effective than simply treating the soil with increasing quantities of lime. The first pass modifies/conditions the soil with lime, and then the second pass stabilises the fill with cement. This two-shot process greatly reduces the risks associated with high sulfates, and of reversal of the lime improvement on further wetting.
- The nature of the plant used for the treatment of soil with binders has a significant influence on the successful outcome, with the best results being achieved using purpose-built soil stabilisation plant (see Appendix 2). This specialist plant processes the soil to a fine tilth and spreads the binder evenly, both of which allow optimum mixing of the binder. The plant can also add water in a controlled manner. All of this enables the best quality of fill compaction and a very consistent product. This plant, however, cannot be used in soils that contain cobbles or a high coarse gravel content. Other plant can be used effectively in the right circumstances, but the fill will be less consistent, and the results poorer.
- Lime modification can be successfully achieved using agricultural plant or general earthworks plant (rather than specialist stabilisation plant). An example of such an earthworks site undertaking the modification process is illustrated in Figure 10.5. At this site ancient Clyde Alluvium of low organic content soft to firm clay was suitable for use as a fill material in all regards except for a high water content, and hence low undrained shear strength (MCV of 5 to 6). The soil was improved by lime modification, using 1% lime, to create a fill acceptable for highway embankment construction (consistently achieving an MCV in the range 9 to 11). The modification process enabled the material to be retained on a site in an inner city location that had limited opportunities for landscape earthworks.

For further guidance on the undertaking of soil stabilisation the soil stabilisation section of the Britpave website is a good reference as there are individual reports on many aspects of the process (<http://www.britpave.org.uk>).

Figure 10.5 Lime modification process to reduce water content of cohesive fill using agricultural plant for highway embankment construction



10.5.3 Mechanical conditioning

As noted above, over-consolidated clay and some weak rocks such as shale and mudstone can be difficult to compact. These fills tend to be at water contents significantly below their plastic limit, consequently there is a high risk of post-compaction changes occurring (see Section 8.3.4). With these materials it is very difficult to achieve uniform compaction throughout the full thickness of the fill. When compaction to high standards is necessary, a mechanical conditioning method (in practice probably a rotovator), which both breaks down the chunks and can add water should be considered. The disadvantage of this technique, apart from the expense of having more equipment on the site, is that the filling process is geared to the rate at which the equipment can deliver the conditioned fill. More than one processor on site may be necessary.

For major earthworks such as embankment dams, the alternatives reported for conditioning large volumes of over-consolidated clays (Kennard *et al.*, 1978) include:

- hosing directly onto the clay during spreading, including hosing directly on to the bulldozer blade
- hosing into scrapers as they discharge
- piped spray irrigation systems
- mixing the clay with water by sluicing at the borrow pit
- spraying from a specially fabricated water tank mounted on a converted motor scraper, combined with disc harrowing to reduce material to specified size.

The last of the options explored by Kennard *et al.* appeared to have been most successful. However, all of the techniques struggled to achieve uniform wetting throughout the fill.

Often a layer only a few millimetres thick is thoroughly wetted at the surface of the fill layer or lump of clay, with the rest of the material remaining dry. Vaughan *et al.* (1978) produced evidence showing that even in the long term the water does not redistribute through the fill and both wet and dry zones remain. Therefore, for over-consolidated clays it is necessary to use a method that will reduce the clay down to lumps of as small a size as possible, and then add moisture in a controlled manner on a regular basis during a mellowing period, thus allowing the material to break down and increase in water content towards the plastic limit.

The need to increase the water content of sand and gravel or low plasticity fine fills is relatively common for earthworks undertaken during hot or dry and windy weather conditions, because these fills can lose moisture quickly. These fills are much easier to work than the over-consolidated clays described above, but the need to achieve an even distribution of added moisture throughout the fill material still causes difficulties. The most common approach employed on site is simply to spray water from a bowser onto the surface of each fill layer. However, this is not an efficient method and results in some fill that is far too wet while the majority of the fill is unchanged. A significant improvement can be achieved by the use of either a rotovator or a power harrow to mix the moisture through the full layer of fill. An option worth considering is to utilise modern stabilisation plant as this provides the most efficient form of rotovation, and also delivers a carefully controlled spray of water that is mixed evenly through the soil. Alternatively, the water content of granular fill can also be achieved at a processing area by spraying a stockpile and mixing it with an excavator or loading shovel to create an acceptable fill material prior to transportation to the placement area.

10.5.4 Stockpiling

This approach requires the fill to be placed in conveniently sized heaps, on a granular base, which is itself laid to falls so that the drained off water can be removed. It is limited to free-draining materials, such as granular soils. Consideration must be given to whether the grain size is such as to allow free drainage: a small amount of fines in an otherwise sandy soil could drastically lengthen the time necessary for drying, and the process might not be viable as a consequence.

10.5.5 Aeration

With this technique, the fill is spread in thin layers over as wide an area as possible and dried to the desired placement water contents using discing or harrowing. Charles (1978) states that the change in water content required to produce a given change in undrained shear strength is a linear function of fill plasticity index and quotes the expression

$$\Delta \text{ water content} = \frac{PI \times \Delta c_u}{5.8 \times c_u} \quad (10.1)$$

where PI denotes the plasticity index and c_u the undrained shear strength. The expression is reportedly valid for the undrained strength range $20 < c_u < 100 \text{ kN/m}^2$.

As with stockpiling, aeration means that the fill must be placed in the open where, in the British climate, it can just as well receive rainfall as sunshine. The technique is

consequently best suited to those locations having plenty of space together with a recognisably dry season, otherwise the whole filling process becomes more than usually weather dependent.

10.5.6 Consolidation by drainage

This technique is most often employed for major projects such as embankment dams where a clay fill is particularly wet, and significant pore pressures could be set up during construction. Its use in large road embankments has also been reported. Bishop and Vaughan (1962) showed that the pore pressure parameter r_u was highly sensitive to water content varying from about zero at OWC – 1% to 0.6 at OWC + 1% for the particular sandy clay fill studied. They report that a side slope of about 1 (vertical) in 5 (horizontal) would have been necessary for a factor of safety of 1.5, had horizontal drains not been adopted.

Horizontal drainage layers are laid at vertical intervals, which depend on the vertical coefficient of consolidation c_v of the compacted fill and hence control the rate at which pore pressures dissipate. The drainage layers are themselves laid to falls sufficient to drain the water from the consolidating fill. The disadvantage of this method is that the drainage layers require material meeting the usual filter rules, if clogging is to be avoided, which may be expensive and may not always be easily available.

Gibson and Shefford (1968) investigated the efficiency of horizontal drainage blankets with respect to surrounding clay fill. They found that a drainage blanket only 10^3 times more permeable than the fill was 'likely to be useless'. An acceptable efficiency was achieved by using a drainage blanket 3×10^4 times more permeable than the fill. This general approach is used fairly widely in Sweden, but it is not widely used in Europe.

Grace and Green (1978) provide a graph showing the relationship between permeability of fill and distance between drainage layers for 90% consolidation.

10.6. Special problems

10.6.1 Load bearing fill

The topic of building structures on fill material is one that was considered in detail by the Building Research Establishment, and their work should be referred to when undertaking such projects (Charles and Watts, 2001). There can be unacceptable differential settlements if the nature of the fill and its thickness vary substantially. Rogers (1992) recommends that variations in fill depth under a building should not exceed 15% of the average fill thickness, in order to reduce the possibility of deleterious differential settlement.

Feather edges (one part of the structure resting on natural ground and the other on fill) should be avoided. Where possible, the site should be configured such that structures are built either on natural ground or on fill of generally uniform thickness. In practice, some stepping of the natural ground may be necessary (Figure 10.6). As far as practicable, vertical step height should be small in comparison to step length to minimise vertical discontinuities. Checks for settlement and differential settlement in relation to applied loading from structures should be made.

Particular care is needed where structures are to be placed over or near the high wall of a former quarry or open-cast mine. This problem was investigated by Skinner and Charles (1999) who distinguished between:

- quarries backfilled to the top of the former quarry walls, and
- quarries completely buried by backfilling operations.

For the first case, they found that with very steep-angled high walls (usually the situation with former rock quarries or open-cast mines), even small fill settlements produced potentially damaging ground movements at the fill surface where it adjoined the high wall. For the second case, the geometry of the former quarry high wall was less important than the ratio of the depth of superimposed fill to depth of the former quarry; the larger this ratio became, the smaller the settlement, although its effects occurred over a much wider area.

For full details, Skinner and Charles (1999) should be consulted but it is clear that high walls can threaten the stability of structures built on fill. Figure 10.7 offers suggestions as to how treatment may proceed.

10.6.2 Fill at multi-purpose sites

A multi-purpose filled site could comprise load bearing and non-load bearing (landscaped) areas. Different levels of compaction may be selected for each purpose and, if available, different fills. Should it be decided that a lower level of compaction is warranted for the landscaped area, a transition zone around the load bearing area should

Figure 10.6 Stepping natural ground to achieve more uniform foundation conditions: (a) part of foundation on natural ground, part on fill; (b) foundation placed entirely on fill

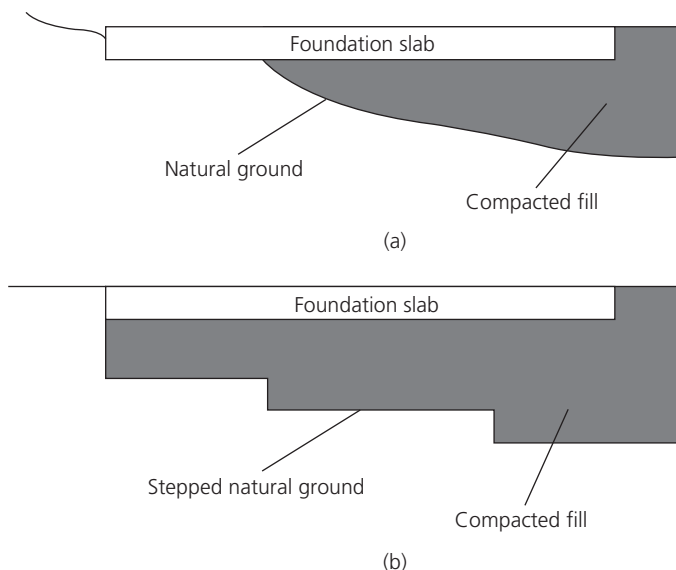
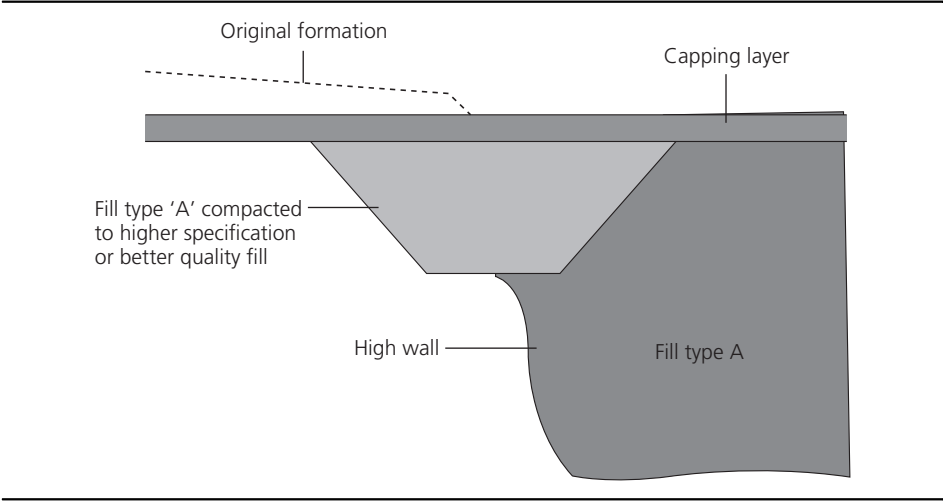


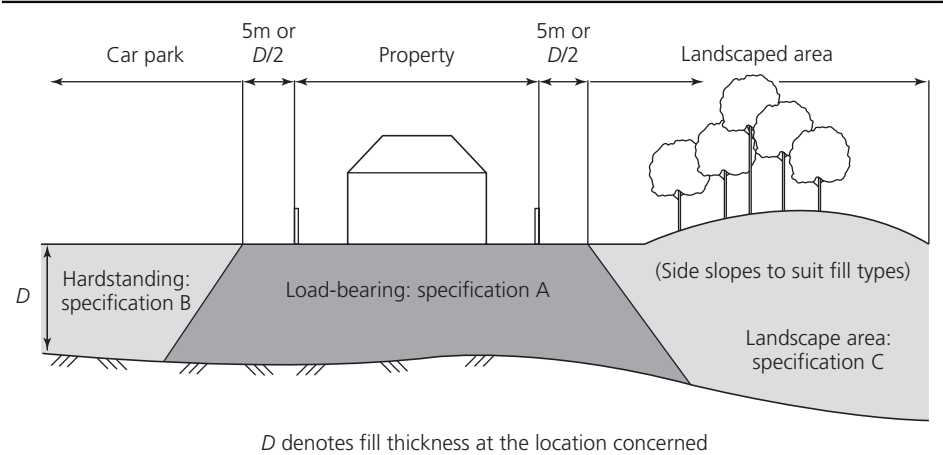
Figure 10.7 Nature of measures required to reduce settlement adjoining a quarry or open-cast high wall



be provided. In Figure 10.8, a transition zone width of half the fill thickness or 5 m, whichever is the greater, is suggested for preliminary purposes. The final dimensions to be adopted will depend on the fill types and the degrees of compaction selected.

There may be large boulders at some sites that could be impossible to incorporate into compacted fill in the usual way. Their removal off site could be expensive. Assuming that the fill were deep enough in relation to the boulder sizes, a practical way of coping would be to place them at the bottom of the excavation, sufficiently far enough apart to enable placement and compaction of fill to proceed around them.

Figure 10.8 Typical compaction scheme beneath mixed-use area



The discussion on hierarchy of fills (see Section 8.3.1) should be referred to for further details.

10.6.3 Stresses from compacted fill on retaining walls

Solutions are available for estimating stresses acting on the back of retaining walls due to compacted fill. Ingold (1979) presents a solution for free draining (granular) soils and Clayton *et al.* (1991) for clay soils.

10.6.3.1 Granular fill

Assuming that the granular fill is placed and compacted in thin layers behind the retaining wall, the horizontal pressures will vary, from residual passive to active. Starting at the base (Figure 10.9(a)), the stress acting will be the residual passive, which can be shown to be (Ingold, 1979)

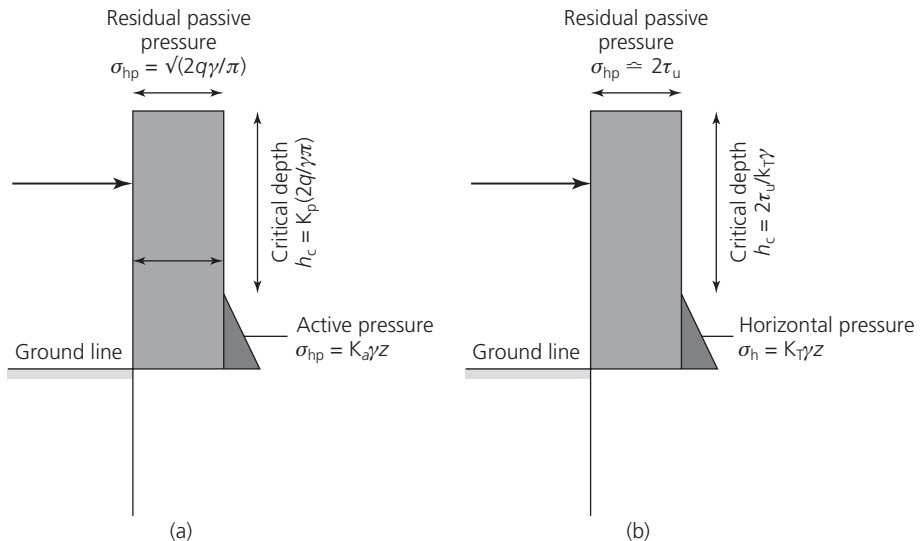
$$\sigma_{hp} = \sqrt{(2q\gamma/\pi)} \quad (10.2)$$

where q is the line load due to the compaction equipment and γ is the unit weight of the fill. (Note that the fill's angle of shearing resistance ϕ is not involved.) Given uniform fill thickness and line load, σ_{hp} will be constant.

However, as the compacted fill ascends higher and higher behind the wall, there comes a depth h_c below the top of the wall where the active pressure σ_{ha} will begin to exceed the residual passive pressure σ_{hp} , where

$$h_c = K_p \sqrt{(2q/\pi\gamma)} \quad (10.3)$$

Figure 10.9 Stresses from compacted fill on the back of retaining walls: (a) granular fill; (b) clay fill



and

$$\sigma_{ha} = K_a \gamma z \quad (10.4)$$

The terms K_a and K_p are the active and passive earth pressure coefficients, respectively, and z is the distance from the top of the wall. Note that in Figure 10.9(a) at the very top of the wall, passive conditions will not develop. As this distance is small, it may be neglected for most practical purposes.

The above described approach is highly idealised. It assumes that all the movements necessary to assure active and passive conditions will occur, which, depending on wall design and construction, may not be the case. Moreover, it assumes that the residual passive stress is controlled by the line load q and not by the compactive effort; in other words, the residual passive pressure would be developed whether one or any number of passes of the roller were in contention.

10.6.3.2 Clay fill

Stresses behind the back of a retaining wall are more difficult to estimate for clay soils because of the possibility of swelling (see below). A solution, illustrated in Figure 10.9(b), is similar in principle to that for granular soils. Again, starting at the base, the residual passive pressure $\sigma_{hp} \approx 2\tau_u$ exists behind the wall until, at a critical depth h_c below the top of the wall, the horizontal stress σ_{ha} begins to exceed the residual passive pressure σ_{hp} , where

$$h_c = 2\tau_u / K_T \gamma \quad (10.5)$$

In this expression, τ_u is the undrained shear strength of the fill, K_T is the ratio of total stresses (σ_h/σ_v) and γ is the fill unit weight. Below h_c , the horizontal stress becomes

$$\sigma_{ha} = K_T \gamma z \quad (10.6)$$

The problem in adopting this solution lies in estimating K_T . As noted, clay fill may swell, particularly if formed from over-consolidated clay excavated from depth. Therefore K_T at the end of construction may be much smaller than K_T in service. Furthermore, the size of K_T will itself depend on wall construction, particularly whether or not movement is permitted, which would reduce the stresses on the back of the wall. Estimation of τ_u also presents problems, if formed from over-consolidated clay, because of the difficulties in compacting to produce a homogeneous whole. In practice, the best approach for such soils, and one likely to produce reasonably consistent results, is to employ a mechanical rotovator (see Section 10.5.3) to reduce the clay to chunks of relatively small size (much smaller than layer thickness) and of reasonably uniform water content.

10.6.4 Hydraulic fill and placing fill in water

This book has been prepared for land-based earthworks projects, where fill is placed in dry conditions with fill compacted in layers. However, earthworks can also be constructed by placement of fill into water, with the fill material and depositional process being selected and managed to achieve a desired outcome. In some cases the methods can

be described as creating ‘hydraulic fill’, but this term is often misused. This section provides some general comments on hydraulic filling, which is a very different mode of earthwork construction, and brief comments on other methods of filling into water.

The *Dictionary of Civil Engineering* (Scott, 1987) defines hydraulic fill as ‘embankment material carried by water in flumes (open channels) or pipelines’. This provides a useful point of reference to differentiate the various methods that are commonly used for placing fill into water, which can be summarised as follows.

- Hydraulic fill methods include:
 - dredging by true hydraulic fill methods
 - placement of fine grained mine waste by pumping, and
 - placement of sand backfill around pipes.
- Placing fill into open water (by non-hydraulic fill methods) include:
 - dredging and placing the fill by bottom dumping out of a barge
 - land reclamation by pushing fill into open water
 - land-based projects where a depression is infilled without dewatering and SHW Class 6A well-graded granular fill is placed directly into the water, and
 - mechanical placement of large rocks onto the bed of a sea, lake or river.

For hydraulic fill the material is placed in a liquid state, as a mixture of soil particles and ‘process’ water, in a deliberate manner so that the soil particles accumulate as a deposit, and densify as further fill is placed above to create a body of fill. When the fill includes a significant fines content (as is normally the case for mine waste) then it is critical that the deposition process results in the body of fill including permeable layers that allow the density of the fill to improve with time by drainage and consolidation. Construction of hydraulic fill is managed by experience of the rate of deposition of the particles allowing for the nature of the particles and the rate of flow of the solution.

There are various dredging methods for achieving placement of the fill, including pumping a hydraulic fill, or dumping the fill from a barge, the latter is not strictly a hydraulic fill but is often described as such. This is clearly very different to land-based earthworks. A successful outcome does rely heavily on the experience of the specialist of undertaking this sort of earthwork. For projects in which the construction involves the creation of an earthwork by dredging, then it is recommended that the engineer refers to the *Hydraulic Fill Manual for Dredging and Reclamation Works* (van’t Hoff and van der Kolff, 2012) as this covers all aspects of this relatively specialist process in a very usable format.

One hazard associated with dumping the fill from a barge is that the mass of fill tends to drop through the water as a large body, hitting the sea bed with significant impact rather than settling out of suspension, and this can result in an uneven crater profile being created with successive impacts.

Large areas of deep hydraulically placed fills, such as dredged sand fill deposited for reclamation purposes, bring their own challenges for design and construction. Hydraulic fills tend to be placed with low relative densities ($30\% < D_r < 50\%$ for bottom dumped

hydraulic fill) and generally need subsequent ground treatment to achieve the required strength and settlement characteristics for the development of the reclamation. Traditionally, dredging practices would focus on using relatively clean sands for land reclamation purposes, but recent environmental concerns limiting the release of fine materials have led to the necessity to retain more of the fines within the reclamation fill material resulting in a greater need for ground treatment.

For dredging projects the selection of the borrow source is a critical activity and requires adequate ground investigation data to ensure that the source of fill will be consistently appropriate for the works, because hydraulic fill is very sensitive to fines content. The fill quality can also be checked in the hopper prior to placement. Selection of the method of placement is also critical to success and requires an experienced contractor. Control tests are usually performed when the fill has been brought up to final level and some form of densification such as vibroflotation is employed to produce the required degree of compaction. Static cone penetration tests (CPTs) made in situ (BS 1377:1990: Part 9: Test 3 (BSI, 1990b)) are a preferred method for control purposes because the technique is rapid and relatively economic; the equipment when properly calibrated and maintained is accurate.

Initially, the relative density D_r is set for the particular engineering requirements of the site and the static cone penetration test is employed to control the value selected. However, there may be difficulty in relating reliably the cone resistance q_c to the relative density D_r . Although correlations exist between q_c and D_r (Lunne *et al.*, 1997; Meigh, 1987), there is significant scatter in the results of the relatively few sand samples tested, and the Engineer has to decide whether the granular fill concerned is of low, medium or high compressibility; an unenviable task unless significant additional testing is performed. The matter is discussed in detail by Lunne *et al.* (1997), including an approach for deciding compressibility. Note the importance of selecting the correct cone sensitivity for the fill type and depths employed.

Other methods of control include the standard penetration test (SPT) and the dynamic cone penetration test (Tests 3.3 and 3.2, respectively, of the above quoted BS 1377:1990 (BSI, 1990b)).

Some land reclamation projects are undertaken by simply pushing dry granular fill into open bodies of water, which can be planned to achieve acceptable results. This approach is relatively common in the Middle East using locally won sand fill and managing any ground improvement required in a similar way to that described above for dredged fill. Historically, land reclamation was often achieved to create port facilities, or to infill old docks, by dumping any available material into open water, including soft alluvial clays; this cannot be planned in the way described above and so is not considered to be an engineered fill.

Finally, it is relevant to comment on coarser grained fills that are mechanically placed into water. These fills include very coarse granular material placed as part of marine engineering projects, such as carefully selected high strength rock fill that is placed for a specific purpose, for example to avoid wave erosion. A more general earthworks case

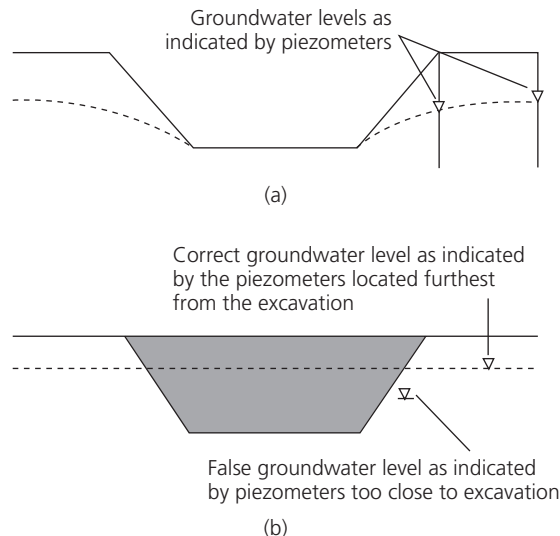
is that included within the SHW as Class 6A, which is selected well-graded granular fill material that can be mechanically placed below water. Only very clean granular fill (with less than 5% fines) is acceptable for this method of placement. The specified grading for Class 6A fill was selected to avoid any requirement for compaction. However, it is important to remember that Class 6A fill would tend only to be used to create the lower layers of a highway embankment. In this situation the overlying mass of the embankment constructed above water would tend to improve the density of the Class 6A fill; as a general guide a minimum of 3 m height of fill above water would be advisable for this purpose.

10.6.5 Earthworks at former quarry and open-cast sites

Former quarry sites or other large open excavations can act as major sinks, affecting the groundwater table for large distances. It can take a number of years after the cessation of dewatering the excavation for the groundwater level to recover fully to a level that is in equilibrium with the regional hydrogeology. Boreholes sunk close to an open excavation when determining the feasibility of backfilling may consequently show a depressed groundwater level. This may lead the engineer to believe that the water table will be at a lower elevation when the excavation is backfilled than in fact will be the case. This is illustrated in Figure 10.10. A false impression of the amount of collapse settlement will be obtained because collapse settlement is proportional to the saturated fill thickness (see Sections 3.5.1 and 4.5.3).

Most open-cast sites are backfilled at the same time as excavation progresses, so the Engineer and the engineering geologist have to investigate groundwater conditions before work starts. This is normally done by sinking boreholes and installing standpipes or

Figure 10.10 Right and wrong ways of estimating groundwater levels in backfilled excavations: (a) former quarry (open-cast) excavation showing position of groundwater table at the time of the site investigation; (b) backfilled excavation showing final groundwater level in fill



piezometers before open casting begins. The groundwater table measured at this stage is usually a guide to what can be expected on completion of excavation and subsequent backfilling, allowing for the normal seasonal influences.

The hydrogeology of open-cast sites can be particularly complicated with flooded worked-out seams day-lighting into the excavation, which may require special pumping measures as excavation proceeds. The worked-out seams should always be blocked off during backfilling but water-tightness cannot be guaranteed; the water level in the fill could rise up to, or higher than, the elevation of the seam. In the past, long-term water levels in the mines were stabilised by deep pumping and, if still in operation, this could affect the water level in neighbouring open-cast sites. Where open-cast workings are planned, it is essential to check whether such pumping is, or has recently been, in operation and, if so, what the long-term plans are for its continuation.

When backfilling a major excavation or open-cast site is under consideration, a thorough hydrogeological study should be undertaken, with expert advice, so that there can be few doubts as to the extent of the groundwater rise and the extent of possible collapse settlement.

The design of the earthworks for infilling major excavations of this nature requires particular consideration of the risk of collapse settlement of the fill on groundwater inundation. It should be noted that the collapse potential of some fills will not be eliminated despite the achievement of a field dry density of 95% of the standard Proctor test. The earthworks specification should be determined to suit this nature of filling, including a requirement for the fill to be compacted to less than 5% air voids if particularly sensitive structures are to be constructed on the fill (BS 6031:2009 Clause 8.4 (BSI, 2009)).

10.6.6 Investigations of solution features

Solution features are a relatively common problem in chalk and limestone areas, and to a lesser extent in Triassic strata that include thick layers of gypsum. The solution features can cause problems for earthworks projects even where there is a cover of younger Tertiary or superficial materials in excess of 10 m or more. Over geological time, acid rainwater and run off finds its way into the body of the soluble rock through fissures and joints, dissolving and enlarging them until, in many cases, major voids are formed. At some stage, the roof of the void collapses into the space beneath, a process that can continue upwards to the ground surface. Most solution features tend to occur where the cover comprises granular soils, rather than clay soils, primarily because of the permeability to rainfall and run off.

The collapsed material that fills the void reflects the nature of the overlying strata, but it is often weak and compressible in comparison with the host rock. The earthworks engineer should be concerned by solution features for the following reasons.

- Voids filled with weak compressible infill may be exposed by cuttings or be present close to the underside of embankments and will have a particularly adverse influence on any structural foundations associated with the earthwork.

- Earthworks change water flow paths, and this can result in an increased rate of dissolution of the soluble rock, triggering collapse in previously marginally stable strata. Design should ensure that any ponds or infiltration zones are located remote from the proposed earthworks.
- Solution feature infill will be less stable than the unweathered solid geology that will have been assumed for the design of the earthworks slopes, which will necessitate local remedial measures.
- Weak compressible infill may appear in the formation, requiring special measures for pavement design, including for example, removal and replacement, or a thick capping layer with geosynthetics to reduce the impact associated with future collapse movement.

Mathews *et al.* (2000) provide a summary of methods for locating solution features, reproduced in part in Table 10.2.

Apart from drilling, where the cores or samples allow a visual record that should be reasonably reliable, the success of the other methods depends in part on the contrast

Table 10.2 Summary of methods for locating solution features (adapted from Mathews *et al.*, 2000)

Method	Dependent property	Type of data	Comments
Direct methods			
■ Drilling	Visual/index properties	Vertical profiling	Expensive and time consuming; numerous boreholes required ^a
■ Static and dynamic cone testing	Penetration resistance	Vertical profiling	Cheaper than drilling but numerous tests necessary
■ Trenching	Visual	Sectioning	Limited depth of investigation
Geophysical methods			
■ Magnetic	Magnetic susceptibility	Lateral variation	Rapid coverage of large areas; has been used successfully ^b
■ Resistivity	Resistivity	Vertical profiling /sectioning/lateral variation	Reasonably rapid coverage; has been used successfully ^b
■ Conductivity	Conductance, inductance	Lateral variation	Rapid coverage; has been used successfully ^b
Remote sensing	Reflected EM radiation	Lateral variation	Cheap; provided rapid coverage but not all solution features have surface expression

^a Drilling can take several forms: e.g. cable percussion; cable percussion with tube sampling (continuous or otherwise); rotary drilling with air or fluid flushes; rotary core drilling also with various flushes

^b Refers to use of the method in locating solution features

afforded between the chalk or limestone and the infill contained within it. Clear distinctions may be drawn between, say, the penetration resistance within or outside a void if the chalk or limestone forming the void surface is hard and unweathered and the weak and compressible infill. If these conditions do not apply, there may be doubts. For this reason, it is best to persevere with several methods, using borehole control wherever economic circumstances permit. Certainly, remote-sensing records should be the first to be examined, along with detailed geological records and the results of a walk-over survey and geomorphological mapping.

Judgement and expertise in allocating the various resources economically and effectively are necessary in locating solution features. For further details, Mathews *et al.* (2000) should be consulted. Information on the locations of dissolution features in the UK is provided by Edmonds and Kirkwood (1990).

10.7. Earthworks validation

Earthworks validation is a process involving a systematic set of actions that are planned and undertaken to provide confidence that the earth structure has been constructed to the Specification requirements and will perform satisfactorily in service.

10.7.1 Quality assurance and quality control

The quality control procedures adopted during earthworks projects are generally considered in terms of quality assurance (QA) and quality control (QC), which can be broadly summarised as follows.

- QA is the testing stipulated within the Specification to demonstrate that the requirements of the Specification are achieved, and to provide the supporting evidence.
- QC testing is additional testing undertaken as the works progress to check that the Specification method of compaction should deliver the required quality of finished earthwork, and enable the method of working to be adjusted when appropriate.

Under the traditional forms of contract, QC testing was commonly instructed by the Engineer during construction on projects. For Design and Build contracts, QC testing should logically be instructed by the Designer and Contractor, but this logic often seems to be ignored and the construction team simply undertakes the QA testing to satisfy the minimum requirements of the Specification. Failure to realise the importance of QC testing tends to demonstrate a lack of earthworks experience because QC testing will often help to avoid delays or problems of poor earthworks being constructed.

The nature of the QA testing will depend on the form of contract. On an SHW method compaction project it will be testing to check the fill is acceptable for inclusion in the works. However, on a performance specification project it could simply be plate load and/or large scale skip load (zone) testing undertaken on the completed earthwork; clearly in this case QC testing will be essential to give any confidence that the end performance criteria will be met.

In view of the tendency for QC testing to be limited, it is more important than ever that the designer carefully considers the testing stipulated within the Specification to ensure that the works progress without problems and that the quality of the completed earthwork is acceptable for the intended end use. For example, it is advisable for the designer to consider the likely borrow pits that will be used and the periodic 'source material' testing that will be appropriate (to avoid unsuitable fill being brought to site and incorporated in the works), and include testing requirements within the Specification.

Whichever approach is taken in terms of the split between QA and QC testing, it is advisable to start the project or a new fill material with a high test frequency in order to understand the particular issues of that site and the fill material. It is advisable to include within the early works an earthworks trial, with appropriate levels of testing, to check that the intended method of working is appropriate. Once a consistent quality of fill is being achieved by the routine earthworks, consideration can be given to reducing the testing frequency.

10.7.2 Testing fill material properties

A fundamental aspect of the earthworks quality assurance process of a project should be checking that the fill to be placed has appropriate material properties. These tests will include properties such as: grading, water content, Atterberg limits, compaction properties, MCV and so on, which are considered as part of the design process described in Chapter 8.

10.7.3 Compacted layer thickness

Although it is critical that the site team ensure that the thickness of each layer of fill is carefully controlled, actually estimating the thickness of a layer of fill during construction is relatively difficult and requires experience. The problems on site include the fact that the fill is pushed out as a working face ahead of the fill being compacted, the front face is not a distinct step and is likely to be either thinner or thicker than the working area being compacted. Furthermore, the most important thickness is actually the 'compacted layer thickness' (not the thickness of the loose fill that is spread out as a layer). The compacted layer thickness can be checked periodically by surveying the working surface during the earthworks (i.e. at fill sampling and in situ testing positions), but this requires counting the number of layers placed which is time consuming and difficult to do in practice.

Given these difficulties, a common approach adopted for general bulk earthworks is to observe some of the works to estimate the layer thickness, and then compare the volume of fill placed each day with the plant in operation (and efficiency of use of that plant) to check the likely maximum volume of fill that could actually be placed and compacted given the Specification restrictions on layer thickness and operational plant speed. A simple method for checking the 'potential output of compaction plant' is set out in HA 70/94 Clause 4.8 (HA, 1994).

In cases of dispute during the works it can be useful to excavate a trial pit through the placed fill and log the face as in many fill materials the successive layers of fill will be evident.

For backfill around structures the layer thickness can be managed more easily by spray marking at the required layer thickness points onto the structure that is to be backfilled.

10.7.4 Testing completed fill material

Once the fill has been placed and compacted the compliance testing tends to fall into two main classes:

- in situ density testing (which will be compared to the compaction curve for the fill)
- stiffness and bearing capacity tests.

The method and frequency of testing employed should allow for the regular movement of earthworks plant. The ultimate goal is to select a testing regime that provides a rapid means of obtaining accurate, reliable and repeatable test results, without endangering testing personnel or impeding the progress of the earthworks. Achieving the right balance for the project will be an important aspect of the earthworks validation.

The most common forms of in situ density tests are the sand replacement method, and nuclear density tests, both of which are detailed in BS 1377:1990 (BSI, 1990b). The advantage of the nuclear density test is that it is much faster to perform than the sand replacement method, but it is not suited to all fill types and has the problems of requiring careful handling procedures. An additional option is now the electromagnetic soil density gauge, which is covered by ASTM D7830/D7830M-13 (ASTM, 2013); this is less well established but does have the advantage of avoiding the handling problems of the nuclear device. For both the nuclear and electromagnetic density meters, it is essential that the equipment is formally calibrated in the laboratory for each fill type on a monthly basis, and the results provided to site.

There are various methods of testing earthworks to provide an indication of strength, stiffness or bearing capacity of the compacted fill. However, all of them tend to prove less than ideal, with some of the difficulties being as follows.

- The static plate load test is a slow procedure and the results can vary considerably due to slight differences in test method, some of which are reduced by recording the stiffness on the second load cycle (referred to in Europe as E_{v2}), but it is often difficult to have confidence in the results.
- The dynamic plate tests (such as the lightweight deflectometer (LWD)) or the Mexe cone penetrometer (often referred to as the Mexe probe) can be a useful check of earthworks operations because these tests are very quick to undertake. However, it is very difficult to use these as a compliance test because the results are influenced by various factors (such as the presence of cobbles, skill of the operator, and even atmospheric moisture conditions); consequently it is difficult to achieve an acceptable level of repeatability on some fill types.
- Portable penetration tests such as the TRL dynamic cone penetrometer (DCP) are relatively quick to perform and provide a useful indicator of CBR over the top 1 m of fill tested (thus avoiding the problem of the ground surface being changed

by recent weather conditions that limits the effectiveness of the conventional in situ CBR test).

- Undisturbed samples recovered from clay fills or stabilised fills enable laboratory testing to confirm undrained strength properties, but this results in unacceptable delays in construction.
- Full-scale static CPTs or boreholes with SPTs are used by some clients as a check of the completed earthwork (i.e. as a due diligence check), but this is of no value to control the works during construction and the results are difficult to relate to conventional earthworks compaction tests.

It is apparent from the above brief descriptions that there are considerable problems with managing earthworks projects based on testing the properties of the compacted earthwork fill. This is the main reason why the Highways Agency moved away from end product specifications and developed the method specification, because it is far easier, and generally more successful, to test the properties of the fill to ensure that it is in an acceptable condition prior to placement and then simply undertake a standard method of compaction. However, even with method compaction it is important to undertake a limited amount of in situ density testing to check that the level of compaction being achieved by the method is as expected.

10.7.5 Quality control plan

A quality control plan should be prepared at the start of the project to ensure that the QA/QC procedures are clear and effective. Ideally this document will describe how the site team will implement the requirements of the earthworks specification (such as those in Appendices 1/5 and 6/1 if the SHW is being used). The quality control plan should include items such as:

- selection of test/sample location
- frequency and method of sampling/testing
- information to be recorded at each test location (note – x , y , z coordinates should be recorded, which can be adequately recorded by hand held GPS)
- storage of test samples
- criteria for acceptance
- document control procedures
- non-conformance procedures
- site instrumentation and monitoring requirements during the works, and
- organisation chart showing who is responsible for each activity.

10.7.6 Continuous compaction control

Some modern vibratory rollers are equipped with a compaction measuring and documentation system that enables real time assessment of the effectiveness of the compaction process. The use of this type of data as part of the earthworks quality control procedure is termed ‘continuous compaction control’ (CCC).

In some cases the direction, amplitude and/or frequency of the roller’s vibrations can be automatically adjusted during operation; this has led to the term ‘intelligent compaction’.

This terminology tends to lead to confusion and is therefore best avoided. CCC is a more realistic description of the technology.

The vibratory rollers are instrumented with accelerometers that enable continuous real time measurement and reporting of the reaction of the soil during the compaction process. The purpose-built vibratory rollers are equipped with instrumentation that records and analyses the operation of the plant (e.g. travel speed, location, vibration frequency) and the vibration response. The increase in compaction of the fill is recorded as the work progresses, and weak areas can be identified. It is important to realise that the CCC system only provides a means of monitoring relative compaction; it does not directly measure the compacted density of the fill, which still needs to be measured by conventional testing techniques.

There are a number of operational factors that can influence the measured values and the specific plant suppliers' requirements would need to be carefully adhered to. In addition, for the interpretation of the measured results to be reliable, it is required that the ground surface is even (i.e. no ruts) and the fill relatively homogeneous (i.e. strongly varying material types such as cobbles and variable fines or water content should be avoided).

Calibration of the plant should be made to correct the dynamic measuring values recorded (CCC values) to the required acceptance values obtained by conventional tests such as static or dynamic deformation modulus, or the degree of compaction. During operation checks should be made of the reproducibility of the results reported by the equipment. For the results to be valid it is important that the earthworks are very carefully controlled with essential parameters such as fill type, water content, layer thickness and so on remaining consistent during the earthworks.

The manufacturers state that the method is suitable for all types of fill. However, the limitations described above do prevent usage with very coarse fills. Furthermore, the plant manufacturers recognise that special attention must be given to the water content of fine soils (i.e. soils with >15% by mass passing a 63 micron sieve).

A significant advantage of the CCC method is that the continuous measurements enable the plant operator to identify when the benefit of successive compaction passes starts to reduce, and enable informed judgements to be made about the optimum amount of compaction to be applied to a given fill and layer thickness.

Drawing on the authors' experience with other dynamic tests, it is anticipated that CCC could become sufficiently well developed to provide a primary control for simple and consistent granular fills. However, for mixed or cohesive fills CCC will probably remain simply an indicator of variability across a site. It is apparent that modern compaction plant of this type that is fitted with differential global positioning systems to record the exact location does enable a check to be made compared with a method specification approach such as SHW Table 6/4.

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Chapter 11

Cuttings: some design and construction considerations

11.1. Introduction

Cuttings are excavations made into the natural ground in order to achieve a design formation level. Unlike embankments (Chapter 12), design is concerned solely with the properties of the in situ soils and rocks, which are usually undisturbed, except insofar as the excavation process itself may introduce disturbance. This may be important with rock slopes where blasting, if used, could fracture and weaken the host rock.

11.2. Cuttings in soil: method of approach

This chapter concentrates on geotechnical aspects of cutting design and construction. More general advice is given in the Highways Agency Advice Note HA 44/91 (HA, 1995) and a list of limit states appropriate to cutting design is detailed in Eurocode 7 (BSI, 2004).

Cutting slope stability is usually considered in ultimate limit state where a portion of the slope surface suffers movement of sufficient magnitude that a definitive slip surface can be observed and slip debris occurs at the toe of the original slope surface. Serviceability limit state conditions can, however, also be important where relaxation of the slope surface, usually during construction, can cause settlement behind the crest of the slope that affects adjacent assets.

For a given geometry (i.e. depth and side slope), cut-slope stability in soil depends on:

- strength and density properties of the natural ground, allowing for its soil fabric or structure
- groundwater and/or pore pressure conditions, including pore pressure redistribution over time
- presence of features such as weak layers, unfavourably located bedrock or ancient failure surfaces.

With respect to the analysis of cut-slope stability the designer should consider the following points.

- Assessment of geotechnical information: an assessment of all geotechnical information from site investigation reports should be undertaken in conjunction

with design requirements such as cutting depth and proximity of adjacent assets and structures. This information should relate to the full cutting depth if the ground investigation is commissioned prior to the commencement of construction. It is common, however, for widening projects of existing cuttings, for example, motorway widening, that information from the original ground investigation relates to the material that was excavated as part of the original construction. Additional information may have been gained by ground investigation of the originally constructed cuttings, either as asset assessment or for design of the widening works. There is usually a difficulty, however, in investigating the full cutting depth of an existing asset, as access is not always available to the crest of the cutting slope. In addition, many asset owners carry out asset assessments involving inspection and listing of defects since construction that form an additional source of geotechnical information.

- Assessment of design properties: shear strength and density properties are assigned to the soil(s) and groundwater, or pore pressure conditions are assessed: soil structure or fabric, weak layers, ancient failure surfaces or unfavourably located bedrock are taken into account. Care should, however, be taken in the selection of design values as a conservative choice of value may have already introduced a partial safety factor prior to the application of safety factors required by the relevant design codes. More discussion is given in Nowak (2012a).
- Analysis of cutting sections: the deepest parts of the cutting are usually identified and section drawings prepared based on site investigation information; worst case conditions are usually assumed unless there is borehole or trial pit information to the contrary. The influence of soil strength properties, groundwater and pore pressure conditions on factors of safety for given slopes would normally be assessed.
- Choice of analysis method: stability is normally assessed using effective stress methods and relevant strength, groundwater (or pore pressure) data to provide the factor of safety for the long-term or permanent works condition. Drainage or other measures may be introduced if the desired factor of safety is not achieved. If an estimate of the stability for the short-term or temporary works condition is necessary, total stress methods and the relevant strength data may be adopted. The calculated factor of safety may be compared with the minimum values quoted in Chapter 13.

The procedure outlined above is repeated at structure and other significant locations, and at intermediate cutting depths, using appropriate site investigation information.

It is easy for the less experienced engineer to concentrate on what is often a relatively small length of cut slope, as shown on the drawings, and to lose sight of the 'big picture' when it comes to stability analysis. It should be remembered that the area containing the proposed cut may be part of a much larger pattern of unstable ground, requiring wider reconnaissance than simply boreholes and trial pits at the cut slope location. For this reason, expert engineering geological (and geomorphological) appreciation may be necessary at an early stage in the site investigation, employing the appropriate remote sensing aids. This also applies to embankment stability analysis (Chapter 12).

Stability analysis is thus a complicated procedure that requires more than routine allocation of soil properties and pore water pressure conditions to a proprietary computer program; it should be under the control of a suitably qualified and experienced geotechnical specialist at all times.

11.3. Stability analysis: long-term condition

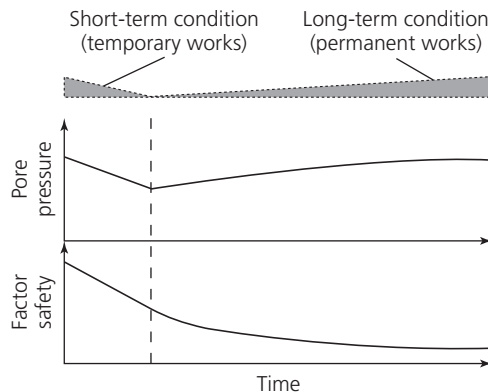
11.3.1 General

Cutting slopes with a life of decades are normally designed in terms of effective stress. This implies that, were failure to occur, it would take place so slowly that no excess pore pressures would be set up in the failing soil. For this design approach, and with slow draining clay, shear strength properties are measured in the laboratory on undisturbed specimens in consolidated drained shear (BS 1377:1990: Part 8: Test 8 (BSI, 1990)). Alternatively, consolidated undrained tests with pore pressure measurements may be made (BS 1377:1990: Part 8: Test 7).

Figure 11.1 illustrates pore pressure and factor of safety changes with time, following cutting formation. After the cut is made, the pore pressure decreases initially to a minimum at the end of construction (the short-term condition) and then increases slowly over the long term. In many cases pore water suctions can develop initially close to the surface of the cutting face. The corresponding factor of safety decreases rapidly with cut formation and then declines more slowly as the pore pressure increases with time. Bromhead (1992) states that post-cut pore pressures are unlikely to return to their pre-cut values. Applying the factor of safety to a design incorporating the initial pre-cut pore pressure should therefore ensure that the most severe pore pressure condition has been met. (This procedure would be over-conservative if the design included allowance for drainage.)

As noted in Section 3.1, in British conditions there are normally consolidated and over-consolidated soils, with the latter ranging from lightly to heavily over-consolidated,

Figure 11.1 Variation of pore pressure and factor of safety with time for a cut slope (adapted from Bromhead, 1992)



depending on the over-consolidation ratio. However, it is unlikely that many terrestrial deposits are truly normally consolidated in Britain, if only due to the changes in groundwater level that occurred in post-glacial times, and most softer deposits are lightly over-consolidated to some degree. Whether or not a soil is over-consolidated has a major impact on its behaviour in cut-slope stability; likewise its behaviour as an embankment foundation (Chapter 12).

11.3.2 Strength properties (effective stress)

11.3.2.1 Peak strength

Triaxial test results are interpreted using conventional Mohr's circle analysis and so-called best-fit tangents to produce the peak shear strength; this comprises the peak effective cohesion c' and the peak effective angle of shearing resistance ϕ' .

A c' value is usually observed with over-consolidated clays but its magnitude can be complicated by sampling or test procedures. Working with failed cut slopes in brown fissured London Clay, Skempton (1977) showed that specimen size strongly influenced the peak c' (Figure 11.2(a)). Values of c' for 38 mm dia. test specimens were much larger than those for 250 mm specimens, although the peak ϕ' values were identical in both cases. This was considered to be the influence of the fissures present in most over-consolidated clays, with larger specimens containing a more representative collection and hence lower c' values.

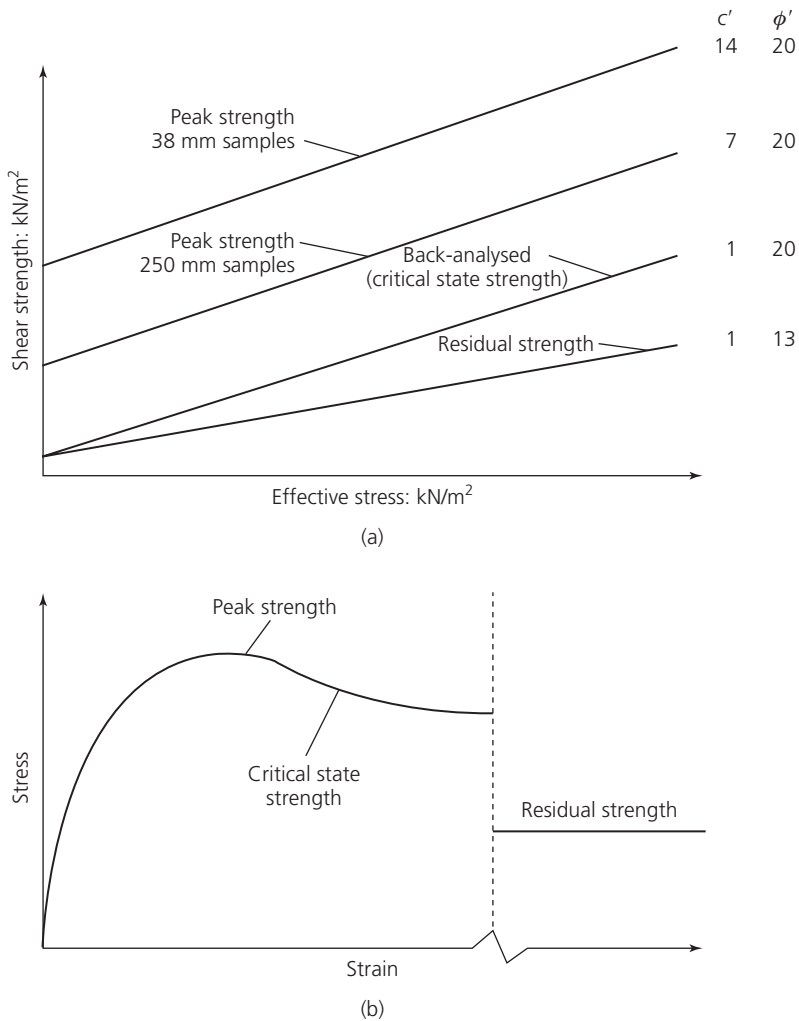
With normally or only lightly over-consolidated clays, the peak cohesion intercept c' is usually zero.

When a granular soil fails no excess pore pressures are set up, even if failure occurs relatively rapidly, because the soil is free draining. Design values of ϕ , the angle of shearing resistance, may be obtained from the results of triaxial tests or from shear box tests (BS 1377:1990: Part 7: Test 4 or 5 (BSI, 1990)). Static cone penetration tests made at the cut location, and to the expected cut depths, may be made, employing correlations such as the one given in Figure 4.4 (Chapter 4). There are also well established correlations between the results of standard penetration tests in granular soils and ϕ as described in Clayton (1995). It should be noted that even a small quantity of fine material (silt and clay) can reduce the free draining qualities of granular soils; in many cases, fine material occurs within well-defined layers or lenses, rather than evenly distributed throughout the granular soil, and this can alter drainage characteristics (see also discussion in Section 11.3.3). Trial pits and/or samples should be inspected carefully to determine the disposition of any fines.

11.3.2.2 Critical state strength

Experience shows that cut slopes in over-consolidated clays containing fissures fail at lower shear strengths than the use of peak values would imply, even allowing for full dissipation of negative pore pressures (see Section 11.3.3). Investigations indicate that, as strain increases around the shear surface, progressive failure reduces the shear strength from the peak to the critical state strength, at which shearing continues under the given stress regime without any change in volume. Dilatancy in the failure zone, accompanied

Figure 11.2 Mohr's envelope and stress versus strain curve for over-consolidated clay: (a) Mohr's envelopes of stress for over-consolidated clay (after Skempton, 1977); (b) stress versus strain curve for over-consolidated clay



by water content increase, is probably responsible for the strength reduction; fissures or other fabric features are probably central to the process. Muir Wood (1990) refers to the soil in these circumstances as being continuously remoulded and churned up, with random structure. A typical stress ν strain curve for an over-consolidated fissured clay is illustrated in Figure 11.2(b), where the peak, critical state (and residual) strengths are also shown.

Reference was made above to the work of Skempton (1977) and the dependence of peak c' on sample size. Skempton also back analysed cut slopes in brown fissured London

Clay, which had failed over several decades: it was found that they yielded even lower c' values, only slightly in excess of unity (Figure 11.2(a)); ϕ' again remaining the same. Skempton termed the back-analysed strengths the fully softened strength, which is regarded as being identical to the critical state strength, mentioned above.

The very small c' values for over-consolidated fissured clay suggest that they could be ignored in any cut-slope stability analysis in this material. However, Chandler and Skempton (1974) rejected this approach as being unnecessarily conservative and recommended the use of small effective cohesion intercepts between 1.5 and 2 kN/m². They also noted that the assumption of zero cohesion would lead to the conclusion that the limiting slope was, contrary to experience, independent of depth. Chandler and Skempton (1974) also believed that the application of zero or very small c' values would be incorrect for over-consolidated intact clay, like some Glacial Till, and pointed to the results of Skempton and Brown (1961), in which a cohesion intercept of 9 kN/m² was measured for an over-consolidated but intact clay till. They also mentioned that the use of a zero or very small effective cohesion intercept would be quite inappropriate for shale or other materials in which diagenetic bonding could lead to significant cohesion values.

11.3.2.3 Residual strength

When strain continues in over-consolidated clay, the clay platelets begin to orientate about a failure surface and, with continuing strain, they become polished and slicken-sided. In this respect, the clay microstructure is quite unlike that at the critical state strength (Muir Wood, 1990); the strength along these failure surfaces is much lower than the critical state strength, with which it should not be confused (Figure 11.2(a) and (b)). Such failure surfaces are common in British over-consolidated fissured clays and are termed 'ancient failure surfaces' in this book. They are a frequent cause of failure of both cuts and embankments and site investigations should be directed towards their discovery. A discussion on residual shear strength of failed cutting slopes in Gault Clay on the A21 Sevenoaks Bypass is given by Symons and Booth (1971).

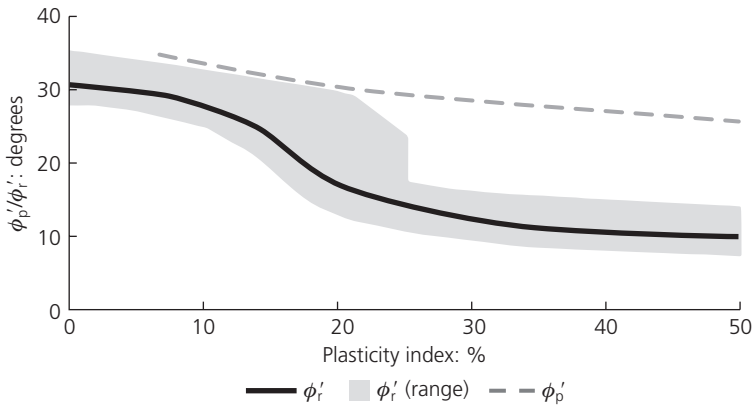
Estimates of the residual shear strength likely to exist on ancient failure surfaces in clay soils may be obtained from extended strain shear box tests on undisturbed samples (BS 1377:1990: Part 7: Tests 4 and 5 (BSI, 1990)) or from ring shear tests (BS 1377:1990: Part 7: Test 6; Bromhead, 1992). The residual cohesion value c'_r is often ignored in stability analyses involving ancient failure surfaces and the assessed value of the residual angle of shearing resistance ϕ'_r only is adopted; correlations between ϕ'_r and plasticity index, and also ϕ'_r and clay fraction are available (Figure 11.3).

Residual ϕ'_r values measured by the ring shear device may be less than the back-analysed values by between 1° and 3°, assuming c'_r to be zero (e.g. Trenter and Warren, 1996).

11.3.2.4 Selection of design strength

There is a significant number of possible approaches to parameter selection for cut-slope stability analysis. 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. Worst credible

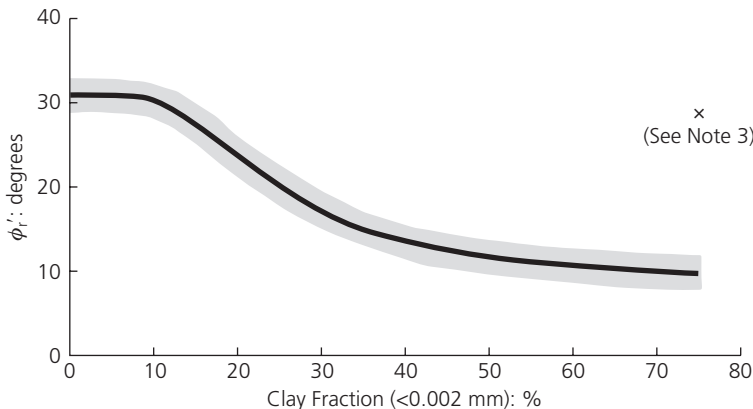
Figure 11.3 Residual angle of shearing resistance ϕ'_r versus plasticity index (a) and versus clay fraction (b)



Notes:

1. ϕ'_r is a best fit line for 20 test results reported by Skempton *et al.* (1989)
2. ϕ'_r range – the shaded area illustrates the range of test results reported by Skempton *et al.* (1989) and BS 6031:2009
3. ϕ'_p – trend line from Figure 12.2 for comparison

(a)



Notes:

1. ϕ'_r is a best fit line for 20 test results reported by Skempton *et al.* (1989)
2. ϕ'_r range – the shaded area illustrates the range of test results reported by Skempton *et al.* (1989)
3. A single result for clay sized-particles that do not comprise clay minerals (e.g. 'rock flour') illustrates that there is little reduction in ϕ'_r with increasing content of this material

(b)

parameters may be adopted 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 (BSI, 2004).

It may be viewed as conservative generally to use lower design parameters for long-term stability of earthworks slopes when likely failure surfaces are shallow with no immediate effect on the overall failure of the earthwork immediately subsequent to failure.

Bond and Harris (2008) equate characteristic design values to the ‘moderately conservative’ value used for the design of retaining walls, after Gaba *et al.* (2003). This equates to a low cautious average of the design data set. They state that ‘the difference between CIRIA 104 moderately conservative value and Eurocode 7 cautious estimate is merely one of semantics’. This should be treated with caution in light of the discussion above.

The following are offered as suggestions for first time failures in clay soil.

- For fissured or otherwise structured over-consolidated clay, adopt a critical state strength with a zero cohesion intercept c'_{cs} , if a zero intercept is measured in the laboratory. If a positive intercept is obtained, reduce it, if necessary, to a maximum of 2 kN/m². In either case, adopt the angle of shearing resistance ϕ' recorded in the tests.
- For intact (non-structured) over-consolidated clay, use the measured c' to a maximum of say 5 kN/m², again keeping the ϕ' value as recorded in the tests. The fresher and more unweathered the soil, the higher the cohesion intercept, to the limiting value quoted.
- For normally or lightly over-consolidated clay, use peak strengths, putting the cohesion intercept c' equal to zero if necessary (often zero cohesion will be measured).

For failures involving the reactivation of ancient failure surfaces:

- Adopt the measured residual angle of shearing resistance ϕ'_r , with c'_r put to zero. As noted, if measured using the ring shear apparatus, ϕ'_r may be slightly conservative and this should be remembered when assessing the factor of safety (Chapter 13).

11.3.3 Groundwater and pore pressures

11.3.3.1 General

As part of the stability analysis, the existing (pre-cut slope) groundwater table should be established. This may be done by installing Casagrande piezometers on the line of greatest slope of the proposed cut, typically at the crest of the slope, the toe of the slope and, for larger cuts, roughly at the midpoint. More than one line of piezometers may be necessary, depending on cut size and length. The depths to which the piezometers are installed will depend on the groundwater table and the strata succession, particularly on special features such as ancient failure surfaces discussed in Section 11.5.

Selecting a design water table demands much experience. Ideally, piezometric and rainfall data extending over many years should be collected in order to determine the highest possible groundwater table. This is seldom practicable and for many projects resources are such that piezometer readings are only collected for a period of months; fortunately, rainfall data for extended periods are available from the relevant authorities. Most

British sites show a rise in groundwater table in the period October/November to January/February, thereafter declining over the spring and summer, probably reaching their lowest levels in August/September time. Readings taken in the late autumn/early winter period are likely to encounter the highest groundwater tables and everything possible should be done to ensure that piezometer data are collected over that period. Even so, the groundwater table established may not necessarily be the highest on record; that would depend on antecedent rainfall at the site in question, going back many months or possibly years. For this reason, even if groundwater table readings are available for the late autumn/early winter period, they should be interpreted conservatively.

In addition to the seasonally affected groundwater table, allowance should also be made for storm effects. Skempton *et al.* (1989) investigated the storm response of the groundwater table at five sites having different ground conditions and slopes (and hence run off characteristics). They found a definite relationship between rise in groundwater table ΔH and the storm rainfall R for each site. They suggested that, as a first approximation, the ratio $\Delta H/R$ could be taken as 4 and could be employed as a characteristic parameter for any particular site. This suggests a method for allowing for the effects of a winter storm on the design groundwater table. Rainfall records for (or adjoining) the site in question may be consulted for the worst winter storm and ΔH calculated using the above ratio. This figure may then be added to the elevation of the measured highest winter groundwater table to give the design groundwater level, that is:

Design groundwater table elevation = ΔH + highest measured winter groundwater table elevation

For the reasons already given, this may not be the highest possible groundwater level, but provided a conservative assessment is made of the available winter seasons' piezometer readings, the error may not be excessive. For further details, Skempton *et al.* (1989) should be consulted.

Attention should be paid to the possible presence of aquicludes, which could support perched groundwater tables. Identification of impermeable layers, which may be very thin, can be difficult during the site investigation, especially if the work is done during a dry spell, when tell-tale seepages in boreholes and trial pits may be absent. The presence of aquicludes should always be expected in glacial soils, as should water-bearing layers or lenses of granular material.

The groundwater table selected for cut-slope design purposes will also depend on the nature and the extent of the permanent drainage measures proposed. Guidance on the reduction in the groundwater level due to various drainage configurations can be obtained from Bromhead (1992).

11.3.3.2 Special problems with fine soils

Unlike free draining granular soils or rocks, where water flows through the inter-particle spaces or through cracks, fissures or joints, the permeability of clay soil is too low for 'flow' in engineering terms. Therefore a cut made in clay reveals no free flowing water

(except from granular pockets or weepages from fissures). The low permeability of clay soils means that the 'response time' of Casagrande piezometers is such that they will take a long time for equilibrium groundwater level to establish, may only record the groundwater table response to rainfall events of long duration, and will show little if any response to a few hours rainfall. It is the same for the engineering operations of cutting (unloading) and filling (loading). These operations when lasting only hours or a few days will not be reflected in full groundwater table changes in Casagrande piezometers installed in clay soils. A more rapid measurement of pore pressure can be established using vibrating wire piezometers that will also record changes due to rainfall, loading/unloading and tidal fluctuation. These piezometers can be installed with a data logger to establish real time changes in groundwater level.

The magnitude of the pore pressure change within the clay body on excavation depends on the soil's consolidation history, that is, whether normally or over-consolidated and, for these latter soils, the over-consolidation ratio. After a cut has been made, the original pore pressures existing in the clay mass will reduce, even becoming negative for some heavily over-consolidated materials. Thereafter, the pore pressures will begin slowly to increase taking up a distribution appropriate to the new stress and drainage conditions imposed by cut formation.

The rate at which pore pressure redistribution (sometimes termed equilibration or equalisation) occurs depends on several factors including the magnitude and nature of the stress changes, as well as the coefficient of swelling of the clay itself. For heavily over-consolidated clays like the London, Gault and Lias, redistribution can take decades. Quite often cuts that have remained stable for years fail because pore pressures rise to a value sufficient to promote instability. Skempton (1977) and Chandler (1984) found that redistribution could take 40 years or more, as evidenced by the failures of cut slopes in brown fissured London Clay.

11.3.3.3 Modelling of groundwater

Groundwater is usually modelled as a phreatic surface or as r_u , the pore pressure coefficient. The latter is defined as the ratio of the pore pressure on the failure surface u to the vertical total stress from the overlying soil σ_v (i.e. $r_u = u/\sigma_v$). It should be noted that unless pore pressures are artesian the maximum value of r_u is 0.5.

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.

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 (2010) also suggests 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 under-drained and the higher value where no slope vegetation is present.

It is common to simulate the increase of water content close to the slope surface, described by an increase in r_u value in the upper 1.5–2 m 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).

11.3.4 Analysis

11.3.4.1 General

Detailed stability analysis is normally conducted using proprietary computer programs of which there are a number. The majority are based on a relatively few methods of analysis, a summary of which is given in Table 11.1.

Where no features exist to constrain the location of the potential slip surface, methods of analysis include:

- infinite slope
- circular arc.

Where such features exist, for example fissuring or relic slip surfaces as shown in Figure 11.4, potential failure surfaces must cater for them; methods include:

- non-circular
- wedge (and sliding block) analysis.

A detailed discussion of the various methods will be found in Bromhead (1992).

11.3.4.2 Infinite slope

The majority of failure surfaces in cuts (and embankment fills) are at shallow depth (e.g. Crabb and Atkinson, 1991). For a slope whose length is long in comparison to the depth to a potential failure surface, an infinite slope stability approach may be adopted; the failure surface is assumed to be a plane parallel with the embankment surface. The factor of safety for this condition (Skempton and De Lory, 1952) is given by

$$F = \frac{c' + \gamma z(1 - r_u \sec^2 \beta) \cos^2 \beta \tan \phi'}{\gamma z \sin \beta \cos \beta} \quad (11.1)$$

where β is the cut (or embankment) slope angle to the horizontal, c' is the effective cohesion, ϕ' is the effective angle of shearing resistance and r_u is the pore pressure coefficient.

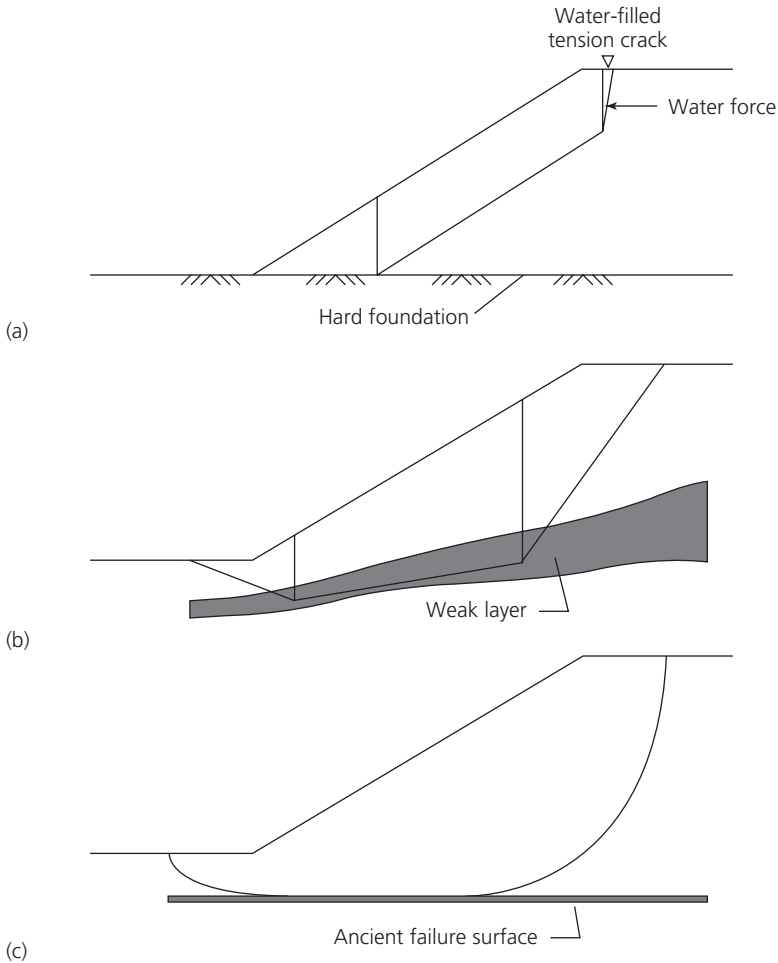
The case when $c' = 0$ (critical state strength) is given by

$$F = \frac{\tan \phi'}{\tan \beta} (1 - r_u \sec^2 \beta) \quad (11.2)$$

Table 11.1 Summary of some stability analysis methods (adapted from Geotechnical Engineering Office, 1984)

Reference	Assumptions	Advantages	Disadvantages	Recommendations
Bishop (1955) circular	Considers force and moment equilibrium for each slice. Rigorous method assumes values for the vertical forces on the sides of each slice until all equations are satisfied. Simplified method assumes the resultant of the vertical forces is zero for each slice	Simplified method compares well with finite element deformation methods (average F within 8%); computer programs readily available	Circular arcs do not always fit the failure surface (in the case of failed slopes) or the likely constraints of topography and features such as hard strata (in the case of potential failure surfaces)	Useful where circular failure surfaces can realistically be assumed
Janbu (1972) non-circular	Generalised procedure considers force and moment equilibrium for each slice. Assumptions on line of action of inter-slice forces must be made. Vertical inter-slice forces not included in routine procedure and calculated F is then corrected to allow for vertical forces	Realistic failure or potential failure surfaces can be adopted. Routine analysis can be easily handled by a programmable calculator or by hand	Published correction factors are for homogeneous materials and use of routine procedure can produce large errors in slopes composed of more than one material. Factor of safety is usually underestimated in these cases. Generalised method does not have the same limitations	Very useful where there are topographic and other constraints on geometry of the failure surface. Limitations of the routine method must be considered. Routine method not suitable for embankments where the potential failure surface extends deep into underlying foundation soil
Morgenstern and Price (1965) non-circular	Considers forces and moments for each slice; similar to Janbu procedure	Considered more accurate than Janbu; computer programs readily available	No routine (or simplified) method. Computer solution essential and often very time consuming	Very useful where there are topographic and other constraints on the geometry of the failure surface. Most useful for back-analysis of existing landslides
Sarma (1979) non-circular	A modification of Morgenstern and Price method, which reduces the number of iterations necessary by the application of earthquake forces	Considerable reduction in computing time without loss of accuracy	Computer and programmable calculator solutions available	As for Morgenstern and Price

Figure 11.4 Slip surfaces for use with cut (and embankment) slope stability analysis: (a) sliding block mechanism (from Jewell, 1996); (b) wedge mechanism; (c) non-circular mechanism



11.3.4.3 Circular arc

Using this method, sufficient slip surfaces are analysed to permit the grid of circle centres to be contoured. Experience shows (Chandler and Peiris, 1989; Spencer, 1967) that the minimum factor of safety is most likely to be found when contouring a series of ellipses whose major axis is at right angles to the slope and slightly up-slope of its mid point. Trial circle centres should be adopted with this in mind.

11.3.4.4 Non-circular

Note the remarks made in Table 11.1 concerning the limitations of the method of Janbu (1972). If a computer program based on this method is being employed, it would be wise to determine whether or not it uses the routine or 'generalised' procedure.

11.3.4.5 Wedge analysis

The wedge analysis is a force equilibrium method and results cannot be directly compared with the Morgenstern and Price (1965) and Janbu (1972) methods, which utilise a combination of force and moment equilibrium. The difference between these methods increases as the factor of safety increases. The Geotechnical Engineering Office (1984) points out that the factor of safety obtained from a wedge mechanism is sensitive to the inclination to the horizontal of the inter-block forces and the inclination of the surfaces between the blocks. An analysis to determine the sensitivity of these factors for a given problem would appear appropriate, especially for larger slopes and/or where the consequences of failure could be particularly severe (Chapter 13).

Greenwood (1985) believes that in conditions where the earth pressure at rest value K_0 is high (such as in heavily over-consolidated clay at shallow depth), the wedge analysis will provide a lower factor of safety than the circular arc.

Jewell (1996) put forward an alternative to the infinite slope comprising two sliding blocks, with a water-filled tension crack at the top, which is really a special case of the wedge analysis. Unlike the infinite slope approach, it applies when the ratio of the length of the side slope to the depth to the potential slip surface is relatively small. The depth of tension crack used in this method may be calculated from the expression

$$z_0 = \frac{2c'}{\gamma} \tan(45 + \phi'/2) \quad (11.3)$$

Peak strengths may be adopted when using Equation 11.3, because tension cracks form at an early stage in the cut (or embankment) slope and they are unconnected with the critical state or residual strength processes.

The factors of safety computed using the above procedures may be compared with the minimum values suggested in Chapter 13.

11.3.5 Some results based on British motorway experience

In a valuable contribution, Perry (1989) collected data on the performance of embankments and cuts for a number of English and Welsh motorways. Information was provided for various height ranges, geological ages, drainage conditions, side slopes and other factors, with stability for the particular material under examination being expressed as a percentage of the failed length of slope to the total length of slope surveyed. Of the 570 km of earthworks involved, failure affected some 4%; failure for both cuts and embankments usually comprised a combination of translational (slab) and rotational movements, only exceeding 1.5 m depth in 5% of the slopes investigated. The greatest incidence of failure occurred with over-consolidated clay of Tertiary age and older.

A selection of results for cut slopes is given in Table 11.2. Clay soils (particularly over-consolidated) were chosen because of their greater incidence of failure.

A height range from 2.5 m to 5 m was adopted because Perry reported that 76% of the cuts surveyed were lower than 5 m high.

Table 11.2 Performance of British motorway cut slopes from 2.5 m to 5 m high (from Perry, 1989)

Geological stratum	Age of construction: years	Length surveyed: m	Slope angle: v : h	Percentage failure
Glacial Till	3 to 18	4142	1 : 2.5	0
	22	204	1 : 2.5	20.5
	7 to 17	3291	1 : 2	0
	18	1750	1 : 2	1.0
	22	2319	1 : 2	3.7
	25	463	1 : 2	5.8
London Clay	6	533	1 : 3	1.9
	10	543	1 : 3	3.2
Gault Clay	10	353	1 : 2.5	3.8
	22	299	1 : 2.5	4.4
Lower Lias	4.5	529	1 : 2	0
	25	894	1 : 2	1.4

Generally, failure rates are seen to be reasonably low, usually less than 5%; the exception is a section of 22-year-old Glacial Till cut, where a failure rate of 20.5% was recorded. The small length surveyed (204 m) suggests special factors operating. Understandably, without borehole information, Perry was unable to draw attention to such matters as ancient failure surfaces, which could be present at some of the failed sections.

The incidence of failure increases with the age of the earthwork, thus demonstrating the importance of age in stability assessments, together with a clear idea of the degree of maintenance work acceptable. In this respect, Perry (1989) suggests that for long-term stability (corresponding to less than a 1% failure in 25 years), side slopes of 1 in 5 would be necessary in Gault and Lias Clays, both for cuts and embankments in excess of 5 m high. Such slopes are substantially shallower than the vast majority recorded during Perry's survey and would have a major impact on first time project costs.

11.4. Stability analysis: short-term condition

11.4.1 General

The total stress approach to stability analysis assumes that cut-slope formation takes place so rapidly that there will be no change in the water content of the soil or a development of pore water suction. The soil acts in an undrained manner despite the engineering operations. It is assumed that the results of undrained shear strength measurements made on samples taken from the cut area before construction begins (typically at the site investigation stage) will be representative of the strength of the soil during the main works. Note that the undrained strength of a clay soil is critically dependent on its water content: a relatively small change creates a corresponding large change in undrained shear strength, which is why the assumption of the undrained condition is so critical.

Experience shows that provided the drainage paths in the cut slope are not too short (which would enable rapid drainage), or that the construction period is not too long, the total stress approach may be adopted for cuts in clay soils over the short term. The above two caveats are, however, important: the engineer must be convinced by the site investigation that there are no significant drainage paths in the cut slope soil. Such features could include abundant fissures or foliations in over-consolidated clays, granular layers in an otherwise clay till or light cementing, which could sustain open fissures. Moreover, it must be demonstrated that any significant delays to the construction programme are unlikely to affect the short-term condition.

For the design of a 12 m high temporary cut face at 2V : 1H in Dublin Boulder Clay at the northern end of the Dublin Port Tunnel project, analysis showed that the presence of thin granular layers considerably reduced the short-term effect of pore water suction on stability and that failure was likely to occur within days rather than within months if no granular layers were present.

The doubts surrounding the practicability of applying the total stress analysis together with the difficulty in selecting parameters in total stress terms (see below) are reasons why some engineers are reluctant to adopt this approach, preferring to analyse in effective stress terms, making the best estimate possible of pore pressure conditions after cut formation.

11.4.2 Strength properties (total stress)

A major disadvantage with the total stress approach is the scatter of results normally obtained from undrained triaxial tests made on conventional driven tube samples and the difficulty of deciding on suitable design shear strength values as a consequence. The presence of fissures and foliations in the clay is an obvious cause of scatter resulting from testing, but sample disturbance may also lead to inaccuracies. Vaughan *et al.* (1993) made a major contribution to the study of sample disturbance in clay soil and part of their results are summarised in Table 11.3.

An approach to determining the undrained (total stress) shear strengths for stability analysis purposes could be as follows.

- For soft (normally or lightly over-consolidated) clay: sample disturbance will result in a reduction in shear strength, the amount depending on the soil's plasticity (Table 11.3). A mean of the available test data is suggested; the lower the plasticity, the less conservative the mean. It may be preferable to use the results of in situ vane tests (BS 1377:1990: Part 9: Test 4.4 (BSI, 1990)) rather than the results of conventional undrained triaxial tests made on samples.
- For stiff (over-consolidated) clay: as indicated in Table 11.3, sampling effects are negligible for low plasticity material, and a conservative assessment of the mean is suggested. For high plasticity clay, sampling may result in an unrepresentatively high shear strength, and a lower bound assessment may be considered
- For granular soils, employ a correlation such as the one shown in Figure 4.4 (Chapter 4).

Table 11.3 Effect of sample disturbance on undrained strength
(from Vaughan *et al.*, 1993)

Material type	Effect on undrained shear strength
Soft clay:	
■ Low plasticity	Very large decrease
■ High plasticity	Large decrease
Stiff clay:	
■ Low plasticity	Negligible
■ High plasticity	Large increase

Guidelines on minimising sample disturbance are provided by BS EN 1997-2:2007 (BSI, 2007) and BS EN ISO 22475-1:2006 (BSI, 2006). The latter document categorises sampling methods in relation to their acceptability specific to the determination of specific laboratory test parameters. The determination of strength parameters is prescribed to sampling by the use of thin walled tube samplers. This was already covered by the use of pushed in piston samplers for soft fine soils, but has led to the development of thinner walled sample tubes being employed for sampling stiffer fine soils than in the past. Less sample disturbance can also be achieved by sampling stiffer fine soils by the use of block samples in trial pits or the use of rotary coring.

11.4.3 Analysis of cut slopes

All of the methods proposed in Section 11.3.4 are applicable to the total stress analysis; most proprietary computer programs have an effective stress and a total stress option. Note that the expression for the factor of safety for the infinite slope condition in total stress terms (Atkinson, 1981) is

$$F = \frac{c_u}{\gamma H \sin \beta \cos \beta} \quad (11.4)$$

where c_u is the undrained shear strength, γ is the unit weight, H is the depth to the failure surface and β is the inclination of the slope to the horizontal.

Factors of safety obtained may be compared with the minimum values proposed in Table 13.5 (Chapter 13).

11.5. Effect of shallow depth features on stability

11.5.1 General

Although deep cuttings and excavations are the obvious exceptions, earthworks are usually near surface structures. Consequently, when considering the design and construction of cuts (and embankment foundations), investigation must take into account shallow depth features, such as glacially induced soil fabric and solifluction deposits and their influence on stability. Such features were generated during the Pleistocene glaciations that affected most of Britain (Jones and Keen, 1993; Trenter, 1999). As its name suggests,

glacially induced soil fabric is a product of glacier or ice sheets; solifluction deposits occurred in the frozen periglacial lands beyond the glacier or ice sheet front.

There is an extensive literature dealing with the whole subject of shallow depth features and their engineering significance, of which those mentioned here are a small sample. A full discussion is given in Hutchinson (1991).

11.5.2 Glacially induced soil fabric

Fabric denotes the directional properties of a soil and rock, including solids and voids. It contains such features as layers, lenses, laminations, fissures, cracks and joints. The latter three items are often referred to as discontinuities and have a substantial influence on stability.

Few if any discontinuities, including shear surfaces, are recorded in granular till; clay tills, especially the more plastic variety, are most likely to contain them and to provide a rich source of potential stability problems for both cuttings and embankment foundations. More details of glacially induced soil fabric are provided by Trenter (1999).

A good example of glacially induced soil fabric controlling the stability of cut slopes was described by McGown *et al.* (1974). Fissures in clay till striking parallel with and dipping towards a 14 m deep cut at Hurlford, Ayrshire, caused failure of the cut slope within weeks of construction; no problems occurred on the other side of the cutting where the fissures dipped into the cut slope.

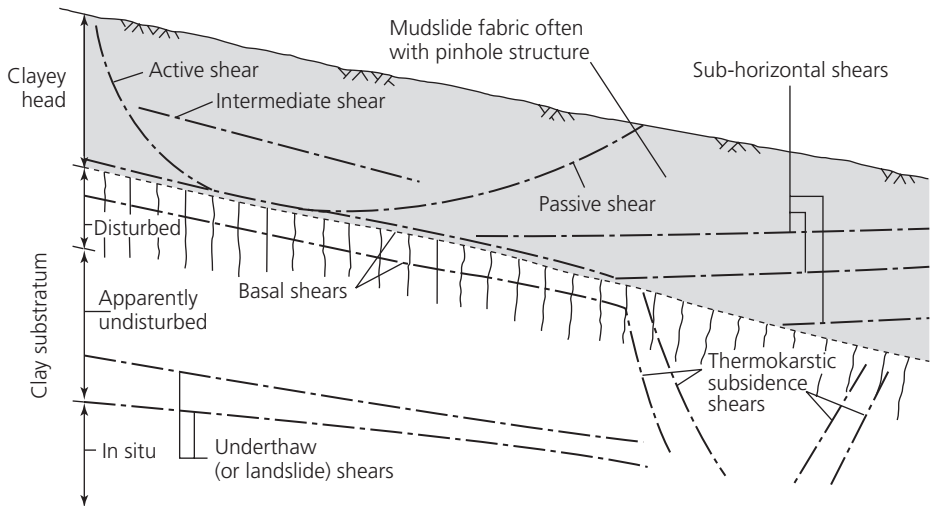
11.5.3 Solifluction deposits and shear surfaces

Solifluction deposits were formed in a periglacial environment as a consequence of the high pore pressures set up during thawing of surface soils and, as such, may mantle very gentle slopes having gradients of only a few degrees. Lithologically, the deposits reflect the materials up-slope from which they were derived and they may attain thicknesses from a few tens of millimetres to several metres in accumulation zones down-slope.

Provided that they are uncemented or that they have not been over-consolidated by desiccation, solifluction deposits are normally softer than the deposits from which they were derived. This is immediately apparent to the experienced geotechnical engineer or engineering geologist carrying out a trial pit investigation. Less easily observed are slip surfaces that are found in the more clayey solifluction deposits, which include those formed from the over-consolidated clays such as the British Gault, Weald and Lias. Figure 11.5 illustrates the variety of slip or shear surfaces common in clayey solifluction deposits; shears in four directions are illustrated. Also shown are the basal, thermokarstic and underthaw shears occurring in the soils beneath. For full details, Hutchinson (1991) should be consulted. An account of solifluction deposits in southern England, with their associated ancient shear surfaces, is also given by Weeks (1969) and Skempton (1976). A case study of solifluction features in London Clay and Reading Beds material during construction of the M25 Motorway in Buckinghamshire is given by Spink (1991).

The effect of soil structure on shear strengths obtained using total stress testing methods was mentioned in Section 11.4.2.

Figure 11.5 Main types of shear surface in solifluction deposit and underlying stiff clay strata (from Hutchinson, 1991)



11.5.4 Other shallow depth features

In glacial successions, laminated silty clays of glaciolacustrine origin are notorious for promoting failures, not only of cut slopes but of landslides as well (e.g. Trenter, 1999). Intra-formational shears (very thin low strength clay seams, often continuous over a wide area), for example volcanic ash derived or bentonitic layers, have been reported in clay shales and mudstones (Rogers, 1985), and may have markedly lower shear strengths than the adjoining soils or rocks. Rogers notes that they most often occur when sandstones and limestones are interbedded with argillaceous sediments.

Where cuttings are to be formed in clay soils or weak rocks containing ancient failure surfaces, note that 'residual shear strengths' may operate along the slip surfaces themselves (see Section 11.3.2). If undetected during the site investigation and not taken into account in design, ancient failure surfaces may cause major failures, even with relatively flat cut slopes. Unfavourably located bedrock can also contribute towards instability of overlying soil masses when cut slopes are made. In addition, instability can occur in rock slopes formed by the removal of soil cover where peri-glacial cambering has produced a rock mass with an open joint structure. This feature was experienced during the construction of cuttings in Oolitic Limestone the A419 where, once topsoil and weathered material had been removed, the bedrock was encountered as a series of individual blocks of, generally, 500 mm × 500 mm size with a spacing of some 100 mm between each block.

11.6. Drainage and frost-heave

11.6.1 General

Details of drainage design will be found in the appropriate texts but, in general, permanent works drainage includes:

- cut-off drains
- surface water interceptor drains
- counterfort and slope drains
- herringbone drains.

11.6.2 Cut-off drains

These drains are installed through a pervious stratum such as a water-bearing sand or gravel layer into underlying impermeable clay or bedrock. They comprise a granular filled trench, usually with a geotextile surround between the granular fill and the host stratum, and with a perforated pipe at the base, designed to take the discharge. Normal filter rules apply with respect to the granular fill. Proper connections should be made between the cut-off and any field drains up-slope. The disadvantage with cut-off drains is that they may not always reach the impermeable material, so that the drain empties its contents into the pervious stratum producing the contrary to the desired result. Furthermore, if falls are insufficient, water may drain out of the pipe through any discontinuities in the underlying impermeable stratum causing its softening. Care should be taken to see that water from the drain is channelled away from the toe of the cutting.

11.6.3 Surface water interceptor drains

Unlike cut-off drains, which are designed to cope with groundwater, surface water interceptors are designed to collect run off that would otherwise daylight on the cutting slope. They are, therefore, installed on the uphill side of cuts excavated in side-long ground. They may consist of open channels, often concrete lined, or granular filled trenches similar to cut-off drains. The former are not recommended because open channels block easily; moreover, an improperly designed concrete lining could crack permitting ready access for water to the underlying soil. As with cut-off drains, water must be channelled away from the cutting toe.

11.6.4 Counterfort and slope drains

Counterfort drains are deep granular filled trenches excavated into the slope, at centre to centre spacings dependent on the permeability of the host soils; methods for calculating drain size and spacing are given by Bromhead (1992). Much more expensive than cut-off drains, because of the quantity of excavation and granular fill needed per linear length of cut, they are also much more effective: first, because they can pick up irregular or inconsistent layers and lenses of permeable material and are therefore ideal for glacial terrains; second, because they give strength and stiffness to cuts in otherwise weak ground. For this reason, they are often employed for reinforcing cuts in water-bearing material containing ancient failure surfaces. Note that the flows discharging into the toe drain linking the counterforts can be large and the toe drain itself requires careful design; inadequate capacity would mean that water backed up in the counterforts, keeping water pressures high and possibly promoting water 'break-outs' part way up the slope.

Slope drains are similar in principle to counterforts except that they are much shallower, only extending one or two metres below the slope surface. Consequently they are unable to pick up the same amount of groundwater flow. However, given that most instability is at shallow depth, it may be that they fulfil a useful function in lowering the groundwater

table near the slope surface, with beneficial results. Due to their smaller size, they will not have the same stiffening effect on slopes as the much larger counterforts and may not be successful in stabilising slopes containing ancient failure surfaces.

11.6.5 Herringbone drains

As noted in Section 11.6.6, flatter slopes are not always more stable than steeper slopes. Drains, comprising shallow granular filled trenches excavated into the slope in a herringbone pattern, would appear useful in such circumstances. They can also pick up layers and lenses of permeable material, which would otherwise discharge water at the slope face, perhaps producing dangerous back-sapping.

A good discussion of geological problems attendant on some forms of drainage is given by Barton and Thomson (1986).

11.6.6 Some experience in British conditions

Permanent drainage of cut slopes is critical to their long-term stability. In a survey of cuts in England and Wales, Perry (1989) reported that the greatest percentage of failures occurred where no drainage had been provided. In an experiment in over-consolidated fissured Gault clay, a 9 m high by 25 m wide slope constructed at 1 (horizontal) in 2 (vertical) was brought to failure by the controlled addition of water, thus conclusively demonstrating, if such demonstration were needed, that a rising groundwater table can cause slope failures (Petley *et al.*, 1991). Farrar (1985) describes how the installation of slope (counterfort) drains at 5 m centres stabilised an existing cutting in the brown London Clay of the A12 trunk road. Failure first occurred some 35 years after construction, after which the slope was cut back to between 10° and 13°, and instability continued until the drains were installed.

In over-consolidated soils, flatter slopes are not always more stable than steeper slopes, according to a study by Parsons and Perry (1985). It was conjectured that rainwater run off was less easily shed from flat slopes. Potentially deleterious infiltration into the body of the clay slope occurred through features such as desiccation cracks and fissures more readily with flatter slopes than with steeper slopes, producing a build-up in pore pressures at shallow depth. Drainage should not be neglected simply because shallow side slopes have been selected. Herringbone drains, discussed in Section 11.6.5, would appear useful in such cases.

If, as is usually the case with cut slopes excavated below the general groundwater table, permanent drainage measures are to be installed, their construction may be married with the temporary works drainage to improve efficiency and reduce cost. Filters for permanent drains may become clogged during construction at particularly muddy sites and this should be kept in mind.

11.6.7 Outfall of drainage measures

Any drainage design should consider the discharge of collected water. This can be into existing water courses or pipework, possibly through ponds or attenuation facilities to remove fine material and contaminants. Alternatively a sustainable drainage system

(SUDS) can be designed. Details of SUDS systems can be found in Woods and Kellagher (2007). In all cases, discharge should be agreed with the relevant authorities, for example the Environment Agency and water companies.

11.7. Vegetation

The effect of vegetation on long-term stability is not extensively considered in the design of new earthworks slopes. Perry *et al.* (2003) 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 attributes are indicated in Figure 11.6.

Perry *et al.* (2003) indicate that most root reinforcement extends generally to 50 mm depth below the slope surface, with tree roots penetrating up to 3 m depth.

Norris *et al.* (2008) build on the work of Greenwood *et al.* (2004) and Norris and Greenwood (2006) in developing the concept of an increase in soil cohesion due to root growth (c'_r). Typical values of c'_r from historical data after Norris *et al.* (2008) are summarised in Table 11.4.

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.

Figure 11.6 Forces exerted on a slope by vegetation (after Norris *et al.*, 2008)

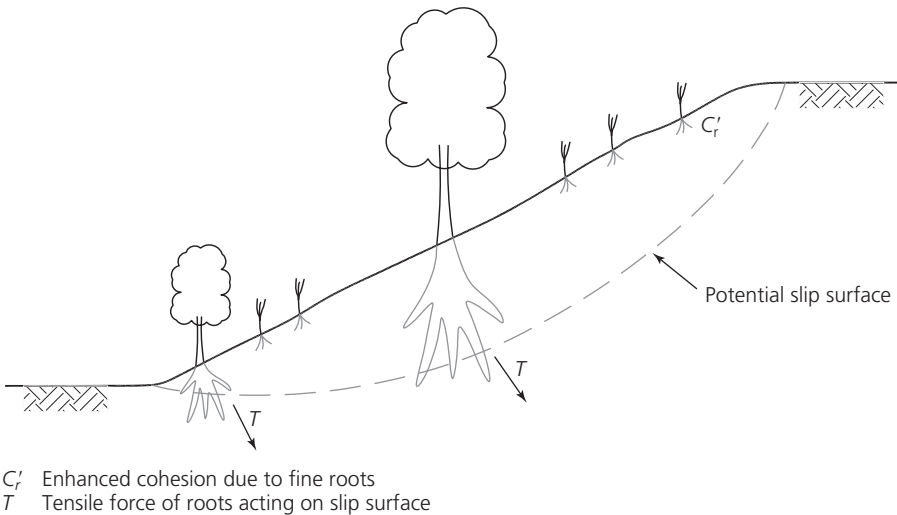


Table 11.4 Shear strength contribution from vegetation

Vegetation type	c'_r : kN/m ²	
	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

A practical observation of enhanced c' due to plant growth is provided by the Longham Wood Cutting Field 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 *et al.* (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 (London Underground, 2010) allows a reduction in design r_u of 0.05 where cutting slopes are vegetated.

11.8. Cuttings in rock: method of approach

11.8.1 General

Cut-slope stability problems normally arise in rocks because of the discontinuities they contain and their degree of weathering, rather than the weakness of the intact material. Discontinuities may be bedding, joint, cleavage or fault planes created by the geological evolution of the rock mass itself, or they may be fissures or fractures caused by civil work, such as blasting. However caused, discontinuities and the rock's degree of weathering are fundamental to stability and to the design of cut slopes in rock.

Design rests on a thorough analysis of all available discontinuity data, because in many cases they control the location of potential failure surfaces or of groundwater seepage or flow. Indeed, a failure surface may occur along or through a major discontinuity set. Different circumstances may operate for weathered rocks whose behaviour in stability terms may be more akin to soil than to rock, depending on the degree of weathering. Instability is liable at the junction between the highly weathered and the less weathered materials. Because the intact strength of weathered rocks is relatively low (compared with that of fresh unweathered rock), water pressure can have a major influence on stability in weathered terrains.

Some weak rocks are relatively uniform and lacking in significant structural features; in such cases, the difference between them and most over-consolidated (stiff fissured) clays lies in their natural water content, which is normally substantially lower than their plastic limit. For such materials, the slopes at which cuts can be engineered depend on shear

strength and not discontinuities. Nevertheless, the shear strength for such materials can be substantially higher than for over-consolidated clays. By reference to Mercia Mudstone cliffs at Sidmouth, Devon, up to 100 m high, Chandler (1984) demonstrated a high effective cohesion value c' of 40 kN/m² and an angle of shearing resistance ϕ' of 34°; similar high effective strengths were described for chalk cliffs. Lower effective strengths, although still considerably above those of over-consolidated clays, were noted for higher plasticity weak rocks, where progressive failure might be anticipated. For further details, Chandler (1984) should be consulted.

11.8.2 Stereonets and classifications

11.8.2.1 Stereonets

An indication of the stability of rock slopes can be obtained by plotting the discontinuity data on a stereonet and comparing their orientation with that of the proposed cut slope. Figure 11.7 demonstrates such plots for three typical types of rock failure: plane, wedge and toppling. In assessing the likelihood of failure, account has to be taken of the angle of shearing resistance ϕ acting along the potential failure surface (see below). The stereonet approach is a specialist operation, requiring input from a suitably qualified and experienced rock mechanics engineer or engineering geologist and permits ready visualisation of the discontinuity pattern and its relation to cut slope geometry. It should always be attempted before more complicated analytical techniques are adopted. For full details, Hoek and Bray (1997) should be referred to.

11.8.2.2 Classifications

Preliminary indications of cut-slope stability in rock can be obtained using rock classification schemes. Bieniawski (1989) listed various attributes of such schemes. They include the ability to:

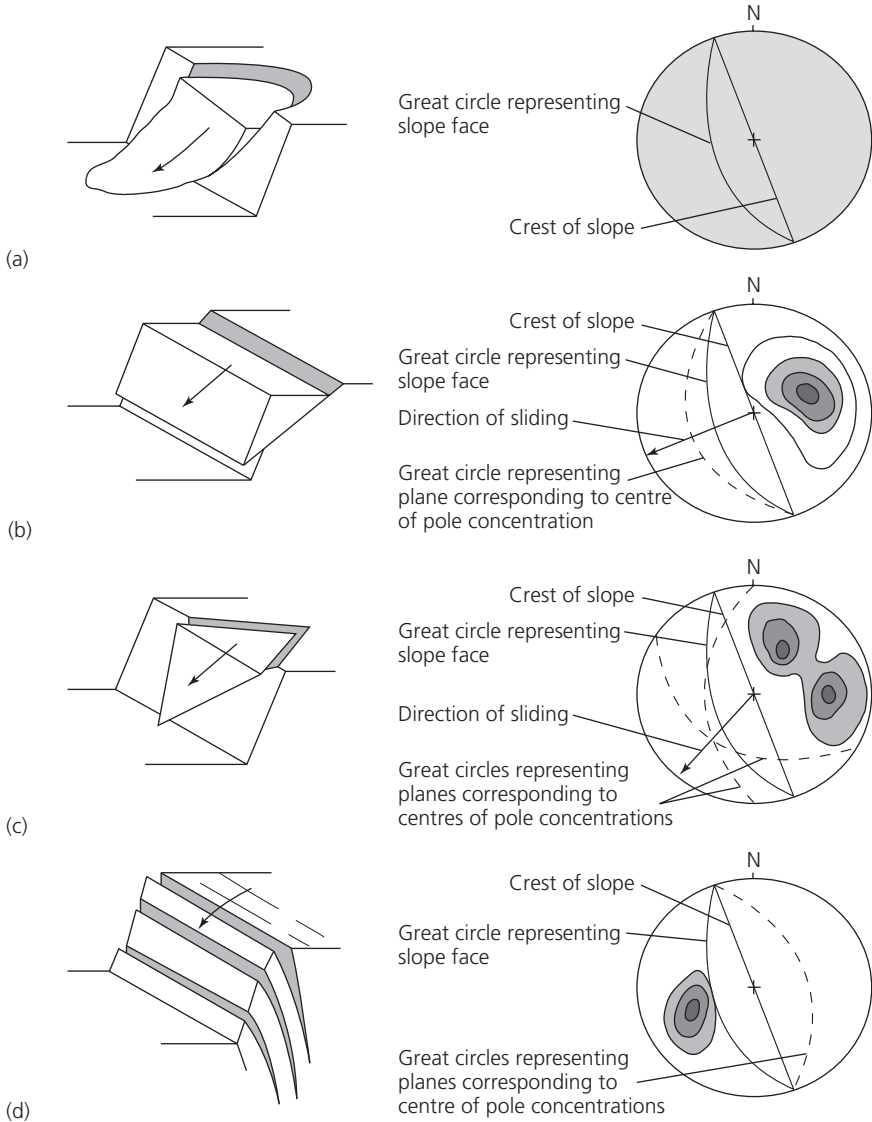
- identify the most significant parameters influencing rock mass behaviour
- identify and group rock masses having similar engineering characteristics
- provide a basis for communication between site and design office and the various disciplines working on the project.

A well known classification scheme for tunnels in rock was proposed by Bieniawski (1979, 1989). This scheme was modified for rock slopes by Romana (1985) and is presented in Figure 11.8. The information required includes that obtainable from both rotary core drilling (specifically the rock quality designation) and from logging exposed rock faces (the joint adjustment rating). It also takes into account the method of slope formation (the adjustment rating for methods of excavation). The strength of the rock mass classification approach lies not so much in the answer it provides but in the opportunity it affords systematically to register various relevant data. Like the stereonet, which also registers physical data in a highly systematic way, it is a good accompaniment to the more analytical approaches available.

11.8.2.3 Measuring discontinuity data

Fundamental to both the stereonet and the classification approaches are reliable discontinuity data. They are best obtained from existing cuts or from natural rock exposures

Figure 11.7 Main types of failure in rocks and stereoplots of the discontinuity conditions likely to cause them: (a) circular failure in overburden soil, waste rock or heavily fractured rock with no identifiable structural pattern; (b) plane failure in rock with highly ordered structure such as slate; (c) wedge failure on two intersecting discontinuities; (d) toppling failure in hard rock, which can form columnar structure separated by steeply dipping discontinuities (from Hoek and Bray, 1997)



using the line-sampling or scan-line technique. This requires a line (scan-line) to be run out and discontinuity data (e.g. dip, strike, openness, roughness, in-filling, as required by Anon., 1977) recorded, together with the distance between each discontinuity. The data are then plotted to scale. Other lines are run out at regular intervals, both parallel to the

Figure 11.8 Modification of the geomechanics classification for rock slopes by Romana (1985) (from Bieniawski, 1989)

Parameter		Bieniawski (1979) ratings for RMR (ranges of values)						
1	Pointload strength	>10	4–10	2–4	1–2			
	UCS strength: MN/m ²	>250	100–250	50–100	25–50	5–25	1–5	<1
	Rating	15	12	7	4	2	1	0
2	RQD: %	90–100	75–90	50–75	25–50		<25	
	Rating	20	17	13	8		3	
3	Discontinuity spacing	>2 m	0.6–2 m	200–600 mm	60–200 mm		<60 mm	
	Rating	20	15	10	8		5	
4	Condition of discontinuities	Very rough surfaces. Not continuous. No separation. Unweathered wall rock	Slightly rough surfaces. Separation <1 mm. Slightly weathered walls	Slightly rough surfaces. Separation <1 mm. Highly weathered walls		Slickensided surfaces. Or gouge <5 mm thick. Or separation 1–5 mm. Continuous		Soft gouge >5 mm or Separation >5 mm. Continuous
	Rating	30	25	20		10		0
5	Groundwater in joint	Completely dry	Damp	Wet		Dripping		Flowing
	Rating	15	10	7		4		0

Rating refers to the Rock Mass Rating (RMR) of Bieniawski

Adjustment rating for joints						
Case		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
P	$[\alpha_j - \alpha_s]$	>30°	30–20°	20–10°	10–5°	<5°
T	$[\alpha_j - \alpha_s - 180°]$					
P/T	F_1	0.15	0.40	0.70	0.85	1.00
P	$[\beta_j]$	<20°	20–30°	30–35°	35–45°	>45°
P	F_2	0.15	0.40	0.70	0.85	1.00
T	F_2	1	1	1	1	1
P	$\beta_j - \beta_s$	>10°	10–0°	0°	0–(–10°)	<–10°
T	$\beta_j + \beta_s$	<110°	110–120°	>120°		
P/T	F_3	0	–6	–25	–50	–60

P denotes plane failure; T denotes toppling failure; [] denotes numerical value only.
 α_s denotes slope dip direction; α_j denotes joint dip direction. β_s denotes slope dip; β_j joint dip

Adjustment rating for methods of excavation of slopes					
Method	Natural slope	Pre-splitting	Smooth blasting	Regular blasting	Deficient blasting
F_4	15	10	8	0	–8

Tentative description of slope mass rating (SMR) classes					
$\{SMR = RMR(rating) - (F_1 \times F_2 \times F_3) + F_4\}$					
Class number	V	IV	III	II	I
SMR	0–20	21–40	41–60	61–80	81–100
Description	Very poor	Poor	Fair	Good	Very good
Stability	Very unstable	Unstable	Partially stable	Stable	Fully stable
Failures	Large planar or soil-like	Planar or large wedges	Some joints or many wedges	Some blocks	None
Support	Re-excavation	Extensive corrective	Systematic	Occasional	None

original and at right angles, to the extent that access and the degree of exposure make possible. Once collected, the data may be presented in both stereonet and classification formats.

Where no existing cuttings or natural exposures are available, the engineering geologist normally has to fall back on the discontinuity data obtained from rock cores. A high standard of logging is required, using the methods presented by Anon. (1970). It is not as reliable as line sampling for the following reasons.

- Unless special measures are taken (see below), the true dip direction of the discontinuities is usually unknown.
- It can be difficult to differentiate between drilling-induced cracks and geologically formed discontinuities.
- The degree of openness of the discontinuities cannot reliably be inferred because of the process of core extraction.

The first of the above-mentioned difficulties can be overcome, in some cases, by directional drilling, or by employing special equipment. A useful discussion of a number of techniques for this purpose is given by Hoek and Bray (1997). The performance of three different techniques of assessing true dip direction in Middle Coal Measure sandstones is given by Webber and Gowan (1995). The use of closed circuit TV for down-the-hole surveys is reported by McMillan *et al.* (1995). A discussion on the methods of investigating discontinuities in rock cuttings is also given by Matheson (1985).

11.8.3 Drainage aspects

As already noted, water pressures acting on the surface of any discontinuities can substantially reduce the stability of cut slopes in rock. The amount of reduction and hence the need for drainage can be gauged during design but is likely to be greatest in intensely fissured and fractured rocks, and/or those that are highly weathered. Because weathering of rocks can be so variable, both in depth of penetration and horizontal extent, borehole and trial pit information may mislead. A flexible approach should be adopted during construction in those cut areas believed to be both weathered and below the groundwater table; more elaborate drainage measures may be necessary than are proposed for the unweathered rocks along the route. Such an approach will require the input of a suitably experienced engineering geologist and drainage engineer during the construction stage. As far as possible, standard drainage details should be employed for various weathering regimes to reduce the delays due to re-design at this stage.

11.8.4 Cuts in weathered or fissured rock

If not foreseen during design, weaker weathered or intensely fissured and jointed rocks in the cut slopes can result in delays and cost overruns. The same applies to rock at formation level. Both the side slopes and the formation should be inspected at an early stage during construction. This should permit decisions to be made on site as to the nature of any side slope support necessary for the permanent works and the thickness of the pavement construction.

Hencher and McNicholl (1995) describe how rock destined for cut-slope formation was found on excavation to be substantially more weathered than expected on the basis of site investigation information. Complete re-evaluation of slope stability was necessary, including the installation of support measures (see below) and the purchase of additional land to permit shallower slopes. The weathering of seat-earths in Coal Measures rocks can provoke difficulties and the same authors describe temporary stabilising works for sections of a trunk road through these conditions, designed on a modular basis, 15 m in length.

In difficult terrains, a variety of designs for both cut slopes and pavements should be prepared, in advance of the main works, graded in accordance with the degree of weathering and/or fissuring. Construction may then proceed if these conditions are discovered, with the minimum of delay.

11.9. Stabilisation of cutting slopes

11.9.1 General

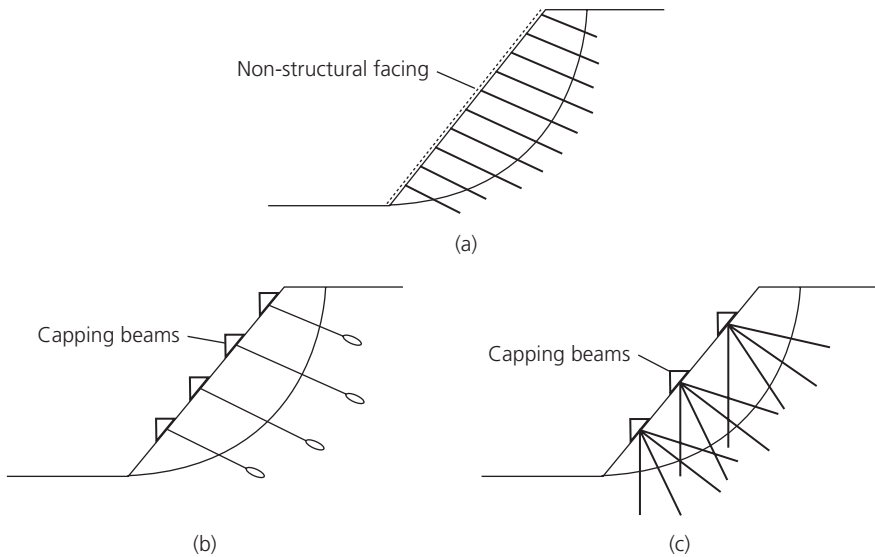
Cut slopes may require stabilisation for the following reasons:

- failure of the slope as originally designed
- strictly limited land-take, requiring steeper slopes than the ground would safely provide unaided. This is common in an urban or sub-urban environment
- ground conditions at the time of construction are found to be less favourable than those foreseen at time of the site investigation. As a consequence, either shallower slopes have to be adopted (resulting in increased land-take) or the originally planned side slopes are maintained but a form of stabilisation has to be adopted
- road widening is necessary.

For these conditions, stabilisation may be achieved by soil nails, anchors or reticulated mini-piles (Figure 11.9). Other methods of stabilisation include drainage and embedded, gravity, gabion or crib walls. An account of the various methods available for slope stabilisation, together with comprehensive references and case studies is given by Hutchinson (1977). Further information is also provided by Nowak (2012b).

11.9.2 Soil nails, anchors and mini-piles

Soil nails provide a method of increasing cut-slope stability in appropriate soils and weak rocks (Figure 11.9(a)). Typically, they comprise linear steel members driven, or drilled and grouted, into the cut sides at centre to centre spacings typically from 1 to 2 m. Soil nails are installed top-down by successive small cuts, usually of the order of 2 m or such depths as are necessary to maintain stability. Construction employs the following sequence: excavation, nail installation, face construction and (where necessary) drainage. Unlike anchors (discussed below), there is usually no need to provide a structural retaining face because no significant compressive stress is introduced into the soil (except to ensure that the facing is pressed hard against the natural ground). All that is normally necessary is a light construction such as shotcrete or fabric seeded with vegetation to prevent soil ravelling about the face plates.

Figure 11.9 Approaches to stabilising cut slopes: (a) soil nails; (b) anchors; (c) reticulated mini-piles

Nails are likely to be most successful in over-consolidated clays, shales, mudstones and other weak or weathered rocks. Their use with normally consolidated clays or loose granular soils would require careful consideration. Although functioning at low stress levels, an environment not normally favourable for corrosion, it is usual in Britain to provide some form of corrosion protection, such as plastic sheathing to the steel and at the connections to the facing, when used in permanent as opposed to temporary works. A best practice guide for soil nailing is provided by Phear *et al.* (2005) and design by BS 8006-2:2011 (BSI, 2011). Further information about soil nail design is provided by Whitbread (2012) and construction by Ball and Gavins (2012) and BS EN 14490 (BSI, 2010a).

Anchors (Figure 11.9(b)) comprise corrosion-protected steel cables drilled and grouted into the ground, each comprising a bonded (or grouted) length, a free tendon length and an anchor head. They are stressed and consequently introduce significant compressive forces that have to be reacted against; this is normally done by a capping beam or beams located at the cut face. Installation is by the same top-down procedure as soil nails, although given the time taken in stressing the cables and forming the capping beams, progress may be slower. Anchors are applicable to a similar range of soil and weak or weathered rock types as soil nails. The principal difficulty with anchors is estimating the bond between the grouted length of anchor and the surrounding soil; each anchor has to be comprehensively tested on installation by applying 150% of design load for permanent and 125% for temporary installations, with displacements monitored (BS 8081:1989 (BSI, 1989)). Information on ground anchor construction is given by Judge (2012) and on rock stabilisation techniques is provided by Nicholson (2012).

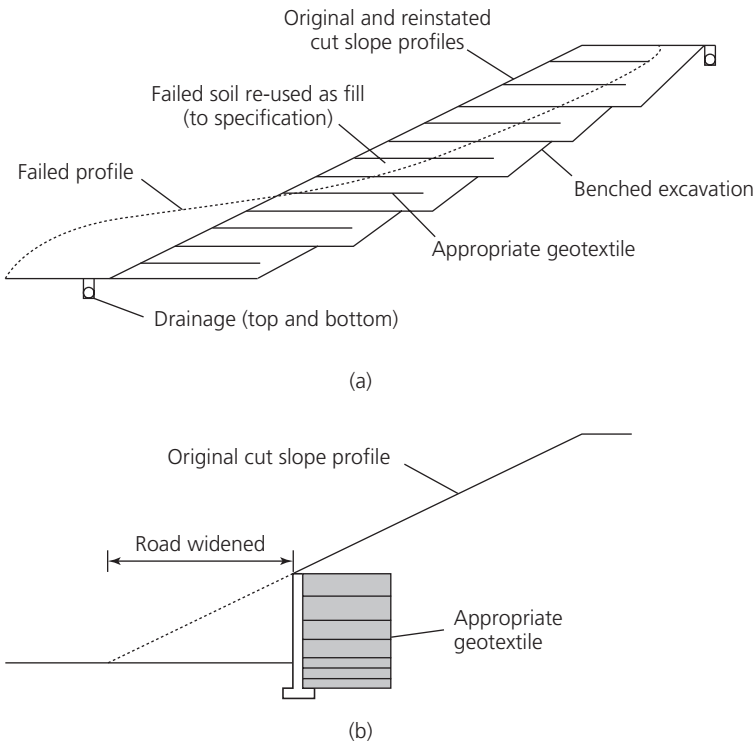
Mini-piles (pali radice) are most often employed for underpinning foundations but their use for stabilising slopes has also been reported (e.g. Ellis, 1985; Lizzi, 1984). Pile diameters vary typically up to 300 mm for foundations but those reported by Ellis for stabilisation purposes were 100 mm dia. drilled and grouted with a single 25 mm reinforcing bar. Mini-piles are normally reticulated (Figure 11.9(c)) and equipped with capping beams to cater for bending and tension. In this they differ from soil nails, which are installed without capping beams and which rely on the soil–structure interaction of the large number of nails to reinforce the potential failure zone.

11.9.3 Reinforced earth applications

There are several reinforced earth applications for failed slopes and the technique is also frequently adopted for road widening purposes. Reinforced earth is bottom-up construction and cannot be used for the stabilisation purposes already described. However, it has wide application in both cut-slope stability repair and in highway widening (Figure 11.10).

The nature of the fill is a major consideration because it directly affects the cost of the scheme. Jones (1996) reports that well-graded granular soil makes the best reinforced

Figure 11.10 Reinforced earth applications for: (a) cut slope repair; (b) road widening



earth fill and that effective stress transfer is poor with clay fill; this means that reinforcement resistance cannot be so rapidly generated and that construction rates may be slowed. BS 8006-1:2010 (BSI, 2010b) states that the best fills are those it terms ‘frictional’ and ‘cohesive frictional’ and comprise fill types 6I, 6J, 7C and 7D of the SHW (TRL, 1998). These fills can be employed for any structural category (e.g. retaining walls, embankments and bridge abutments). Cohesive fills and soft chalk (saturation water content greater than 29%) should only be employed for slopes of retained height 1.5 m or less and where failure would result in minimal damage and loss of access.

For further details Jewell (1996), Jones (1996), Manceau *et al.* (2012), Jenner (2012) and BS8006-1:2010 (BSI, 2010b) may be consulted. Calculation methods for reinforced soil and soil nails are set out in the Highways Agency Advice Note HA 68/94 (HA, 1995). The *Specification for Highway Works* (HA, 1998, 2013) also provides requirements for the method of compaction of fill materials for reinforced earth structures.

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Chapter 12

Embankments: some design and construction considerations

12.1. Introduction

Embankments are raised on natural ground in order to achieve a design formation level. Compared to the design of cuttings, the design and construction of embankments requires consideration not only of the embankments themselves (the fill), but of the engineering performance of the ground forming the foundation on which they are built. Embankments are usually constructed from remoulded materials whose engineering properties may be more difficult to predict than those of the undisturbed natural ground. Design of embankment slopes cannot be seen in isolation from the strength and settlement characteristics of the foundation: a granular fill that would perform safely at say 1 (vertical) in 1.5 (horizontal) on a hard foundation could not be placed at this slope on soft clay without the risk of excessive settlement and/or bearing capacity failure. Therefore, for this particular type of fill, it may be necessary to slacken the embankment side slopes to prevent failure of the embankment toe and/or to strengthen the founding strata to prevent overall failure or excessive settlement over the design life of the earthwork.

Limit states for embankment performance are detailed in Eurocode 7 (BSI, 2004).

Ultimate limit state failure can have been considered to have occurred if failure of the embankment construction or underlying foundation results in significant movement along a defined failure surface. This type of failure would result in reconstruction of the failed section to return the embankment to its intended use.

Serviceability limit state failure can be considered as a settlement over time, either of the embankment fill material or foundation material of sufficient total or differential magnitude to prevent operation, for example settlement of rail track or highway carriageway. This type of failure usually results in increased maintenance such as re-ballasting of railway tracks or reconstruction of highway carriageway.

12.2. Method of approach

With respect to the design of an earthworks embankment the designer should consider the following points.

- Assessment of geotechnical information: an assessment of all geotechnical information from site investigation reports should be undertaken in conjunction

with design requirements such as embankment height and width at formation level. This information will usually relate to strata comprising the embankment foundation and its geotechnical properties. It is common, however, for widening projects of existing embankments, for example motorway widening, that additional information may have been gained by ground investigation of the originally constructed embankments, either as asset assessment or for design of the widening works. In addition, many asset owners carry out asset assessments involving inspection and listing of defects since construction that form an additional source of geotechnical information.

- Assessment of fill properties: traditionally, material for engineered embankment fill would be utilised from adjacent cuttings, particularly when the earthworks scheme is in balance. If, however, the scheme is in deficit then additional fill material will have to be won from adjacent borrow pits. The nature of the fill material from adjacent cuttings will have been ascertained from the ground investigation carried out as part of the design process. On most schemes, however, the constructor will carry out additional investigation for acceptability at the commencement of the construction programme. In the case of adjacent borrow pits, or more distant fill sources, these sources may not have been defined as part of the design process and information on fill properties may not be available to the designer until the construction begins. The latter was the case for the construction of the M74 Completion project in Glasgow where no local fill sources were available and the contractor identified fill sources during the initial construction phase. Even when an earthworks scheme is in balance the availability of material from cuttings for use as embankment fill may be constricted due to barriers along haul routes such as river, road or rail crossings. In this case, bridging of these barriers may not allow efficient movement of material from cut to fill with respect to the overall construction programme, and local borrow pits may be required to be filled in with material from the main line cuttings at a later date. This was the case on the A1 (M) Motorway contract from Dishforth to Darrington in Yorkshire, where crossing of existing motorways, railways and the River Aire led to the construction of the main embankments for the interchange with the M62 motorway with locally imported unburnt colliery shale rather than site won material. On schemes where existing infrastructure is being widened, for example motorway widening, it may not be feasible within the construction programme to use material from adjacent cuttings for embankment fill as the activity is not possible to coordinate within the construction programme and no adjacent site for stockpiling is available. These contracts may also involve the import of embankment construction material and the disposal of material from cuttings in noise or environmental bunds along the scheme, as was the case with the M25 Widening design, build, finance and operate project.
- Presence of structures and services: the requirement for earthworks up to structures or where the alignment crosses exiting services may be critical with respect to differential settlement of the structure and the adjacent earthwork. Most construction contracts now contain specific criteria relating to differential settlement at this interface. In addition, settlement of new embankment

construction at crossing locations of existing services may be critical to their continued satisfactory performance if they cannot be diverted.

- Analysis of embankment sections: normally, the highest parts of the embankment would be identified and section drawings prepared. The worst case foundation conditions are usually assumed where the embankment is highest, unless there is reliable investigatory evidence to the contrary such as lower embankment heights underlain by soft ground. Other critical locations are selected, for example at structures, and section drawings produced.
- Long-term stability: long-term stability is normally investigated using effective stress methods, taking into consideration relevant strength and groundwater (or pore pressure) data. This provides the factor of safety for the permanent works condition and the calculated value may be compared with the minimum value quoted in Chapter 13. Provision may be made for drainage or other measures if the desired factor of safety is not achieved
- Short-term stability: short-term stability is investigated using total stress methods and relevant strength properties. (An alternative effective stress approach is discussed below.) This provides the factor of safety for the temporary works condition and the calculated values may also be compared with those given in Chapter 13. Undrained analysis methods would also be considered for embankments founded on compressible cohesive materials as the increased pore pressure in the founding strata is unlikely to dissipate in the short term. This condition is more likely to indicate ultimate limit state failure than long-term drained conditions when the increased pore pressure has dissipated. It is usually controlled by staged embankment construction allowing increased pore pressure to dissipate prior to placing of the next stage of fill material, hence restricting the amount of pore pressure increase.
- Likely failure mode: the likely failure mode of the engineered embankment fill can be judged against past studies. 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. Similarly, Reid and Clark (2000) also observed similar depths of failure on slopes generally greater than 4 m high. This concurs with the work of Perry (1991), where shallow failures were more critical in slopes of greater than 5 m high.

Investigation of these failures indicates that the most likely mechanism for failure is one of water 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.

- Loading: use of Eurocode 7 requires that actions be considered as part of the analysis. With respect to infrastructure these would take the form of permanent loads applied as a uniformly distributed load for rail or highway loading. Information on these loads is contained in Eurocode 1 (BSI, 2008, 2010a).
- Settlement of the embankment foundation: having established the embankment geometry, the settlement of the embankment is then normally calculated. This is usually done as a profile along the longitudinal axis. For major embankments,

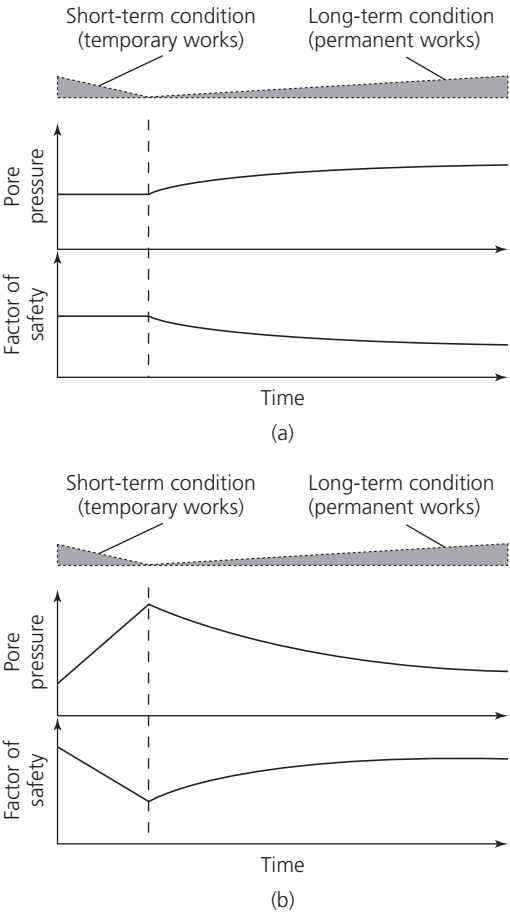
such as dams, motorways and railways, and where foundation conditions can vary significantly in a lateral direction, more than one profile should be calculated, say along the centre-line of each carriageway and at each shoulder for a road embankment. Calculations are particularly appropriate at structure locations and beneath drains to determine the possibility or otherwise of the reversal of drainage direction, as a consequence of settlement.

12.3. Stability analysis: long-term condition

12.3.1 General

Figure 12.1 illustrates the way pore pressure and factor of safety vary with time, both for the embankment fill and the underlying foundation soil. Depending on its over-consolidation ratio and the depth at the borrow from which it was taken, pore pressures

Figure 12.1 Variation of pore pressure and factor of safety with time for embankment fill and foundation: (a) embankment fill (slope); (b) embankment foundation (adapted from Bromhead, 1992)



may be low or even negative in the fill when placed on the embankment (Chandler, 1978). Over time, the pore pressures will gradually reach equilibrium with the climatic and other factors operating (Figure 12.1(a)). Therefore, the factor of safety of the fill material, which began at a relatively high level, reduces to a minimum, possibly decades after construction. In this respect, embankment fill behaves in a similar fashion to a cut slope. However, because pore pressure distribution is probably more complicated in embankments (given the different materials often employed as fill), pore pressure response may be less predictable.

Pore pressure behaviour differs completely in the underlying foundation soil. It begins to increase after the fill is first placed and reaches a maximum usually at, or shortly after, embankment construction; it then declines slowly to approximate its pre-construction value. The corresponding factor of safety decreases after the foundation is loaded to reach a minimum at, or shortly after, completion of construction and then increases slowly as pore pressure decreases (Figure 12.1(b)). Therefore, in stability terms, the embankment fill and the embankment foundation behave quite differently, reaching their minimum factors of safety at quite different times, that is, in the short term for the foundation and over the long term for the fill.

The following sets out the approach to determining the essential properties of strength, groundwater (or pore pressure) and the method of analysis that may be adopted for analysing the long-term condition.

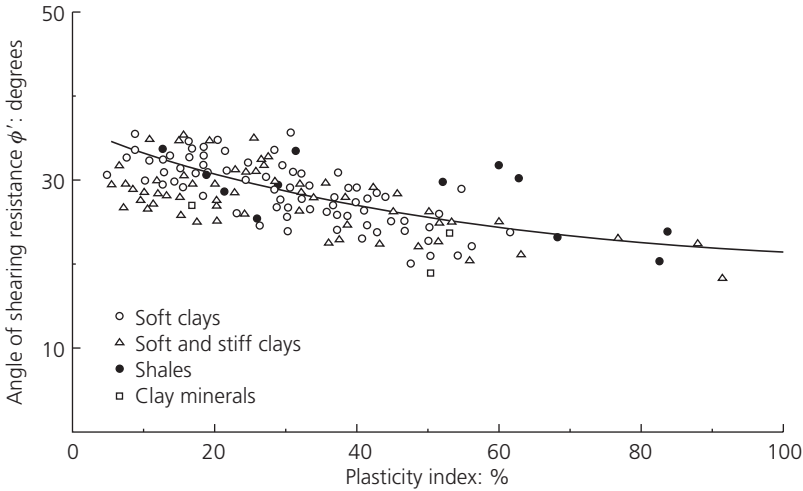
12.3.2 Strength properties

12.3.2.1 Embankment fill

Clay fill: consolidated drained or consolidated undrained triaxial tests with pore pressure measurements may be made on clay fill samples, tested against a back pressure to ensure that any air present in the fill remains dissolved in the pore water. The water contents selected for the tests will normally correspond to the designer's best estimate of the equilibrium water content of the fill material in the permanent works condition. This could be the water content of the fill as received from the borrow, or the optimum water content if the fill is to be conditioned to this or to a related value (such as optimum water content plus or minus 1%). Bishop and Henkel (1962) showed that c' for compacted clay fill reduced rapidly as the initial sample water content increased, ϕ' being relatively little affected; low c' values may therefore be expected for clay fill placed wet of optimum water content.

Clay fill placed relatively dry may 'wet-up' and soften over a long period of time (refer to Chapter 8). Monitoring a 7 m high embankment formed from Gault Clay, Crabb and West (1986) found that the sides had wetted up (from rainfall) as had the foundation (from the underlying groundwater table). They also observed that the toe of the embankment was wettest and considered that failures were likely to originate at this location. These observations were made only some 15 months after construction and demonstrate how rapidly clay fill softening can occur. Bishop and Henkel (1962) describe the results of strength tests on compacted clay in which water was allowed to circulate in the sample to investigate softening effects. The authors drew attention to the substantial reduction in c'

Figure 12.2 Relationship between angle of shearing resistance ϕ' and plasticity index (from Terzaghi *et al.*, 1996)



for the softened sample (about 50% of the ‘unsoftened’ value), although ϕ' remained virtually the same in both cases. This argues strongly for suitably low c' values for design, whenever a significant increase in fill water content is anticipated.

Given the above factors, it is difficult to offer any but the most general suggestions on strength property selection for clay fill. Fully softened or critical state strengths ($c' = 0$) may apply at the low effective stresses operating on the shallow depth failure surfaces typical of many embankments (e.g. Crabb and Atkinson, 1991) but for deeper failure surfaces, use of a c' value seems appropriate, its size depending on the results of tests made at the expected long-term water contents. Nevertheless, Skempton and Coats (1985a, 1985b) and Penman (1995) also adopted critical state strengths for deep failure surfaces in fill, when analysing the Carsington embankment dam failure.

A relationship between the peak angle of shearing resistance ϕ' and plasticity index is given in Figure 12.2. Critical state and peak angles of shearing resistance are normally similar in fine soils (although the cohesion intercept is zero for the critical state strength). Hence, and as a first approximation, ϕ' values obtained from the figure may provide some information on critical state angles of shearing resistance ϕ'_{cs} .

Granular fill: the strength properties of granular fill may be determined using the procedures suggested in Section 4.3.3. As granular fill is usually free draining, methods of test involving pore pressure measurements are not normally appropriate. However, only a relatively small proportion of fines (silts and clays) is required substantially to reduce a granular fill’s permeability to the point that pore pressures are set up at failure. Penman (1971) and Penman and Charles (1976) suggested that soil or fill with permeabilities

greater than about 10^{-5} m/s would not exhibit excess pore pressures: this suggests in turn that permeability tests on samples compacted to expected water content and density should be employed to determine whether or not the assumption of free draining fill is justified.

Multiple fills: here the designer is faced with the difficulty of deciding which fill, or combination, will govern the shear strength of a given section of the embankment. The following design approaches are suggested.

- Design on the basis of the parameters of the weakest and most compressible fill available. This is normally a sensible alternative when say 80% or more of the total fill comprises one type only; minor fill constituents can be placed in the core of the embankment and near the top, where they would usually have least effect on stability or settlement.
- Where several fill types are in contention, each having a good chance of being employed at any given chainage, use the parameters appropriate to the weakest and most compressible. This is clearly conservative but the alternative sometimes adopted, which is to attempt to mix fills, is unsound: first, because it is often very difficult to mix fills in situ with a bulldozer blade, nor is it often desirable (see Section 3.3.2); second, the proportions used in the mixtures made up in the laboratory for shear strength and other tests to support this method are often no more than guesswork, as far as the actual conditions on site are concerned.
- Another option where several fill types are in contention is to design on the basis of the weighted average shear strength (and other relevant engineering properties such as unit weight). Suppose, as an example, ϕ'_1 to ϕ'_4 were the angles of shearing resistance of four fills having volumes V_1 to V_4 , the design weighted average angle of shearing resistance would be

$$\phi' = \frac{\phi'_1 V_1 + \phi'_2 V_2 + \phi'_3 V_3 + \phi'_4 V_4}{V_1 + V_2 + V_3 + V_4} \quad (12.1)$$

A similar procedure could be adopted for estimating a design cohesion intercept value, c' , assuming one were to be employed, or an undrained shear strength. Note that it is only the fill placed in the potential slip zone that has a bearing on stability and that should be used in calculating the weighted design value; it would be pointless taking into account fill placed well away from any potential failure surfaces.

- Where a significant amount of wet clay fill is to be employed, the designer may specify the chainages between which it should be placed. He may then be free to adopt whatever parameters or indeed conditioning he thinks fit over the specified length. However, this approach may produce difficulties for the programme if the contractor is constrained to place one particular fill at one particular location.

The above procedures may be adopted with fill(s) tested both in effective or total stress terms.

12.3.2.2 Foundation

The effective shear strength of clay foundation soil may also be measured using a consolidated drained or consolidated undrained triaxial test with pore pressure measurements, on representative samples extracted from the natural ground.

There is a major difference between the behaviour of over-consolidated clay after cut formation and during embankment loading. Cut formation unloads the soil, opening any fissures and other discontinuities present, while raising an embankment loads the foundation soil. Fissures are closed, thus reducing permeability. A reduction in shear strength to the fully softened or critical state value is unlikely. Selection of shear strengths for an embankment in over-consolidated clay consequently differs from a cut formed in the same material. The following are offered as suggestions for strength parameter selection for first time failures involving the foundation soil:

- for over-consolidated clay, use a conservative assessment of the data
- for normally or lightly over-consolidated clay, adopt a zero effective cohesion c' together with a conservative assessment of the data.

For failures involving reactivation of ancient failure surfaces in foundation soil:

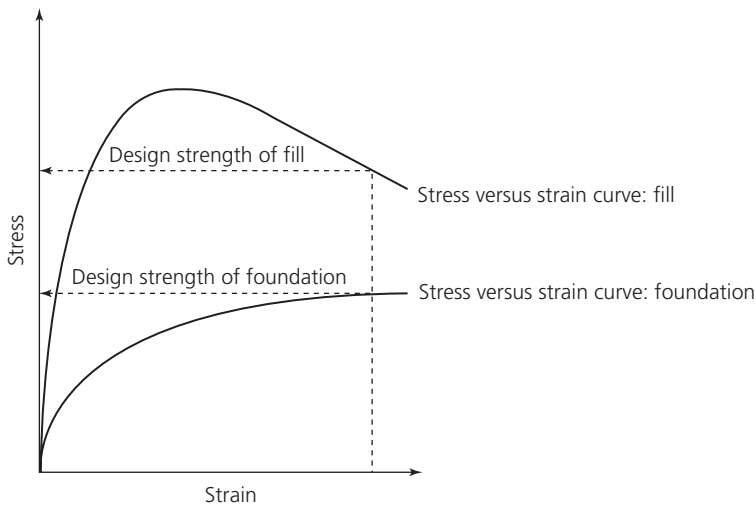
- adopt the measured residual angle of shearing resistance ϕ'_r with a c'_r of zero. Note that if measured using the ring shear apparatus, ϕ'_r may be slightly conservative (see Section 11.3.2) which should be born in mind when assessing the factor of safety.

Relationships between the residual angle of shearing resistance (ϕ'_r), the plasticity index and clay fraction are shown in Figure 11.3 (Chapter 11).

12.3.2.3 Mobilisation of fill and foundation strengths

A matter requiring judgement is the extent to which the maximum shear strengths of the embankment fill and the underlying foundation can be considered to act at the same time, should failure affect both the embankment and the underlying foundation. Remembering that the embankment fill is remoulded, it is likely that during first time failure, the maximum shear strength will be mobilised at a quite different strain from that of the underlying natural soil foundation. For this reason, some engineers prefer to ignore the shear strength component offered by the embankment fill when designing for first time failure. This procedure is conservative but it may not be unduly so if a deep tension crack forms in the fill. Furthermore, if applied, it permits a lower factor of safety to be employed than would otherwise be the case.

If this approach is considered too conservative, some estimate of the maximum shear strength mobilised by the embankment fill and the underlying foundation can be made, assuming that the majority of the failure surface is in the foundation. This is done by comparing the proportion of maximum fill strength mobilised when the foundation soil is at its peak strength. The shear strength operating along the presumed slip surface in the fill can be adjusted accordingly (Figure 12.3). Where ancient failure surfaces are present

Figure 12.3 Selection of design strengths for embankment fill and foundation soil

in the foundation, many designers would employ peak shear strength for the fill because the residual shear strength acting along the ancient failure surface would be mobilised rapidly and is unlikely to vary significantly with strain.

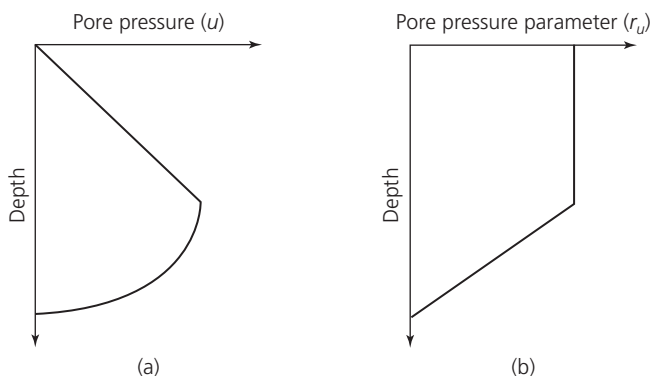
12.3.3 Groundwater and pore pressure conditions

12.3.3.1 Pore pressure in embankment fill

Pore pressure behaviour in embankment fill and the corresponding changes in factors of safety were outlined in Section 12.3.1. Superimposed on the relatively simple model of gradually increasing pore pressures with time, measurements have indicated rapid fluctuations in pore pressure to depths of about 2 m (Vaughan *et al.*, 1978); below this depth, and to about 5 m, they were considered to be seasonal and measurable by conventional Casagrande piezometers. Perry (1991) noted that while fluctuating positive pore pressures could be generated at shallow depth, high suctions (negative pore pressures) existed at the embankment core. This suggests that at least for the higher and larger embankments, equilibration of the suctions could take many years, as found for cuttings by Skempton (1977), Chandler (1984) and discussed in Section 11.3.3. Crabb and West (1986) found that while the side slopes wetted up as did the base, the pore pressures in the core of their 7 m high embankment were far from equilibrium in the period monitored (about 15 months). See also the discussion in Section 12.3.2 (clay fill).

Tension cracks are a frequently observed feature of clay fill and work by Anderson and Kneale (1982) on an embankment formed from over-consolidated Oxford Clay demonstrated that, once formed, tension cracks served to channel water into the embankment, effectively increasing the permeability of the clay fill. Anderson and Kneale (1982) suggest that tension cracks could have two effects: first, by increasing permeability they could help trigger slides (albeit of shallow depth) following periods of heavy rain; second, the enhanced permeability could increase the rate of pore pressure redistribution, leading

Figure 12.4 Diagrammatic representation of pore pressure distribution for embankment slope: (a) pore pressure distribution in embankment slopes; (b) pore pressure parameter r_u distribution in embankment slopes (from Jewell, 1996), reproduced by kind permission from CIRIA



to more rapid softening and the achievement of critical state strength (see Section 11.3.2). Interestingly, the authors found that the location of the tension cracks depended on topsoil depth and vegetation cover, being least frequent when both topsoil and vegetation were thickest. This suggests that such means as top soiling and planting not only increase amenity but also assist in reducing shrinkage or desiccation cracking, thereby reducing the possibility of shallow depth slides. The expression for depth of tension crack is given in Equation 11.3, Chapter 11, and could be used in the calculation of stability of embankment slopes where desiccation cracking is known to have occurred.

A method of approach to pore pressure distribution in embankment clay fill was advanced by Jewell (1996). An increase in positive pore pressure was suggested to about 1.5 m depth below embankment side slope, below which the pore pressure reduced to zero at about 2.5 m deep, as illustrated in Figure 12.4. (Note, as pointed out by Jewell, the depths in Vaughan *et al.* (1978a, 1978b) are somewhat different, possibly because these authors were considering shallower sided embankment dam slopes rather than steeper highway embankments.) Drawing on data by Atkinson and Farrar (1985) and Anderson and Kneale (1980), Jewell (1996) recommends using an r_u value of 0.2 to 1.5 m depth and a value varying linearly to zero from 1.5 to 2.5 m.

12.3.3.2 Pore pressure or groundwater in foundation

Some estimate has to be made of the groundwater or pore pressure conditions in the embankment foundation. Normally, it is assumed that the pore pressure or groundwater changes caused during the construction of the embankment will revert to pre-construction levels, over the long term, permitting the use of site investigation data. However, it should be remembered that construction of the embankment may itself cause changes in the groundwater table; flooding on the upstream side of an embankment built on side-long ground is a possibility. For this reason, most designers take a conservative view of site investigation piezometer information, particularly if they are limited to no more than a wet season's data.

There are some occasions when foundation pore pressures need to be measured during embankment construction. A case in point is when the embankment is being constructed on compressible clay, and stability monitored in term of effective stress. More details are given in Section 12.4.4.

12.3.4 Analysis

12.3.4.1 Embankment fill

Where no features likely to constrain a failure surface are present in fill, the Bishop or Morgenstern methods will normally be appropriate. The infinite slope or wedge analysis (sliding block) methods may also be appropriate. Experience indicates that the majority of failure surfaces that affect embankment and cut slopes are shallow, so shallow depth potential failure surfaces should be investigated in any embankment slope stability analysis. It should be noted that compaction-induced shear surfaces could align failure surfaces (see Section 3.4.5).

12.3.4.2 Embankment fill and foundation

Assuming that there are no features in the foundation soil, the Bishop or Morgenstern methods may be employed, extending through the embankment into the underlying foundation. If such features exist, a wedge analysis or a non-circular failure surface should be adopted, extending through the fill, with part of the surface constrained to align with the feature (Figure 11.4, Chapter 11).

12.3.5 Some results based on British motorway experience

The work of Perry (1989) concerning earthworks for English and Welsh motorways was introduced in Section 11.3.5, where the performance of some cut slopes was examined. Information on a number of embankments taken from his report is given below; a height range from 2.5 to 5 m was selected because embankments less than 5 m high provided some 81% of the sample; clays, particularly the over-consolidated variety, provided most problems and are included (see Table 12.1).

Table 12.1 Performance of British motorway embankments from 2.5 to 5 m high (from Perry, 1989)

Geological stratum	Age of construction: years	Length surveyed: m	Slope angle: v : h	Percentage failure
Glacial Till	7	1307	1 : 2.5	0
	16	392	1 : 2.5	0
	22	311	1 : 2.5	1.5
London Clay	5.6	625	1 : 2	14.6
	10	9499	1 : 2	6.2
Oxford Clay	10	1191	1 : 2	23
	22	1500	1 : 2	7.1
Lower Lias	13	2402	1 : 2	1.7
	25	694	1 : 2	12.6

It is immediately evident that the embankment side slopes are relatively steep with 1 in 2 being selected for all but the Glacial Till. It is, therefore, no surprise that the incidence of failure is relatively high for the over-consolidated clays (London, Oxford and Lias), with percentage failure varying up to 23%. However, it is, at first sight, puzzling that for the London and Oxford Clays the percentage failure is higher for the younger than for the older embankments. This is quite unlike the situation existing for the cut slopes discussed in Section 11.3.5, where not only was the incidence of failure far lower (usually below 5%) but the younger cuts displayed a lower incidence of failure than the older, as would be expected. It suggests that in the case of the younger embankments reported by Perry, the negative pore pressures (suctions) existing in the fill at placement rapidly dissipated as water entered the embankments and c' quickly reduced towards the fully softened or critical state value. The embankments failed as a consequence.

It is of interest to estimate the r_u value existing at failure for these younger embankment slopes. Perry confirms that the failure surfaces were of shallow depth (see Section 11.3.5) and consequently were unlikely to have penetrated the foundation soil. Assuming an average embankment height of 3.75 m and that the effective cohesion has reduced to 3.5 kN/m², there is a $c'/\gamma H$ ratio of 0.05 for a unit weight of 18 kN/m³. If an angle of shearing resistance ϕ' of say 22.5° is assumed for the fills, this calculation yields an r_u value of 0.42 at failure. Thus, for the younger embankment examples concerned, r_u may have increased from a possibly negative value at the time of construction (depending on the over-consolidation ratio of the clay fill and its depth at the borrow pit) to the relatively high value of 0.42 in a period between 5 and 10 years.

Matters concerning construction and drainage are discussed further in Sections 12.7 and 12.8.

12.4. Stability analysis: short-term condition

12.4.1 General

As illustrated in Section 12.3.1, for the foundation soil beneath the embankment, the factor of safety against failure is likely to be at a minimum at or shortly after the end of construction (the 'short-term condition'). As embankment construction normally lasts only weeks or months, a foundation soil comprising compressible cohesive material remains undrained over this period, that is, there is no change in water content between the commencement and end of embankment construction. This is the assumption behind the undrained or total stress method of analysis for the short-term condition.

In using this approach an increase in pore pressure in the founding soil should also be considered as it is likely that this elevated pore pressure will dissipate slowly due to the low permeability of the soil. Aspects of the increase of pore pressure during embankment construction are discussed by Tavenas and Leroueil (1980).

12.4.2 Strength properties

12.4.2.1 Embankment fill

As discussed in Section 3.3.2, Vaughan (1978a) and Vaughan *et al.* (1978) recognise that for clay of high plasticity (liquid limit in excess of 50%) and of low plasticity (liquid limit

less than 50%), both materials exhibit substantially different properties in undrained shear.

High plasticity clay fill should be tested slowly to allow pore pressures to redistribute; Vaughan *et al.* (1978) mention a rate of strain for the slow tests of 0.05%/hour. When tested in this way, an undrained shear strength some 20% less than the strength measured in conventional undrained tests results. The method of sample preparation is also critical for high plasticity clay. The authors consider the best method would be to extract samples from a trial embankment, rather than by testing specimens dynamically compacted in the laboratory using BS rammer procedures, because of the completely different (and unrepresentative) compacted structure that results. However, the sampling and testing procedures described above would put severe constraints on many design and construction programmes and it might be worthwhile considering testing samples statically compacted in the laboratory, testing in the conventional manner, and decreasing the measured strength by a factor of 0.8 to account for the 20% reduction referred to.

Vaughan *et al.* (1978) believe that the compacted structure of a low plasticity clay fill has a negligible effect on the undrained strength and that the conventional undrained triaxial test provides a reliable indication of shear strength for design purposes.

In Section 12.3.2 there is a discussion on the ways of selecting strength properties for design purposes when several fills are involved.

12.4.3 Analysis

12.4.3.1 Embankment fill

As with the analysis for the effective stress approach (Section 12.3.4), the Bishop or Morgenstern methods will normally be appropriate. The infinite slope or wedge analysis (sliding block) methods may also be appropriate.

12.4.3.2 Embankment fill and foundation

Assuming that there are no features in the foundation soil, the Bishop or Morgenstern methods may be employed, extending through the embankment into the underlying foundation. If such features exist, a wedge analysis or non-circular failure surface should be adopted extending through the fill, with part of the surface constrained to align with the feature (Figure 11.4, Chapter 11).

Resulting factors of safety (including those from the calculations made for the embankment fill) may be compared with those quoted as minimum values in Chapter 13.

12.4.4 The short-term condition in terms of effective stress

It is possible to conduct an analysis of the short-term condition in terms of effective rather than total stress, should this be desired. The principal problem with this method is estimating the magnitude of the pore pressure in a clay foundation soil as the embankment is raised. A guide may be obtained by undertaking laboratory pore pressure

dissipation tests (details from Head, 1994), which will indicate the magnitude of the pore pressure rise for each increment in vertical effective stress, as the embankment is constructed. Stability analyses may then be made, in effective stress terms, and the factors of safety calculated for the predicted foundation pore pressures. If it is found that the predicted pore pressure rise is such that the factor of safety would fall to an unacceptably low level, then construction would have to be slowed or halted until pore pressures have dissipated sufficiently to permit construction to proceed safely.

However, it would be most unwise to accept these predictions at face value and construction should only take place with comprehensive monitoring of the foundation pore pressures. Casagrande piezometers are unsuitable in low permeability clay soils (see Section 11.3.3), vibrating wire, hydraulic or pneumatic piezometers should be installed for pore pressure measurement. To avoid damage during embankment construction, the piezometer leads will have to be buried in trenches exiting to appropriately sited gauge houses, away from the filled area. Monitoring of displacements is also highly recommended because it provides further information on embankment stability. Using this information, the accuracy or otherwise of the laboratory test-based pore pressure predictions can then be monitored and the rate of construction adjusted to ensure that factors of safety are always sufficiently high. Many engineers prefer this approach to the total stress method because of the opportunity it gives to monitor progress. It is of course possible to undertake a parallel total stress analysis, if resources permit.

Unacceptably high pore pressures can also be set up in embankment fill, particularly those constructed from low plasticity clay (Vaughan *et al.*, 1978). The same instrumented approach can be adopted in these cases, although note that pneumatic piezometers are not recommended for partially saturated soil (as often encountered with fills) or with gas-generating organic materials (such as might be encountered in some soft compressible soils).

12.5. Settlement

12.5.1 General

Settlement estimates are particularly important where the embankment is in contact with structures; for example, culverts or bridge abutments, where there are marked differences in foundation soils; for example, where Terrace Gravels give way to more compressible Alluvium or where settlement may compromise the intended use of an earthwork.

With respect to traffic loads BS 6031:2009 (BSI, 2009) comments that these 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 to take them into account.

The settlement of the structure at formation level (the top of the embankment) is the sum of the settlement of the foundation soil and the overlying embankment fill itself. Settlement estimates for clay foundation soils take into account primary settlement (ρ_c) and, for some clays, secondary compression (ρ'_c). To this is added immediate or 'elastic' settlement (ρ_i), which is complete at the end of construction. Total settlement

is therefore

$$\rho_t = \rho_i + \rho_c + \rho'_c \quad (12.2)$$

However, embankments are built to an elevation. As noted, immediate settlement is complete at the end of construction, so it is normally possible to 'build out' the immediate settlement by topping up the embankment with fill. This also applies to embankments on granular foundations, the settlements of which are normally complete at the end of construction due to their high permeability and rapid dissipation subsequent to loading. In the case of embankments, therefore, it is usually possible to ignore immediate settlements. Total settlements for most forms of embankment construction on clays therefore become

$$\rho_t = \rho_c + \rho'_c \quad (12.3)$$

The following concentrates on a primary and secondary settlements of clay soils and fill.

12.5.2 Primary settlement

Compressibility: As noted in Section 11.3.1, it is unlikely that many terrestrial deposits in British conditions are truly normally consolidated and most softer deposits are lightly over-consolidated to some degree. The critical property for settlement measurement in lightly over-consolidated soils is the pre-consolidation pressure (p_c). Defined as the maximum vertical effective pressure to which a sediment has been subjected in the past, it controls not only the magnitude but also the rate of settlement. While a most important property in settlement estimates, it is critically dependent on sample disturbance, and graphical techniques to allow for such disturbance are provided in the standard texts. A good discussion of pre-consolidation pressure is given by Simons (1975).

Where the current in situ effective pressure p_o is less than p_c , and provided that the increment of applied stress Δp due to the embankment load is such that

$$p_o = \Delta p < p_c \quad (12.4)$$

settlement of the clay stratum will be small. On the other hand, where

$$p_o + \Delta p > p_c \quad (12.5)$$

substantially greater settlements occur. This is because of the marked difference in compressibility either side of the pre-consolidation pressure, with the compressibility being much larger in stress ranges in excess of p_c , as it mirrors the normally consolidated (virgin compression) line.

For the case where Equation 12.4 applies, the primary settlement of a clay of thickness H having a compression index C_{c1} under an increment of embankment load Δp is

$$\rho = \frac{C_{c1}}{1 + e_o} H \log \left\{ \frac{p_o + \Delta p}{p_o} \right\} (m) \quad (12.6)$$

where e_o is the initial void ratio. Where Equation 12.5 applies, the compression index for $p_o < p_c$ is C_{c1} and for $p_o > p_c$ is C_{c2} . For this case

$$\rho = \frac{C_{c1}}{1 + e_o} H \log \frac{p_c}{p_o} + \frac{C_{c2}}{1 + e_o} H \log \left\{ \frac{p_o + \Delta p}{p_c} \right\} (m) \quad (12.7)$$

Schmertmann (1953) suggested that C_{c1} is equal to the coefficient of swelling C_s . For the more heavily over-consolidated British clays of Tertiary age and older, where the current in situ effective overburden pressure differs greatly from the pre-consolidation pressure, most engineers prefer to work with the coefficient of volume compressibility m_v , rather than the compression index. Where

$$p = H \times m_v \times \Delta p \quad (12.8)$$

Alternatively, an approach using Young's modulus (E') can be employed where

$$p = \Delta p \times I \times H/E' \quad (12.9)$$

and I is the influence factor, H is the thickness of the compressible layer.

If the m_v or E' approach is used the settlement calculated is total settlement (immediate and consolidation). For cohesive soils these should be separately defined. A suitable approach is provided by Padfield and Sharrock (1983).

Compressibility characteristics of clay foundation soil may be determined using the conventional laboratory oedometer test (BS 1377:1990: Part 5: Test 3 (BSI, 1990)) made on samples extracted from boreholes or trial pits. For most fill, particularly those containing coarse sand or gravel, compressibility characteristics may be obtained using larger diameter hydraulic (Rowe) cells (BS 1377:1990: Part 6: Test 3).

In considering settlement of the founding strata below embankment construction it should be borne in mind that the applied embankment load at founding level reduces with depth. 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 Oosterberg and presented by Leroueil (1990).

Rate of settlement: the rate of settlement is governed by the coefficient of consolidation c_v . Like compressibility properties, it is also critically dependent on the pre-consolidation pressure p_c . The expression for c_v is

$$c_v = \frac{k}{m_v \gamma_w} (m)^2 (s) \quad (12.10)$$

where k is the coefficient of permeability and γ_w the unit weight of water. When a soil sample is loaded through the pre-consolidation pressure, k does not change but the coefficient of volume corresponding to m_v increases substantially leading to a sharp

reduction in c_v . As before, it is essential when measuring c_v to select the appropriate stress range, which will normally be the increment of applied embankment load Δp in excess of the current in situ effective overburden pressure p_o . As indicated in BS 1377:1990 (BSI, 1990), the coefficient of consolidation may be derived using either the square root of time or the logarithm of time method. In a study of settlement and pore pressure dissipation of an embankment constructed from London Clay, Farrar (1978) found that the use of the logarithm of time method for deriving the coefficient of consolidation c_v was much more accurate than the alternative square root of time method. The possibility exists that this method will also be more accurate in the case of other over-consolidated clay fill.

The coefficient of consolidation is also particularly sensitive to sample disturbance and also sample size: continuous silt or sand layers that could have a substantial effect on drainage could be missed when sampling, or even when selecting the test specimen from the sample itself. For this reason, many engineers prefer to measure the permeability in situ and to use it, together with laboratory derived values of m_v and Equation 12.10 to obtain c_v .

According to Davis and Poulos (1972), the assumption of one-dimensional consolidation in the embankment foundation is satisfactory where the foundation thickness is less than the embankment width; in these circumstances, the results of conventional oedometer tests will be relevant. For foundation thicknesses that are large in comparison to embankment widths or where cohesive deposits are stratified with sand/silt layers, three-dimensional flow (i.e. vertical and horizontal) becomes important and the results of oedometer tests are less relevant. Use of hydraulic cells that permit radial drainage should be considered in such cases.

12.5.3 Secondary compression

The advent of public-private partnership (PPP) 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 break down and deterioration of the organic structure occurs subsequent to the realisation of primary consolidation.

Secondary consolidation, which occurs independently of loading conditions, can also occur in the form of creep in cohesive materials; particularly in the case of normally and lightly over-consolidated clays where applied load is greater than the pre-consolidation pressure. For over-consolidated clays the pre-consolidation pressure is not usually exceeded and loading is applied on the shallow gradient (re-compression) part of the consolidation curve with the result of very small settlements.

Secondary consolidation can be determined as

$$\rho'_c = C_\alpha \times H \times \log \left(\frac{t_1}{t_o} \right) \quad (12.11)$$

Where C_α is the coefficient of secondary consolidation; H is the thickness of layer (m); t_o is the time for 95% primary consolidation; t_1 is the time after the start of embankment construction.

C_α can be derived from natural water content after the approach of Mesri (1973). It can also be derived as a ratio with C_c , compression index. Mesri and Feng (1991) and Mesri and Godlewski (1977) report C_c/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, 6 months rather than 5 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 of 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).

12.5.4 Embankment self-settlement

The self-settlement of embankment fill, like that of the founding material, is dependent on fill type. The total self-weight settlement of fill can be calculated from

$$p_i = 0.5 \times \frac{\gamma H}{D^*} \quad (12.12)$$

Where γ is the bulk density of fill material (kN/m^3); H is the total height of embankment fill (m); D^* is the equivalent constrained modulus.

Charles (1993) gives values of D^* for a 10 m 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.

12.5.5 Negative skin friction at structure locations

The possibility of negative skin friction acting on the back of bridge abutment walls should always be kept in mind, especially where the abutments are supported by piles and the adjoining embankment fill is placed on compressible soil. Because of the support offered by the piles, the abutments may not settle by the same amount as the

embankment fill and load may be shed from the fill onto the abutment walls, possibly overstressing the supporting piles unless they were designed for this condition.

In addition, where embankments are constructed on compressible material directly behind piled structures, lateral squeezing may induce additional bending moments and shear into the piled foundations. A method of determining these additional forces is presented by Springman and Bolton (1990).

12.6. Effect of shallow depth features on stability

12.6.1 General

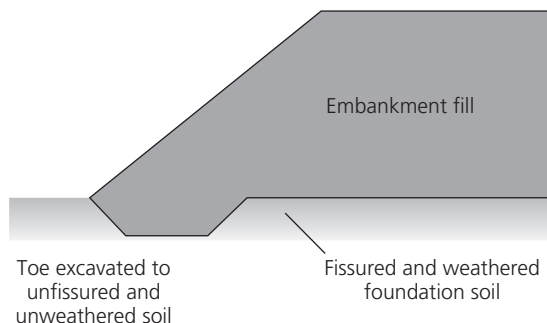
A brief description of some of the more common shallow depth features and their effect on the stability of cut slopes was given in Section 11.5. In the following sections, examples of these features are given in connection with embankments, together with examples of possible ground improvement techniques. A good discussion of various ground improvement techniques for embankment foundations, together with references is given by O'Riordan and Seaman (1994).

12.6.2 Solifluction deposits and ancient failure surfaces

Chandler *et al.* (1973) described four case records of failures involving embankments built on ancient failure surfaces contained within former landslips or solifluction sheets. Three of the four records showed delayed failure. The embankments were constructed safely but probably in the summer months when pore pressures on the ancient failure surfaces were low or negative. With time, after construction, and as the pore pressures on these surfaces increased due to seasonal effects, failure occurred. These records illustrate not only the importance of assigning correct residual shear strength ϕ'_r values to the ancient failure surfaces, but also the importance of adopting realistic pore water pressures for the foundation soils, that is, those that reflect realistic permanent works conditions.

Where soils in which fissures, foliations and ancient failure surfaces are common at or near the surface, the 'toe dig-out' has been proposed (Garrett and Wale, 1985), illustrated in Figure 12.5. The foundation soils at the embankment toe are excavated until stiff and intact materials are met and the excavated soils replaced by well-compacted granular soil

Figure 12.5 Example of toe dig-out (from Garrett and Wale, 1985)



or by other suitable materials. The depth of the toe dig-out and its lateral extent will be determined by the lateral resistance necessary to support the embankment slope.

12.6.3 Soft compressible soil

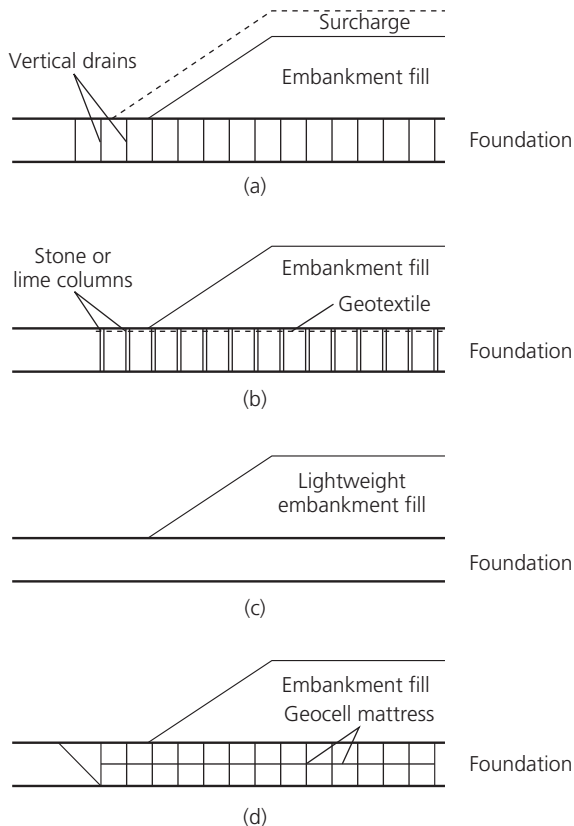
12.6.3.1 General

In soft compressible soil, embankment foundation stability may be enhanced by one or more of five principal methods:

- pre-loading or surcharging
- drainage
- structural elements
- excavation and replacement
- use of lightweight fill.

Some examples of these construction expedients are given in Figure 12.6.

Figure 12.6 Examples of various construction expedients to secure a soft clay foundation
(a) vertical drains with surcharge; (b) stone or lime columns with geotextile; (c) use of lightweight fill (PFA or polyester); (d) excavate, replace by compacted fill and geocell mattress



12.6.3.2 Pre-loading and surcharging

Pre-loading involves raising of the embankment early in the construction programme so that the underlying foundation soil can undergo primary consolidation for the longest possible period during construction. The effective strength of the soil increases as the elevated pore pressure, due to embankment loading, dissipates. The realisation of primary consolidation during construction also reduces the amount of settlement experienced during operation.

On the South Fraser Perimeter Road project south of Vancouver, Canada, constructed between 2010 and 2013, some 70% of the route corridor was underlain by up to 15 m of peat and normally consolidated clay and silt. The client, Province of British Columbia, placed as much fill, comprising land-based or dredged river sand, as possible in 2008 and 2009 at the beginning of the tender period for the scheme. This was up to 2 years prior to the start of construction and allowed the constructor to realise primary consolidation settlement during the construction period without extensive use of ground treatment or drainage measures.

Surcharging is the construction of the embankment to a height above its design level and removing the excess fill as late as possible in the construction programme. This will increase the effective strength of the soil and reduce the amount of settlement during operation. It will also achieve primary consolidation in a shorter time than under the design embankment loading only. On weak soils, checks should be made that the additional load due to the surcharge will not fail the embankment.

A combination of both pre-loading and surcharging is sometimes employed. A method of calculating the surcharge necessary to eliminate primary consolidation under working load is given by Johnson (1970).

12.6.3.3 Drainage

Construction of vertical drains, either as sand filled columns or corrugated plastic strips wrapped in geofabric, prior to embankment construction effect more rapidly the realisation of primary consolidation under applied embankment loading, compared with no treatment of the founding stratum. The solution does not increase the structural capacity of the founding stratum by its inclusion but significantly reduces the drainage path for dissipation of elevated pore pressure. The vertical drains are usually installed at 1.0–2.5 m centres depending on the c_v value of the founding stratum and the period over which primary consolidation is required to be complete. The drainage distance between the vertical drains is usually considerably shorter than the thickness of the founding stratum and two way drainage is effected. A similar effect is produced by the use of vibro-stone columns, which also provide a structural element. The use of vertical drainage incorporates the use of a granular drainage blanket as an embankment starter layer. The technique is often adopted with pre-loading or surcharging (Figure 12.6).

The accelerated realisation of primary consolidation, while removing primary consolidation during operation, will accelerate the onset of secondary consolidation. This becomes important with the use of the public–private partnership contract as concession periods

are up to 30 years and the responsibility for any maintenance due to secondary consolidation will rest with the concessionaire.

12.6.3.4 Reinforcement

Reinforcement, in the form of geogrids or geocells, can be incorporated into the basal layers of embankment construction to provide additional tensile strength (Figure 12.6). This is particularly applicable to prevent failure occurring at the toe of embankments and also to smooth out settlement across the embankment profile.

Geogrids can also be used to strengthen embankment side slopes where the land available for embankment construction is less than that required to form a stable slope. This is particularly applicable to widening of existing infrastructure where the land available is only that within the owner's boundaries.

Both basal and side slope reinforcement should be designed to BS 8006:2010 (BSI, 2010b). Information on the design of basal and slope reinforcement is provided by Manceau *et al.* (2012).

12.6.3.5 Structural elements

The construction of structural elements in the founding stratum can be considered as two types:

- methods of strengthening the soil
- methods of transferring the applied load to a more competent stratum.

The use of vibro-stone columns or soil mix columns produces a block of soil of greater strength than the existing ground and is usually installed on a grid pattern over the footprint of the embankment. The columns 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, usually 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 increase 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, for example Chalk or Mercia Mudstone, or more competent bedrock.

Driven pre-cast piling is used particularly in treating ground on the approach to structures and its use on the M74 motorway completion project is described in Nowak (2013). 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, 2002), for the same purpose. The use of piling for approach earthworks to structures is also discussed by Reid and Buchanan (1984).

A piled raft was adopted for a section of the M60 Manchester ring road (*New Civil Engineer*, 2000). Instead of excavating nearly 200 000 m³ of peat present at foundation level and replacing with imported fill, a 900 m long reinforced concrete raft supported by some 4500 pre-cast concrete piles was constructed. In this way, earthworks in the wet winter weather were avoided and the works proceeded to programme. Another reported advantage was the 72 000 lorry movements, through local roads, saved by the re-design.

This solution may also be considered for unstable sites such as former landslides, where the additional fill load placed directly on the unstrengthened ground could provoke further instability. Snedker (1985) describes a trunk road running transverse to a slipped area in a hillside, formed from Etruria Formation (Etruria Marl), in which ancient failure surfaces to depths of 7 m had been discovered. Two rows of 915 mm dia. bored piles stabilised the hillside and permitted the up-slope cut and fill required by the highway construction.

12.6.3.6 Excavation and replacement

Another option favoured where embankments are to be raised on soft compressible soil is to excavate and to replace by good quality fill, with or without a form of geocell mattress. However, excavation may have to take place below the groundwater table, as may placing and compacting the replacement fill. Such solutions may also require large volumes of unsuitable soil to be taken to tip and large quantities of fill to be imported, which are not environmentally attractive.

On the N8 Fermoy Bypass contract in Ireland, the contractor elected to excavate some 6 m depth of soft clay and peat over a 300 m length of route corridor and replace with excavated sandstone from the adjacent cutting. This solution was considered more efficient in terms of cost and programme than the use of vertical drainage, soil strengthening or structural methods. The excavated material was placed in environmental bunds locally within the site boundary.

The consequences of over-excavation with regard to adjacent services and assets should also be considered.

12.6.3.7 Lightweight fill

Instead of considering strengthening of the foundation soil, a different approach would be to reduce the load imposed by the embankment.

This could be achieved by replacing some or all of the embankment fill by lightweight substances such as polystyrene, pulverised fuel ash or lightweight aggregate (Figure 12.6). These will reduce the initial applied load from embankment construction and, hence, the amount of settlement but will not influence the rate of consolidation.

Nettleton *et al.* (1996) describe how the use of pulverised fuel ash fill for part of the A13 embankment reduced the calculated load on the foundation soils by some 25%. A combination of various approaches may be considered. Note that polyester foam will be damaged if allowed to come in contact with fuel oils; consequently, drainage must be comprehensive and secure if foam is to be adopted. It is also known to be inflammable.

12.7. Construction aspects

12.7.1 Foundation excavation

Prior to placing the first layers of fill, the topsoil is usually stripped and stored for re-use on completed earthworks. This is particularly important when it is suspected that the founding stratum contains ancient failure surfaces, fissures or foliations that may cause failure during or after construction so that these features can be identified.

If not already adopted at the design stage, a toe dig-out solution (Figure 12.5) could be warranted if the inspection revealed more severe conditions than anticipated.

More recently, however, a practice has been adopted of leaving topsoil in place particularly where a competent founding stratum is present, the topsoil is thin, for example less than 400 mm and there is no the requirement for the topsoil in the scheme earthworks balance (Chapter 10). If topsoil is left in place the effect of any additional settlement of the embankment should be considered and a minimum embankment height defined below which the topsoil can remain.

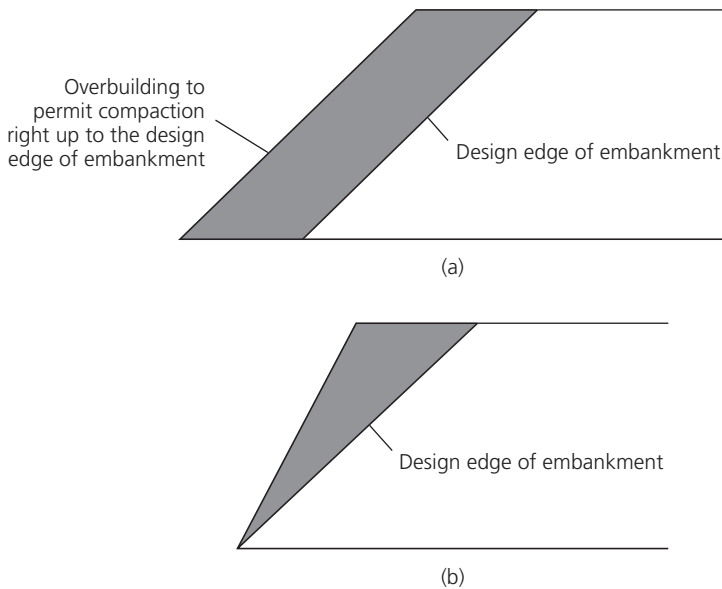
Even though additional settlement from leaving topsoil in place can be accommodated in the design, some earthmoving contractors prefer to remove the topsoil, particularly on cohesive foundations as its presence makes the required compacted density of the lower layers of embankment fill difficult to achieve.

12.7.2 Non-uniform compaction and 'overbuilding'

Compaction may not be uniform across the whole cross-sectional area of the embankment, because of the difficulty of securing full compactive effort at the edges.

The requirement for uniform strength and compressibility characteristics throughout the embankment is consequently compromised. This may be avoided by 'overbuilding', or constructing the embankment wider than is necessary according to design geometrical requirements (Figure 12.7(a)). Alternatively, and if the land is not available, a slope steeper than the final design slope can be constructed and then trimmed back (Figure 12.7(b)). If

Figure 12.7 Increasing the degree of compaction at the embankment edge by 'overbuilding' (a) overbuilding where sufficient land-take exists; (b) overbuilding where land-take is scarce (stability of temporary sleeper profile to be checked)



this latter procedure is adopted, foundation and embankment stability should be checked for the steeper side slopes. Whichever approach is employed, higher levels of compaction can be achieved up to the edge of the final embankment slope with beneficial consequences for stability, although less so for the approach shown in Figure 12.7(b).

As already noted, over-consolidated clays at water contents well below their plastic limit are particularly difficult to compact, and chunks will at best be brought into contact but not kneaded together. This kind of under-compaction (see Section 3.4.3) taken together with non-uniform compaction provides excellent opportunities for water to enter the surface of an embankment, contributing to instability.

12.8. Drainage aspects

12.8.1 Carriageway

Studies of failed slopes in embankments raised from over-consolidated clays (Greenwood, 1985) demonstrated the importance of proper detailing of drainage of the carriageway. For the slips investigated (in eastern England), the failed mass was very wet and extremely soft. It was concluded that water had entered the embankment by means of an extended granular sub-base, that is, the carriageway sub-base had been extended outwards from the underside of the carriageway towards the embankment edges. Water had penetrated and the fill had been softened in this area. No indication was given as to whether the outer edges of the embankments had been compacted by overbuilding (see Section 12.7.2), the absence of which would have accounted for some of the softened state

of the fill. Irrespective of the absence or otherwise of overbuilding during construction, water draining into the edges of embankment fill can only be deleterious and the proper detailing of carriageway drainage is essential.

12.8.2 Foundation

Basal drainage blankets are sometimes provided between the formation and the overlying embankment fill. Properly designed and installed, they can assist in lowering the pore pressures both in the fill and, if present, in the underlying foundation. However, if the incorrect particle size distribution is selected for the drainage blanket, or if otherwise appropriate material becomes contaminated, then basal drainage can have just the opposite effect. Flow of water from the embankment to atmosphere by means of the embankment side slopes may be encouraged and effective stresses in the base of the embankment may be reduced (Figure 12.8(a)). Finlayson *et al.* (1984) put down part of the reason for failure of a 10 m high embankment formed from clay till and London Clay to poorly functioning basal drains. Where their use is necessary, care is required in specifying layer thickness, falls and particle size distribution together with the method of installation so that clogging is avoided (Figure 12.8(b)). It may be necessary to provide a separator between the drain and the foundation or the overlying fill, either as a graded filter or as some form of geotextile.

Note that the oxidation of pyrite in argillaceous fill (over-consolidated clay, shale, mudstone) may produce ferrous sulfate, which may in turn oxidise to ferric hydroxide. If precipitated in drains, it could cause blockages (Highways Agency Design Note HD 41/03; HA, 2003).

Figure 12.8 Effect of properly designed basal drainage blanket on embankment performance: (a) basal drainage blanket designed without falls and/or without proper filter criteria; (b) basal drainage blanket designed with adequate falls and with proper filter criteria

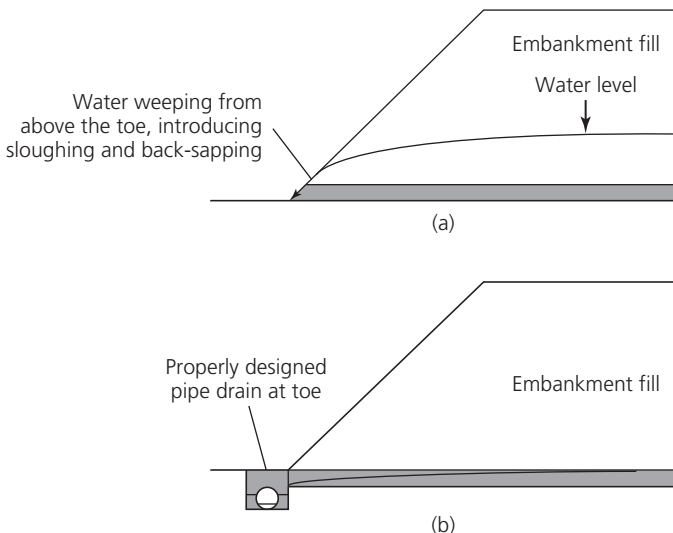
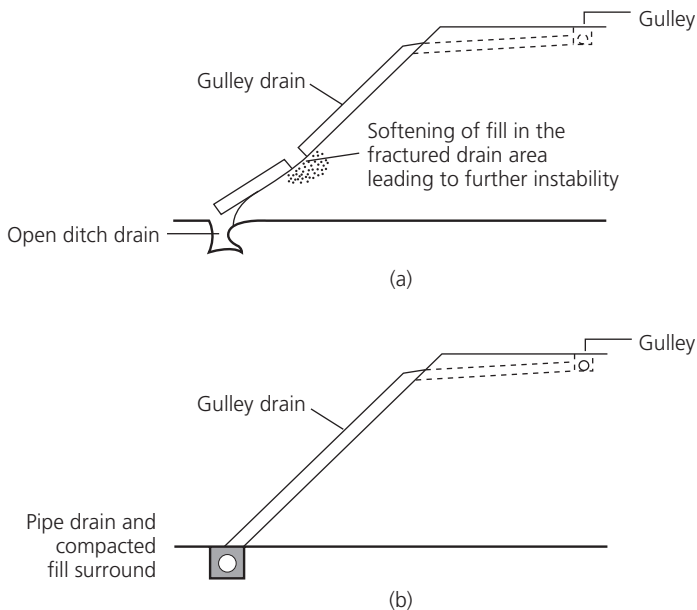


Figure 12.9 Effect of improperly constructed toe-drain on embankment stability: (a) fracture of gulley discharge drains due to failure of open ditch; (b) alternative to open ditch drainage



Where embankments are constructed on side-long ground, water could back up against the uphill side of the embankment unless drainage is provided. Open ditch drainage is often used in these (and other) circumstances. However, there is evidence (Parsons and Perry, 1985) that open ditch drains promote instability of the embankment toe, which may be reflected in shallow depth movements in the overlying embankment itself. This could force the embankment sides to adopt a non-uniform profile and which, in turn, could disrupt gulley discharge drains, causing flow of drainage water onto the embankment slopes (Figure 12.9(a)). The use of properly designed and constructed pipe drains may be preferable instead of open ditches, especially in softer and more compressible foundation soils, if this problem is to be reduced if not avoided (Figure 12.9(b)).

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Chapter 13

Factor of safety of cuttings and embankments

13.1. General

13.1.1 Definitions

A global ‘factor of safety’ has traditionally been applied as a minimum acceptable value against ultimate limit state failure of cuttings and embankments and is in line with other soil mechanics practice for the design of foundations and retaining walls. The selection of a factor of safety depends predominantly on the consideration of two sets of circumstances:

- the technical assessment of geological and geotechnical data collected for a potential slip
- the judgement of the safety and environmental consequences/economic costs of failure.

The circumstances are interlinked and both need to be considered with regard to the stability of new earthworks and existing assets. The assessment of existing assets is discussed more fully in Chapter 14.

An extreme example of the consequence of failure is new dam construction as described by Johnston *et al.* (1999). The required factors of safety reflect the impact of failure that could result in a catastrophic flood event.

The adoption of Eurocode 7 (BSI, 2004) that became law on government projects in the European Union in April 2010 presents a different approach to determining the stability of earthworks, using ‘partial factors’ instead of the traditional global factor of safety approach. The application of this approach to earthworks design (and slope stability assessment) is clearly set out within Clause 7.3 of BS 6031 (BSI, 2009) – a brief summary is provided here.

The required condition to be met by the analysis to Eurocode 7 is

$$E_d \leq R_d \quad (13.1)$$

Where E_d is the design value of the effect of actions and R_d is the design value of the resistance to an action.

The national annex to BS EN 1997-1 for the UK (BSI, 2006) adopts design approach 1 that requires the computation of two load combinations:

Combination 1 = $A1 + M1 + R1$ and

Combination 2 = $A2 + M2 + R1$

Where A is actions, M is materials and R is resistance.

Partial factors are applied to each of these prior to the analysis of slope stability, and an analysis result of greater than 1.0 demonstrates that the slope is considered stable. Bond and Harris (2008) introduce the concept of a degree of utilisation, in which the analysis result is reported as a percentage of the design value of the resistance to an action, rather than a factor of safety.

The approach of Eurocode 7 (BSI, 2004) is very similar to the methodology that has traditionally been adopted in analysis by structural engineers. It should be noted, however, that structural engineering materials are generally homogenous and are of known properties (i.e. they are often manufactured to a particular specification), whereas geological materials are heterogeneous, and properties typically vary within the same material.

In many cases it is advisable to design slopes using the traditional global factor of safety approach, with unfactored input parameters, and then check the final proposed slope for Eurocode 7 compliance.

The technical assessment will be performed on data based on observation and experiment, like strata succession, groundwater conditions, strengths and densities; while the safety, environmental and economic costs will inevitably be based mainly on subjective judgements of matters such as lives at risk, amenity loss, pollution damage and cost to the promoter and to others who may be impacted by the consequences of a failure. The consequences of failure can only be addressed by modifying the global factor of safety in traditional design, or partial factors in a Eurocode 7 design; a point that is recognised by Clause 7.4.1 of BS 6031 (BSI, 2009).

Perry (1989) studied the incidence of failure on existing highway earthworks with a view to designing earthworks side slopes to restrict failure to less than 1% of an earthwork over a 25-year period subsequent to construction. The use of this approach moves away from a calculated factor of safety to one of a risk-based approach. It, therefore, becomes an economic decision whether to design sufficiently shallow side slopes to mitigate the possibility of failure over the design life of the earthwork or whether to accept a small incidence of failure that will impact on the operation and maintenance of the earthwork.

13.1.2 Technical factors

The items for consideration under the heading 'technical factors' are those obtained from observation and experiment, usually, although by no means always, conducted at the site investigation stage. They include ground and groundwater conditions, laboratory and in situ tests, and the nature of the stability analysis.

13.1.2.1 Ground and groundwater conditions

Matters to note include:

- the complexity of the strata succession, its nature and the thickness and heterogeneity of the various layers
- the nature and degree of fabric or structure exhibited by the soils and rocks
- the presence of depositional or post-depositional features such as wash-outs, unconformities, scour hollows, buried channels, solution holes, faults and folds
- the nature and complexity of the groundwater table(s) or the pore pressure condition(s). This is particularly important if only limited standpipe or piezometer data are available on which to base the analysis, or where the hydrogeological conditions are complicated
- the presence of mine shafts or other old workings.

For embankments, the nature, heterogeneity and disposition of the fill is a major consideration affecting the confidence with which fill properties, pore pressure and groundwater conditions can be assessed. As discussed in Chapter 12, pore pressure conditions are particularly difficult to gauge in embankment fills.

13.1.2.2 Laboratory and in situ tests

Matters to note include:

- the number of tests, their representativeness and reliability with respect to the soils and rocks concerned
- the scatter of test results
- the availability of precedent information: important if information on adjoining failures is available, from which engineering properties may be 'back analysed'.

13.1.2.3 Nature of the stability analysis

Matters to note include:

- whether total or effective stress procedures are adopted
- the accuracy of the stability analysis method employed
- the possibility of complicated failure mechanisms not normally treated by routine stability methods.

Except at unity, different methods of stability analysis will not necessarily yield the same factor of safety. Significant differences can occur, depending on the type of analysis used. A summary of assumptions and some of the advantages and disadvantages of the most important methods of stability analysis is given in Table 11.1 (Chapter 11).

A valid design output requires the choice of suitable design input data, which requires the designer to identify 'best estimate' values. Eurocode 7 uses the concept of 'characteristic values' defined in Clause 2.4.5.2 of the code as 'a cautious estimate of the value affecting the limit state under consideration'. BS 6031, Clause 7.2.4 describes how this should be assessed. It is also common practice to use 'moderately conservative' input values, which

are used in retaining wall analysis and described in Gaba *et al.* (2003). This is normally defined as a design value that is slightly more conservative than the mean for a normal distribution data set. Eurocode 7 describes a more cautious approach for setting characteristic values by statistical methods, derived from Eurocode 1. This approach is considered inappropriate for soils.

Care should be taken when adopting a set of design values. A statistically derived set of design values may include low values from testing of disturbed samples or unusually high values of in situ or laboratory testing. It is common to employ engineering judgement in selection of the data set from which design parameters are derived. In addition, data derived from historical back analysis of failed earthworks slopes will generally equate to a situation in which the slope is meta-stable and are likely to be close to worst credible design values. The application of an additional factor of safety if these values are employed may result in an overly conservative assessment.

13.1.3 Safety, environmental and economic factors

The subjective judgements necessary in this category are frequently informed by experts other than engineers; input might be necessary from environmental specialists, planners and economists, especially for the larger structures and the larger risks. For smaller structures, guidance can often be obtained from the relevant officials in the local or county authority in whose area the works are being constructed. Assistance in making these judgements may also be gained by employing a framework of risk analysis and management. Valuable guidance on the application of risk principles can be obtained from the RAMP (*Risk Analysis and Management for Projects*) report (RAMP, 1998); advice directed more specifically to geotechnical problems is given by Clayton (2001).

Unlike decisions made under the technical category, which, as noted, normally await information from the site investigation, decisions made under the heading safety, environmental and economic category can be made well in advance of project implementation, thus permitting firmer control of budget and programme. However, major schemes can, and usually do, invite press speculation and adverse comment from those who consider themselves disadvantaged by the proposals. In this respect, it is essential for the Engineer and others involved to understand the difference between risk, as understood by the expert, and the risk as perceived by the lay public. As it is the public who can, and often do, succeed in modifying original proposals in the light of its perception of the risk the proposals represent, such understanding is essential.

The approach given to assessing safety, environmental and economic risk in the following is similar to that of the Geotechnical Engineering Office (1984), Hong Kong. Its approach was to equate safety risk with the occupation density of the structure likely to be affected by failure. Examples of safety risk, with adaptations considered appropriate for the difference between Hong Kong and the UK, are reproduced in Table 13.1.

Environmental risk does not appear to have been considered by the Geotechnical Engineering Office. It has been added in Table 13.2, subdivided into amenity risk (loss of an exceptional established and mature environment) and pollution risk.

Table 13.1 Examples of safety risk (adapted from Geotechnical Engineering Office, 1984)

Example	Risk to life		
	Negligible	Low	High
(i) Open farmland	×		
(ii) Country parks, lightly used recreation grounds	×		
(iii) Country roads and low traffic density B roads	×		
(iv) Storage compounds (non-hazardous goods)	×		
(v) Town squares, sitting out areas, playgrounds and car parks		×	
(vi) High traffic density B roads		×	
(vii) Public waiting areas (e.g. railway stations, bus stops)			×
(viii) Occupied buildings (residential, commercial, industrial and educational)			×
(ix) All A roads, bypasses and motorways, including associated slip roads, petrol stations and service areas			×
(x) Buildings storing hazardous goods, power stations (all types), nuclear, chemical and biological complexes			×

Risk to adjacent assets may be posed not only by ultimate limit state failure but serviceability limit state movement of ground adjacent to an earthwork under construction where the performance of buildings, services or infrastructure may be influenced by undue settlement. The M74 Motorway completion project runs, for part of its length, adjacent to the west coast main line railway from Glasgow to London. The performance of the track relies on minimising differential settlement across the rails. The design of adjacent embankment construction had, therefore, to consider the settlement effects at the toe of newly constructed embankments up to 12 m high. In addition, the effect of one cutting on the contract, which was up to 9 m deep, had to be considered with respect to the likely horizontal elastic movement of the cutting face as the earthwork was constructed and the associated settlement at the location of the tracks. The former can usually be determined by elastic settlement analysis. The latter is best determined using finite element or finite difference modelling.

13.1.4 Temporary works (short term) and permanent works (long term) conditions

A further consideration to be taken into account in selecting a factor of safety is the nature of the works: whether they are temporary (short term) or permanent (long term) conditions. ‘Temporary works’ defines the construction period of the earthworks or structure; ‘permanent works’ defines the design life period when it is discharging its design function. As such, temporary works are usually of short duration, varying from weeks to months, while the permanent works usually last several decades. Exceptionally, temporary works can last for years, not least because of factors such as project over-runs

Table 13.2 Examples of economic and environmental risk (adapted from Geotechnical Engineering Office, 1984)

Example	Economic and environmental risk		
	Negligible	Low	High
(i) Open farmland, country parks, lightly used recreation areas of low amenity value	×		
(ii) Country and low traffic density B roads, open-air car parks	×		
(iii) Facilities whose failure would cause only slight pollution	×		
(iv) Essential services (e.g. gas, electricity, water) whose failure would cause loss of service for a short period		×	
(v) Facilities whose failure would cause significant pollution or severe loss of amenity (cultivated public gardens, with established and mature trees)		×	
(vi) High traffic density B and all A roads, residential and low rise commercial, industrial and educational properties		×	
(vii) Facilities whose failure would cause heavy pollution			×
(viii) Essential services whose failure would cause loss of service for a prolonged period			×
(ix) Buildings whose failure would cause extensive structural damage			×
(x) All A roads, bypasses and motorways, including associated slip roads, structures and service areas			×
(xi) Buildings storing hazardous goods, power stations (all types), nuclear, chemical and biological complexes			×

and industrial action. ‘Short term’ and ‘long term’ are geotechnical terms illustrating periods during which the assumption of undrained (total stress) and drained (effective stress) conditions can realistically be made. ‘Temporary works’ and ‘permanent works’ are engineering terms of much broader significance, representing among other things the engineer’s perception of society’s attitude to acceptable levels of risk.

Staff and operatives on site are usually qualified by experience and/or training to recognise the onset of failure in earthworks and to take appropriate actions to preserve their own safety and that of their colleagues. In this respect, they are unlike members of the lay public who cannot be expected to take such measures. Furthermore, temporary works are usually directly under the contractor’s control and any failure is usually the contractor’s own risk. The temporary works condition also offers the contractor an opportunity to use imagination and expertise to formulate a solution that will achieve (or improve on) both construction budget and programme.

As always with factor of safety selection, the actual factors to be adopted depend on the circumstances specific to the engineering problem in hand. As undrained parameters are usually utilised for temporary earthworks slope design in cohesive soils, the determination of a suitable factor of safety against failure lies with the temporary works designer and will be based on the design values chosen.

13.2. Possible approach to factor of safety selection

13.2.1 General

BS 6031 (BSI, 2009) applies general factors of safety to failures of cuttings and embankments of between 1.3 and 1.4 for first time failures and approximately 1.2 for failures involving ancient failure surfaces.

Similarly, the French Laboratoire Central des Ponts et Chaussées (2010) advise a minimum factor of safety between 1.2 and 1.5 for natural slopes and between 1.3 and 1.5 for cuttings in horizontal ground. They also consider an additional ‘security coefficient’ of between 10% and 30% to take account of uncertainties in design parameters, the temporary or permanent nature of the earthwork and the consequences of failure, together with the degree of confidence in the reliability and durability of reinforcing measures, where utilised.

For embankment dams Johnson *et al.* (1999) propose a factor of safety against failure depending on the loading condition to reflect the consequences of failure, as set out in Table 13.3.

The partial safety factors for use with a design to Eurocode 7, and also included in BS 6031 (BSI, 2009) are given in Table 13.4.

It should be noted that the partial safety factors for actions are developed from Eurocode 1 (BSI, 2002) and the accompanying national annex for buildings (BSI, 2005). Design document PD 6694-1 (BSI, 2011), for earthworks relating to highway structures, advises of different partial safety factors on actions to Eurocode 1 national annex for traffic loads on bridges (BSI, 2008).

13.2.2 Cuttings

Two conditions are normally treated for failures in cuttings: first time failures, involving fresh cuttings, and reactivation failures, involving ancient failure surfaces present in the soil mass. The factor of safety selected for the latter is almost invariably lower than for the former. Failure involving the reactivation of ancient failure surfaces usually takes place slowly, giving time for the appropriate warnings to be given to operatives and the public, and

Table 13.3 Typical factors of safety for new dam construction after Johnson *et al.* (1999)

Loading condition	Typical minimum acceptable factor of safety
End of construction	1.3 to 1.5
Steady seepage with reservoir full	1.5
Rapid drawdown	1.2

Table 13.4 Partial safety factors for analysis to Eurocode 7

	Symbol	Combination 1	Combination 2
Action		A1	A2
Permanent: unfavourable	γ_G	1.35	1.0
Permanent: favourable	$\gamma_{G,fav}$	1.0	1.0
Variable: unfavourable	γ_Q	1.5	1.3
Variable: favourable	$\gamma_{Q,fav}$	0	0
Soil parameter		M1	M2
Angle of shearing resistance: $\tan \phi$	γ_ϕ	1.0	1.25
Effective cohesion: c'	$\gamma_{c'}$	1.0	1.25
Undrained shear strength: c_u	γ_{cu}	1.0	1.4
Unconfined strength: q_u	γ_{qu}	1.0	1.4
Weight density: γ	γ_γ	1.0	1.0
Resistance		R1	R1
Earth resistance	γ_{Re}	1.0	1.0

frequently for remedial measures to be introduced. With fresh cuttings, failure, were it to occur, could take place more quickly, possibly with dangerous consequences for the public.

The first edition of this book proposed different factors of safety against slope failure, reflecting BS 6031 (BSI, 1981), and taking into account environmental, economic and safety risks as outlined in Table 13.5. This table is an interpretation and should not be considered as prescriptive. Comment on this topic is included as an example in the current BS 6031 (BSI, 2009) at Clause 7.4.1.

The actual factor of safety to be adopted will depend on the engineer's assessment of the structure and site-specific conditions, including matters outlined in Sections 13.1.2 to 13.1.4.

Table 13.5 Possible factors of safety for cut slopes

Environmental and economic risk	Safety risk					
	Factor of safety (first time failure)			Factor of safety (reactivation failure)		
	Negligible	Low	High	Negligible	Low	High
Negligible	1.30 (1.10)	1.35 (1.15)	1.40 (1.20)	1.10 (>1.0)	1.15 (1.05)	1.20 (1.10)
Low	1.35 (1.15)	1.40 (1.20)	1.45 (1.25)	1.15 (1.05)	1.20 (1.10)	1.25 (1.15)
High	1.40 (1.20)	1.45 (1.25)	1.50 (1.30)	1.20 (1.10)	1.25 (1.15)	1.30 (1.20)

Figures in brackets represent the factor of safety for the temporary works condition

The choice of factors of safety may vary depending on the quality of the data to hand. A factor of safety less than 1.3 for negligible risk first time failure (permanent works condition) could be considered if good information is available. However, if the quality of the information is poor, it might be necessary to increase the factor of safety for the stated risk; for example, instead of adopting a factor of safety of 1.4 for a low-risk first time failure, a figure of 1.45 or even 1.5 might be necessary. For low-risk reactivation failures (permanent works condition), a value of 1.1 rather than 1.2 could be adopted where there is very good information and monitoring will be performed (see Chapter 15).

Rather than increase the factor of safety, which at best is a step based on informed but qualitative judgements, the designer may feel that more technical data are needed. Contractual arrangements should be such that further geotechnical investigation may be undertaken, if necessary during construction, to achieve the desired level of knowledge and greater certainty in design.

Another alternative is to 'design out' the element of uncertainty surrounding a particular technical factor; for example, if the nature and extent of ancient failure surfaces and/or the appropriate residual shear strength values were in doubt, the designer might feel justified in adopting a more conservative approach to the cut slope design by introducing excavate and replace measures utilising shear keys or the use of structural shear keys.

If little information were available about groundwater table rise, the designer could adopt a more comprehensive drainage than might have been necessary had more complete information been available. The designer should always inform the promoter of the possible cost savings to be gained by further geotechnical investigation.

13.2.3 Embankments

13.2.3.1 Embankment fill

In principle, there would appear to be no difference between the selection of factors of safety for a cut and for an embankment slope. In practice, however, differences may be substantial. Although soils can and do change rapidly throughout the length of a cut, they can be investigated before the main works and the implications for the factor of safety selection determined. This is unlikely to be the case for embankments where, unless the locations of the various fill types are specified, fill can vary markedly and unpredictably in distribution. Moreover, as has been seen, pore pressure response in embankments can be highly uncertain, particularly over the long term or permanent works condition.

Embankment performance also depends on drainage, whose efficiency is controlled not only by the robustness of its design and construction but also by its maintenance. The design and construction of the embankment is very likely to be carried out by different organisations than those that will be responsible for the operation and maintenance. Note that Bishop and Vaughan (1962) wrote: 'The principal uncertainty in the determination of factor of safety (of embankment dams) lies in the estimate of pore pressure represented by the term r'_u .' This statement would probably be challenged by few engineers now, whether for embankment dams or for any other form of major embankment construction involving clay fill.

For these reasons, it may be prudent to adopt somewhat higher factors of safety for embankment slopes than for cut slopes, as can be seen in Section 13.2.1. Use of an enhanced factor of safety should also help to guard against delayed failure of embankment slopes and assist in keeping maintenance costs to a minimum.

13.2.3.2 Embankment foundations

Embankment foundations on soft, normally consolidated cohesive soils and loose granular soils are likely to undergo significant settlement under load from embankment construction. Pore pressures in the underlying soil will increase with embankment loading and, in the case of foundations in cohesive soil, will not dissipate immediately. This could be the critical case for the stability of the embankment as the material is likely to possess a high pore water pressure and would exhibit undrained behaviour in failure mode.

For soft compressible soil a factor of safety as low as 1.3 on the construction condition may not preclude failure if the pore pressure does not drain and the height of fill over time is not controlled. Even when failure does not occur, there may be damage to services already installed in, or adjoining, the embankment, such as drainage lines or ditches due to settlement. Furthermore, cracks could be produced in the fill that could have consequences for the performance of the embankment over its design life, and provide a direct pathway for excess water infiltration.

For over-consolidated clays and granular materials deformation of the foundation during construction is not normally a problem and a factor of safety of 1.3 on the short term or temporary works condition (negligible risk) is probably acceptable.

Table 13.6 from the first edition of this book gave possible factors of safety against the failure of embankment construction after taking these factors into account. As with cut

Table 13.6 Possible factors of safety for embankment fill and foundation

Environmental and economic risk	Safety risk					
	Embankment fill			Foundation		
	Negligible	Low	High	Negligible	Low	High
Negligible	1.40 (1.20)	1.45 (1.25)	1.50 (1.30)	1.30 (1.10)	1.35 (1.15)	1.40 (1.20)
Low	1.45 (1.25)	1.50 (1.30)	1.55 (1.35)	1.35 (1.15)	1.40 (1.20)	1.45 (1.25)
High	1.50 (1.30)	1.55 (1.35)	1.60 (1.40)	1.40 (1.20)	1.45 (1.25)	1.50 (1.30)

1.40 Denotes factors of safety for the stated risks for the permanent works condition; (1.20) denotes factors of safety for the stated risks for the construction condition for over-consolidated clays and granular materials. The construction factor of safety can be increased by at least 15% when constructing embankments on 'soft compressible soil'

slopes, the actual factor of safety employed will depend on the Engineer's assessment of the structure and site-specific conditions, including the various matters outlined in Sections 13.1.2 to 13.1.4. This table is an interpretation and should not be considered prescriptive.

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Chapter 14

Earthworks asset management

14.1. Introduction

Although new earthworks are still constructed, particularly for road and rail infrastructure, the majority of infrastructure in the UK and Europe relies on earthworks constructed in the nineteenth and twentieth centuries. A majority of the rail and water supply infrastructure is over 100 years old and major motorway development was carried out in the period 1960–1990. The result of this is that major asset owners in the UK such as the Highways Agency, Network Rail and Transport for London, together with local authorities have, since the 1980s, developed systems for inspection and maintenance of earthworks assets. In addition, most major asset owners are constrained by property boundaries with respect to upgrade, and most projects, particularly motorway and trunk road widening, involve the modification of existing earthworks to provide additional lane width.

Failure of earthworks assets can result in a danger to third parties and the travelling public and, particularly in the case of railways, may lead to loss of service and potentially loss of revenue.

Asset management is also important to non-public asset owners, for example heritage railways where traffic loading is considerably less but also is funding for maintenance and asset repair. Crapper *et al.* (2014) present a pragmatic approach to asset management on infrastructure of this type.

14.2. Asset management systems

Major infrastructure owners manage their existing earthworks assets to an appropriate management system to ensure an acceptable performance and that the earthworks do not present a risk to users.

The theory of earthworks asset management is described in McGinnity and Saffari (2012) but a 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 modes
- a risk register.

14.3. Asset catalogue and assessment

An asset catalogue is the first step to management and involves the inspection and condition of existing earthworks assets. Procedures for inspection of condition and formulation of an asset register have been developed in the UK by the Highways Agency (HA, 2003), Network Rail (2007, 2011a, 2011b, 2012) and London Underground (2010).

The Highways Agency document HD 41/03 (HA, 2003) categorises earthworks into the following categories after inspection:

- Class 1: areas of defect
- Class 2: areas at risk
- Class 3: areas of repair, strengthening and preventive works.

Some asset owners develop earthworks asset catalogues into the assessment of the stability of earthworks assets. These assessments are usually undertaken after a programme of contemporary ground investigation, particularly with respect to railways where the earthworks were constructed before ground investigation was developed. In the case of highways ground investigation, particularly with respect to motorways, information is available from prior to the original construction and is sometimes supplemented with earthworks construction records. In the UK, borehole information for all assets is available through the British Geological Survey database and for highways through the Highways Agency Geotechnical Data Management System (HAGDMS), which also acts as an asset register for earthworks on all of the HA's major motorways and trunk roads.

Care should be exercised when stability assessments of existing earthworks are carried out to modern standards such as Eurocode 7 (BSI, 2004) or BS 6031:2009 (BSI, 2009) as the analysis may indicate failure of earthworks that, by inspection, are stable. This is usually due to the difficulty in obtaining good quality geotechnical data. Contemporary ground investigations will be carried out on 'live' infrastructure, during engineering hours on railways when trains are not running or on the verge or hard shoulder of roads probably requiring a lane closure. The time available for investigation can constrain the use of conventional cable percussion or rotary boreholes and, particularly in the case of railways, window sampling and trial pitting are more commonly used. Where cable percussion and rotary boreholes are constructed they may be carried out at the crest of cuttings or the toe of embankments, hence not sampling the materials most critical with respect to non-performance or failure.

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 types. 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 cuttings and embankments where the incidence of failure was recorded as less than 5% of total earthwork length. Winter *et al.* (2013) present a risk assessment tool for landslide hazard assessment on the Scottish road

Table 14.1 Typical factors of safety for infrastructure embankments after Perry *et al.* (2003b)

	Worst credible	Moderately conservative
Shallow failure	1.05	1.15
Deep failure	1.10	1.30

network following debris flow events caused by heavy rainfall in August 2004. They present a hazard ranking based on the study of 607 km of the route network in order to prioritise the areas of instability that require remedial treatment as part of an asset management programme.

Factors of safety have also been proposed for assessing the stability of existing earthworks assets. Perry *et al.* (2003a, 2003b) considered the variable minimum factors of safety for infrastructure embankments as shown in Table 14.1.

Egan (2005) considered the acceptable factors of safety for Network Rail earthworks as shown in Table 14.2.

14.4. Stability and performance

14.4.1 General

The failure of earthworks after construction can be considered in both ultimate limit state (ULS) and serviceability limit state (SLS). ULS failures are likely to comprise sufficient movement of the material that replacement of the failed area is required. SLS failures are usually in the form of creep where movement is at a much slower rate but may affect performance of the asset as it results in unacceptable differential movement across or along railway tracks or cracking of a highway carriageway. These failures can initially be addressed by remedial works, such as the addition and re-packing of ballast below railway tracks but if they continue over a period of time they may be of sufficient magnitude that more significant works may be required of a similar scale to that required for ULS failures.

Table 14.2 Typical factors of safety for Network Rail embankments after Egan (2005)

Failure class	Factor of safety	
	Moderately conservative peak parameters	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 pre-works condition	1.1

14.4.2 Shallow instability

Shallow instability usually occurs on the sloping face of a cutting or embankment. 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. Similarly, Perry (1989) concluded 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. Reid and Clark (2000) also observed similar depths of failure on slopes generally greater than 4 m in height. This concurs with the work of Perry where shallow failures were more critical in slopes of greater than 5 m in height.

Investigation of these failures indicates that the most likely mechanism for failure is one of water content or pore pressure increase in the near surface of the earthworks 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.

Vegetation on the slope surface can have a positive and stabilising effect through reinforcement provided by roots and the control of moisture entering the slope surface. This has been developed by Norris *et al.* (2008) from work by Greenwood *et al.* (2004) and Norris and Greenwood (2006). It is also covered by Coppin and Richards (2007). The development of trees presents a maintenance commitment from the asset owner and, particularly in the case of railway cuttings, can pose a danger due to unmaintained growth and the possibility of collapse onto the running rails due to decay or periods of very high winds. London Underground has developed guidelines for vegetation to slopes where remedial measures or tree felling have been undertaken, which proposes the use of grasses, wild flower mixes and low-height shrubs to reduce long-term maintenance (2007).

14.4.3 Deep-seated failures

First time slides have occurred in cuttings comprising stiff over-consolidated clays many years after construction due to pore water equilibration over time. These have been studied extensively by Skempton (1948, 1964, 1970, 1977), Chandler and Skempton (1974) and Chandler (1984). These studies have shown that the strength at failure is lower than the peak strength for the material but greater than the residual strength.

14.4.4 Progressive failure

Progressive failure occurs as a result of the non-uniform development of strains along a failure surface in cutting slopes leading to non-uniform mobilisation of shear strength. Potts and colleagues (1997, 2000) have studied these types of failure where fissured, over-consolidated 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.

Ellis and O'Brien (2007) have attempted to extend this work as a predictive tool for a safe slope angle and slope height where progressive collapse will not occur. Their findings

show for cutting slopes in London Clay shallower than 1V:3H and less than 8 m in height progressive failure is unlikely. For steeper slopes or those greater than 8 m in height, progressive collapse may occur up to 125 years after construction.

Hughes *et al.* (2007) describe delayed failures in cuttings in lodgement till and conclude that long-term strength parameters should be used in the design of cuttings in this material.

14.4.5 Seasonal shrink and swell

Seasonal shrink and swell occurs in earthworks of high plasticity clay due to the high moisture demand of mature trees. It is a particular issue on railway embankments where recharge occurs through the ballast at the top of the embankment but is removed by the trees on the slope. This causes a pore pressure differential through the earthwork, and negative pore pressures commonly develop at the surface of the slope. This can result in the development of cracking of the soil during dry periods (desiccation). This can lead to the formation of water pathways and greater permeability of the soil increasing the depth of the zone of influence of shrink–swell. This may lead to an increase in strain beyond the peak strength, and cyclic loss of strength results in progressive collapse, particularly on cutting slopes. In embankments the shrink–swell leads to a phenomenon of ‘ratcheting’ where horizontal swelling in wet periods does not recover during dry periods. This leads to progressive settlement of the top of the embankment, potentially leading to loss of service caused by settlement that will then require remediation.

Further information on seasonal shrink–swell movements and the effects of vegetation is provided by McGinnity *et al.* (1998), Kovacevic *et al.* (2001), Loveridge and Anderson (2007) and Nyambayo *et al.* (2004).

Briggs *et al.* (2013) report on the influence of pore pressures on London Clay embankment stability during periods of increased precipitation.

14.4.6 Shoulder instability

Shoulder instability is usually found on railway embankments where fine-grained lightweight granular material, predominantly ash, forms the top part of the embankment construction, possibly due to settlement or instability during the original building of the embankment or during maintenance in the past.

The presence of these conditions can lead to ravelling down the slope and narrowing of the shoulder, usually due to dynamic loading from trains during dry periods or flow during periods of heavy precipitation.

14.4.7 Flow failure

Flow failure may occur in high permeability granular material as a result of rapid recharge during a period of high intensity rainfall or a storm event. The rapid recharge could be caused by concentrated surface run off or areas of poor earthworks drainage and, hence, is likely to occur in cuttings and result in mass flow of saturated material.

14.5. Remediation of existing assets

Remediation of shallow slips on existing cuttings and embankments is usually effected by the removal of the failed material and replacement by free draining granular material, or by use of the failed material strengthened by geogrid layers. Alternatively, in the case of cohesive materials, the failed material may be modified by the addition of lime and/or cement. The latter forms of replacement are sometimes difficult to undertake next to a live highway or railway as there is insufficient space to store the excavated material locally, and hence replacement with granular material is the preferred solution.

Remediation may also be carried out where, from inspection, failure is predicted in the medium term or where ULS failure has not occurred, but continuing slow movement is causing disruption to the service, such as cracking in a highway carriageway or excessive lateral or horizontal movement of railway tracks. This could take the form of re-grading cutting or embankment slopes to a shallower slope angle or, in the case of embankments, provision of an additional buttress to the slope if space allows. The provision of additional drainage in the form of batter drains or a crest drain is also commonly used to prevent further deterioration of the asset.

Structural methods are also used to remediate existing earthworks slopes. They are detailed in McGinnity and Saffari (2012) and Nowak (2012) but can be summarised as:

Cuttings

- spaced bored piles
- steel sheet piles
- soil nailing
- toe retaining walls.

Embankments

- anchored bored pile wall
- king post and plank wall
- sheet piling
- toe wall
- verge protection system, for example: Rugei wall.

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Chapter 15

Use of instrumentation

15.1. General

Instrumentation is installed during ground investigation to provide information for earthworks design such as groundwater levels from piezometer readings. Some of this instrumentation may be retained for use during construction to confirm design assumptions.

With respect to the construction of earthworks instrumentation is usually installed for the following purposes:

- to monitor performance of earthworks against design assumptions
- to prevent failure of an earthwork during construction
- to monitor movement of adjacent assets due to earthworks construction
- to determine mode of failure and monitor remedial measures with respect to earthworks failure during the construction process.

During operation and maintenance of an earthwork, instrumentation can be used to monitor medium- and long-term performance in lieu of the implementation of an earthworks or structural solution where serviceability failure may occur. In addition, monitoring can be installed during the construction process for observation of earthwork performance during the short and medium term.

Implementation of any monitoring regime should take due regard of construction activities that may result in loss of monitoring instrumentation due to the movement of construction plant or burial of the instrumentation due to earthworks operations. In all cases, contingency plans should be formulated for the replacement of instrumentation critical to performance monitoring of the earthwork.

Monitoring is commonly allied to the observational method of design and maintenance as detailed by Nicholson *et al.* (1999) and Patel (2012). Eurocode 7 (BSI, 2004) states: ‘Because the prediction of geotechnical behaviour is often difficult, it is sometimes appropriate to adopt the approach known as the “observational method”, in which the design is reviewed during construction.’ Eurocode 7 develops four criteria that must be satisfied before construction begins.

- Acceptable limits of behaviour should be established.
- It must be shown that there is an acceptable probability that the actual behaviour will lie within these limits.

- Monitoring methods should reveal whether or not actual behaviour falls within acceptable limits, noting that the monitoring itself should be sufficiently frequent to enable contingency actions to be taken, and noting that the response time of the instruments and the time necessary to analyse them should also be sufficiently rapid.
- Contingency actions should be prepared that may be adopted if monitoring reveals behaviour outside the acceptable limits.

For any monitoring of an earthwork where the observational method is employed it is critical that trigger levels of performance are established with suitable contingency measures in place, particularly with respect to potential serviceability or ultimate failure and the effect of movement of adjacent assets.

15.2. Types of instrumentation

15.2.1 General

Types of geotechnical instrumentation and their usage are described by Dunnycliff *et al.* (2012) and are summarised below. The Dunnycliff paper should be referenced for further details of equipment descriptions.

15.2.2 Piezometers

Piezometers are used to monitor positive or negative pore water pressure in soil and water pressures in joints in a rock mass. When readings are taken from a number of different instruments in a response zone a piezometric profile can be inferred.

A number of types of piezometer installation are available, namely:

- observation wells
- open standpipe piezometers
- vibrating wire piezometers
- pneumatic piezometers
- flushable piezometers.

Observation wells typically consist of a perforated section of pipe connected to a riser pipe normally 25–50 mm in diameter in a borehole of 75–250 mm diameter. Open standpipe piezometers are similar to observation wells but include a sub-surface seal so that the instrument responds to groundwater pressure only around the filter element. These instruments are normally measured with a dip meter that will register the surface water level. They, however, have a long response time to reach equilibrium water level and, hence, are not efficient for short-term monitoring of water level of cohesive soils with mass permeability of less than 10^{-6} m/s.

Vibrating wire piezometers have a diaphragm separating the pore water from the measurement system. A tensioned wire is attached to the mid-point of the diaphragm and is plucked magnetically by an electrical coil, the frequency of the vibration measured using a frequency counter. Pneumatic piezometers work on the principle of a gas pressure applied to an inlet tube that builds up on a diaphragm and flows to an outlet tube once

the gas pressure exceeds the pore pressure. The gas supply is then turned off and the pressure in the outlet tube equates to the pore water pressure. Both vibrating and pneumatic piezometers have a short time lag and are suitable for soils of all permeabilities. Vibrating wire piezometers can be readily connected to a data logger to record real time values of piezometric level. Pneumatic piezometers cannot be readily data logged but can be read remotely from the borehole position by positioning pressure gauges in a reading kiosk.

Where piezometers are installed to measure the dissipation of increased pore water pressure below embankment construction an assessment of the effect of periods of heavy rainfall causing the rise in the groundwater table should be considered. This will have an effect of reducing the effective stress in the consolidating layer of soft clay and an erroneous interpretation of the rate of primary consolidation settlement may occur. It is, therefore, advisable to install a control piezometer distant from the earthworks construction to record changes in background groundwater level so that results from piezometers in the embankment foundation can be calibrated.

Twin tube piezometers (Ridley 2003, Ridley *et al.*, 2003) have been developed primarily for the monitoring of negative pore pressures in existing railway embankments, specifically London Underground. They are capable of measuring negative pore water pressures to -95 kN/m^2 .

15.2.3 Deformation monitoring instrumentation

Deformation monitoring of earthworks can be effected through surface or installed instrumentation.

15.2.3.1 Surveying techniques

Surveying techniques are commonly employed to monitor movements of both cuttings and embankments and comprise a series of targets monitored on a regular basis or, in real time, if a motorised total station is used. Where embankments are constructed on a stiff foundation the underlying soil will not deform significantly under embankment load during construction, and surface levelling stations will record settlement of the underlying soils as well as settlement of the fill. Care should, however, be taken when targets are monitored to quantify the settlement of earthworks embankments on soft ground as the targets will have been installed on completion of the embankment and will not take account of any settlement that occurred during embankment construction and prior to installation. The global positioning system (GPS) can also be used as a ground control network and is proving very useful for long-term monitoring of dams and other large earthworks, see Rutledge and Meyerholtz (2005). GPS is also extensively used to record electronically the position of defects during asset assessment and to position exploratory holes during ground investigations.

15.2.3.2 Rod and plate settlement gauges

The installation of rod and plate settlement gauges largely overcomes the problem of settlement monitoring from the commencement of embankment construction. They comprise a solid plate, usually of the order of 1 m diameter or square, placed onto the

natural ground surface onto which a suitable length of casing is welded. It is normally necessary to extend the casing vertically upwards as greater fill thickness is placed by either welding or screwing on additional lengths. Settlement of the underlying ground is recorded by levelling the top of the rod.

From time to time, a rod of known length can be introduced into the casing from the surface to the base plate and a reading of the settlement of the underlying natural ground is taken by levelling the top of the rod. The surface of the fill, say 2 m from the casing, is levelled at the same time at say four positions (north, south, east and west) and the results averaged. The difference between the settlement of the underlying natural ground and the fill surface is the settlement of the fill. Settlements may be measured at different times during and after filling and a graph plotted showing settlement against time.

Friction between the fill and the casing may cause the fill to hang up on the casing, so that the settlement at the surface close to the casing may be less than the true value. To avoid this undesirable feature, readings of the fill surface should not be taken too close to the casing. Care should also be taken to site the settlement plate on reasonably flat natural ground beneath the fill.

15.2.3.3 Liquid levelling gauges

Liquid level gauges can also be used to determine the settlement of the foundation of embankment construction on soft ground. They take the form of a tube installed horizontally in the foundation material prior to embankment construction. Relative elevation is determined either from equivalence of liquid level in a manometer or from the pressure transmitted by the liquid. They allow installation to be made without interruption of the earthworks or potential damage of the instrumentation during the filling process. The gauges only provide a means of measuring relative elevations between two or more points. If absolute settlement or heave is required data must be referenced to a benchmark. The same arrangement can be packaged as a probe that is pulled along a buried pipe, thereby providing vertical deformation data at any position along the pipe.

15.2.3.4 Inclinometers

Inclinometers are used for monitoring lateral deformation below the ground surface. They are particularly used for determining the zone of landslide movement, monitoring the rate and extent of horizontal movement of embankments on soft ground or for the monitoring of movement on existing cutting and embankment earthworks assets.

They comprise four components: a vertical guide casing with racking grooves, a portable probe, a portable read-out box and a graduated electrical cable linking the probe to the read-out box. The probe is lowered to the bottom of the casing and a reading is made. Additional readings are made as the probe is raised incrementally to the top of the casing. A set of baseline readings should be taken immediately after installation to which all subsequent readings can then be referenced.

Care should be taken in the construction of the borehole in which the inclinometer tube is installed and the method of installation. On the M4/M25 junction construction project it

was found that spurious readings were obtained from inclinometers that were installed initially at greater than 6° from the vertical. In addition, all readings over time should be taken with the same probe. If a different probe is employed a new set of base readings will be required to be established.

In-place inclinometers are typically used for monitoring sub-surface deformations around excavations or within slopes when automatic or rapid readings are required. The device consists of tilt transducers joined by articulated rods and provides similar information to a conventional inclinometer. In-place inclinometers can be used in conjunction with conventional inclinometers with the in-place instrument installed initially and then replaced by a conventional instrument or vice versa.

A new form of inclinometer installed to measure movement of an existing embankment on London Underground is described by Ridley *et al.* (2012).

15.2.3.5 Extensometers

Fixed borehole extensometers are normally installed in boreholes in rock or soil for monitoring the change in distance between two or more points along the axis of the borehole without the use of a moveable probe. Typical applications are for monitoring deformations around underground excavations in rock or behind the faces of excavated slopes. When the location of a measurement point is determined with respect to a fixed datum absolute deformation data are provided. A mechanical or electrical transducer measures the distance from the face of the collar anchor to the end of the rod.

Magnetic extensometers can be used where the compressibility of one fill type is being compared with the compressibility of another, or where it is necessary to separate the settlement of the fill from that of the underlying soil. Magnets are located in the fill about a central plastic tube grouted into a borehole constructed into the foundation material. A probe lowered down the tube locates the magnets and measures their position with respect to the base of the borehole that is presumed as fixed.

15.2.3.6 Tiltmeters

Tiltmeters consist of a gravity sensing transducer either fixed in place or as a portable device mating with permanently fixed reference points. They are used to measure the change in inclination or rotation in the ground. Various types of transducers are used in tiltmeters and wireless data transmission is also available. More details are provided in Dunnicliff *et al.* (2012).

15.2.4 Suggested uses of instrumentation

The types of instrumentation that could be employed to investigate the performance of newly constructed earthworks or existing earthworks assets are presented by Dunnicliff *et al.* (2012) and are summarised below.

15.2.4.1 Embankments on soft ground

The most frequent use of instrumentation for embankments with a foundation on soft compressible soil is to monitor the rate of primary consolidation settlement and to

determine the overall stability of the embankment construction in the short term. If the calculated factor of safety against failure during or immediately after construction is close to unity instrumentation will be installed to provide due warning of any inherent instability to allow remedial measures to be implemented before failure occurs.

Suggested types of instrumentation are as follows:

- initial groundwater conditions: vibrating wire piezometers (open standpipe/pneumatic)
- initial deformation: surveying methods
- embankment slope stability: surveying methods and inclinometers
- embankment overall stability: vibrating wire piezometers
- consolidation of foundation: surveying methods, rod and plate settlement gauges, extensometers, liquid level gauges, piezometers.

15.2.4.2 Clay embankments

Embankments constructed of cohesive material generally exhibit negative pore pressures immediately after construction due to the change in stress state from cut to fill. Over time the pore pressure will equilibrate to the new stress state and the negative pore pressure will tend to zero and positive pore pressures may establish. That may lead to failure, particularly of the embankment shoulders, soon after completion of construction.

Longer term failures of embankment slopes may occur due to a shrink–swell that results from the presence of vegetation. Similarly, on sparsely vegetated slopes seasonal changes in pore pressure close to the surface may result in shrink–swell during periods of wetting and drying. It is common that the former results in negative pore pressures close to the surface of the slope particularly where large trees are present.

Ultimate stability may be influenced by the permeability of the underlying foundation as higher permeability soils will provide under-drainage whereas low permeability soils may retard the dissipation of any excess pore pressure.

Suggested types of instrumentation are as follows:

- embankment stability: flushable piezometers, surveying methods, inclinometers
- underlying drainage: open standpipe piezometers
- shrink–swell: flushable piezometers, surveying methods, inclinometers, extensometers.

15.2.4.3 Embankment dams

Instrumentation is usually within an embankment dam to study the in-service performance against design predictions with respect to general performance and also that of special foundation conditions or uncommon design features. Where the latter are being monitored they will control the monitoring design and choice of monitoring instrumentation.

The practice of installing instrumentation in embankment dams where no special problems are envisaged has not been fully implemented historically and in the 1960s dams were constructed with no instrumentation. Any failure was then widely publicised and dam engineers felt it necessary to install instrumentation routinely for their own protection.

Peck (1985, 2006) believed in the importance in instrumentation and commented that ‘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’ (1985).

Monitoring regimes for embankment dams are detailed in Dunnicliff (1988, 1993).

15.2.4.4 Cut slopes and landslides in soil

The design of instrumentation of cut slopes requires a good understanding of the likely failure mechanism together with the location and properties of any potential stratigraphic discontinuities.

Instrumentation can be designed to monitor the performance of constructed slopes or existing slopes as part of an asset management regime. It can also be designed to monitor the performance of areas of slope instability.

Definition of the groundwater regime is important for all conditions and can be used as a basis for modification of the design slope angle. Negative pore pressures are likely to be generated during excavation that may give rise to initial stability that will decrease over time. Monitoring will determine the reduction of negative pore pressures and the likely change in stability of the cut slope. For existing assets or areas of instability understanding of the groundwater regime will inform any remedial measures that need to be implemented. In addition, it will determine the effectiveness of any installed drainage measures.

Inclinometers are also commonly used to predict the stability of cut slopes and monitor any instability with particular reference to planes of shear in an unstable slope mass.

Suggested types of instrumentation are as follows:

- groundwater/porewater conditions: open standpipe piezometers, vibrating wire piezometers, flushable piezometers
- surface deformation: surveying methods, GPS, tiltmeters
- subsurface deformation: inclinometers
- long-term stability: rain gauges, load cells.

15.2.4.5 Cut slopes and landslides in rock

Monitoring of cut slopes and instability in rock slopes is likely to use the same types of instrumentation for soil slopes. It should be recognised, however, that rock slopes are more likely to exhibit brittle failure, and normal 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 stability and use rainfall measurements to warn of potential problems.

In addition to the instrumentation suggested for cut slopes in soil above, the installation of crack gauges across discontinuities that may develop into failure planes is commonly associated with an extensive implementation of surveying methods.

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Chapter 16

Safety considerations

16.1 General

Civil engineering projects, on which earthworks are to be undertaken, either as infrastructure earthworks or site preparation for building construction and foundations, represent a potentially dangerous environment for the construction workforce.

In addition, this applies to ground investigation for earthworks projects, particularly with respect to trial pits and trenches.

The main hazards for earthworks operations can be considered as:

- movement of earthmoving and other plant, particularly at speed and reversing
- collapse of unsupported excavations
- damage to (and from) public utilities such as electricity, gas and water mains
- contaminants that may be found on brownfield sites
- chemical additives for earthworks conditioning, for example quicklime and cement
- plant falling into excavations
- blasting operations.

Other hazards include:

- slips, trips and falls on uneven ground
- falling objects from overhead plant and demolition/construction activities
- asphyxiating gases in confined spaces
- groundwater ingress into excavations
- soft compressible soils.

CIRIA Report C662 (CIRIA, 2007) reports percentage fatalities over the period 2003–2006 that could possibly be attributed to earthworks operations as:

- 26% struck by moving/reversing vehicle
- 6% contact with electricity
- 15% struck by falling objects.

16.2 Legislation and guidance

All construction activity in the UK is controlled by the Construction (Design and Management) (CDM) Regulations 2007 and their approved code of practice (HSE,

2007). These documents include required provision under the Health and Safety at Work Act 1984.

Work of duration longer than 30 days or involving more than 500 staff days of construction work or involving demolition will also have to be notified to the Health and Safety Executive (HSE).

It should also be noted that the CDM Regulations also apply to all ground investigation activities where the ground is broken. Further information is provided in the *ICE UK Specification for Ground Investigation* (ICE, 2012).

The CDM Regulations define the responsibilities through the life cycle of a project comprising:

- design
- construction
- operation and maintenance, and
- decommissioning.

They also define the responsibilities of the main parties of the project, namely:

- client
- CDM coordinator
- designer
- contractor.

The CDM Regulations state that risk assessments should be undertaken by designers and contractors and method statements should be produced for all elements of work. The method statements should be disseminated to the workforce by way of regular ‘tool box’ talks.

In preparing method statements and risk assessments the following legislation, produced by the HSE, should also be taken into account:

- the Control of Substances Hazardous to Health (COSHH) Regulations (2012)
- the Control of Asbestos Regulations (2012)
- the Manual Handling Operations Regulations (1992)
- the Confined Spaces Regulations (1997)
- the Construction (Head Protection) Regulations (1989).

The HSE also produce guidance documents, for example:

- *Health and Safety in Construction* (HSE, 2006)
- *Avoiding Danger from Underground Services* (HSE, 2000).

Additional guidance on application of the CDM Regulations is also provided by the following publications:

- Construction (Design and Management) Regulations 2007 – industry guidance for designers, Construction Industry Training Board (HSE, 2007)
- CDM 2007 – *Construction Work Sector Guidance for Designers*. CIRIA Report C662 (CIRIA, 2007)
- CDM 2007 – *Workplace 'In-use' Guidance for Designers*. CIRIA Report C663 (Gilbertson, 2007)
- *CDM 2007, Questions and Answers* (Perry, 2008)
- *Handbook of Site Safety for the Water Industry*. CIRIA Special Publication SP137 (CIRIA, 1997).

The above list is not exhaustive, and additional guidance on safety issues in earthworks engineering is provided by BS 6031:2009 (BSI, 2009) and Clayton (2001a, 2001b).

16.3 Management of health and safety risks

Health and safety risks should be managed through the design, construction and operations and maintenance phases of a project in order that the correct party can manage the residual risk.

Design risks should be recorded in safety, health and environmental boxes on construction drawings and in the design risk register.

The design risks for earthworks contracts would normally be expected to be managed out during construction, for example:

- temporary support of excavations
- excavation in bays to prevent slope instability
- measures to deal with poor/contaminated ground
- measures to deal with surface water and groundwater.

Before work begins, the project director must be satisfied that the staff, both at the office and on the site, thoroughly understand:

- the nature of the project and the risk to health and safety that it might bring in its train, and
- the legislation likely to control or impact on site activities.

Also prior to the start of the project, a risk assessment must be undertaken on all aspects of the work in order, so far as is reasonably practicable, to avoid or reduce the possibility of risk of injury and danger to health to all the workforce. The assessment should be done by, or with, somebody with a thorough understanding of the project and of the importance of each particular work activity. In such a way, the 'ownership' of the various risks, and how they can be mitigated or avoided, should become clear. With this information, and having regard to the various items of legislation, a safety plan may be formulated, which should:

- take account of the need to coordinate with other site users (including visitors)
- establish procedures for site training and induction for new joiners

- ensure that copies of the relevant legislation are prominently displayed in work offices
- establish rules and emergency procedures, including those for first aid
- establish measures for issuing appropriate protective clothing and safety equipment
- establish procedures for regular meetings between the senior personnel on site dealing with safety issues
- establish procedures for documenting and reporting accidents
- devise regular audits to compare site procedures with the safety plan.

For all projects, the organisation's health and safety staff should be consulted to determine the sufficiency of the above for the particular problems in hand.

Staff undertaking in situ tests (such as dry density determinations) are particularly at risk from earth-moving and compaction plant. They should be protected by warning lights and barriers where appropriate.

Safety will be improved if plant is segregated into its own haul roads, away from private cars and pedestrians. Haul road surfaces should be well maintained because this improves braking distances, and watering during dry weather helps to reduce dust nuisance. Plant should be restricted to travelling parallel to the dip of the slope, wherever possible. Tipping at the edge of a fill, without the use of chocks and a banksman, should not be permitted.

The designer should, however, also be aware of the operation and maintenance risks inherent in their design solution. These may include:

- maintenance of drainage where this is critical for slope stability
- the need to load test and re-stress ground anchors
- monitoring of geotechnical instrumentation where this has been installed to monitor long-term performance, for example, embankment settlement over soft ground
- monitoring of environmental instrumentation where this has been installed to monitor long-term performance, for example, expulsion of leachate from embankment construction over an area of landfill.

As part of the operation and maintenance of a project, a health and safety file is required. This will contain construction records, including drawings. The geotechnical report to BS EN 1997-1 (BSI, 2004) should be included as part of this documentation. On highway projects in the UK this information would take the form of the ground investigation report, geotechnical design report and geotechnical feedback report as defined in the Highways Agency design document HD22/08 (HA, 2008).

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Appendix 1

End product specification for engineered fill supporting low rise structures

(from NA Trenter and JA Charles, 1996)

The model end product specification presented below has been retained, unchanged, from the first edition of this book, except that the references to BS 6031 and the Construction (Design and Management) Regulations have been updated. Additional specific guidance regarding the specification and control of fills that are to be placed to support low rise buildings is provided in BRE Digest 427 Part 3 (1998).

This model specification is intended to be used in connection with the construction of low rise buildings on engineered fill, typically no greater than 50 000 m³ in volume and no more than 5 m in average depth. The specification is believed to be suitable for most purposes but there may be special conditions existing at some sites that are not treated in this document and that should be taken into account in arriving at a properly engineered fill. It is emphasised that clay fills can be at least as susceptible to settlement or heave due to climatic, vegetation or other effects, as naturally occurring cohesive soils.

This specification is for contracts that are designed, let and supervised by a consulting engineer. However, the appropriate wording may readily be adapted for other forms of contract, such as design-and-build. It is assumed that a site investigation will have been performed prior to the works and that the geotechnical properties of relevance to enforcing the specification will have been measured.

Guidance on the design of foundations for low rise structures may be obtained from Tomlinson *et al.* (1978), Tomlinson (1995), Atkinson (1993), and from other authorities. Guidance on construction of earthworks may be obtained from BS 6031:2009 (BSI, 2009).

Note should be taken of the requirements of the Construction (Design and Management) Regulations (HSE, 2007) to the extent that they may be relevant to the works.

A1. Engineered fill

[A1.1] Engineered fill is defined as fill that is selected, placed and compacted to an appropriate specification so that it will exhibit the required engineering behaviour.

[A1.2] Fill shall be classified as follows:

- (a) unsuitable fill
- (b) general fill
- (c) restricted fill
- (d) special fill.

[A1.3] Unsuitable fill shall comprise any material so designated by the Engineer and shall include:

- (a) cohesive soils having a liquid limit in excess of 90% or plasticity index in excess of 65%
- (b) chalk having a fine fraction ($<400\ \mu\text{m}$) in excess of 10% at the borrow pit
- (c) any material containing topsoil, wood, peat or lignite
- (d) any material containing bio-degradables
- (e) any material containing scrap metal
- (f) frozen or waterlogged substances
- (g) material from contaminated sites
- (h) material that by virtue of its particle size or shape cannot be properly and effectively compacted (e.g. some slate wastes).

Unsuitable fill shall not be used at any location or part of the site, including landscaped areas.

[A1.4] General fill shall comprise all fill except unsuitable fill, restricted fill and special fill.

[A1.5] Restricted fill shall comprise material that would otherwise be classified as general fill but that contains minerals hostile to the built environment and shall include:

- (a) pyritic shales
- (b) gypsiferous clays
- (c) burnt colliery discard
- (d) pulverised fuel ash
- (e) steel slag
- (f) spent oil shale
- (g) incinerator waste
- (h) some demolition and construction industry waste.

Such fill shall be precluded from use in designated zones, including locations where groundwater may rise to the level of the underside of the deepest foundation and where its use will be condemned by the appropriate authorities on pollution grounds. Such fill shall not be placed to a depth less than one metre from the underside of the deepest foundation.

[A1.6] Special fill shall comprise material that would otherwise be classified as general fill but that contains durable well-graded natural sand and natural gravel or crushed rock,

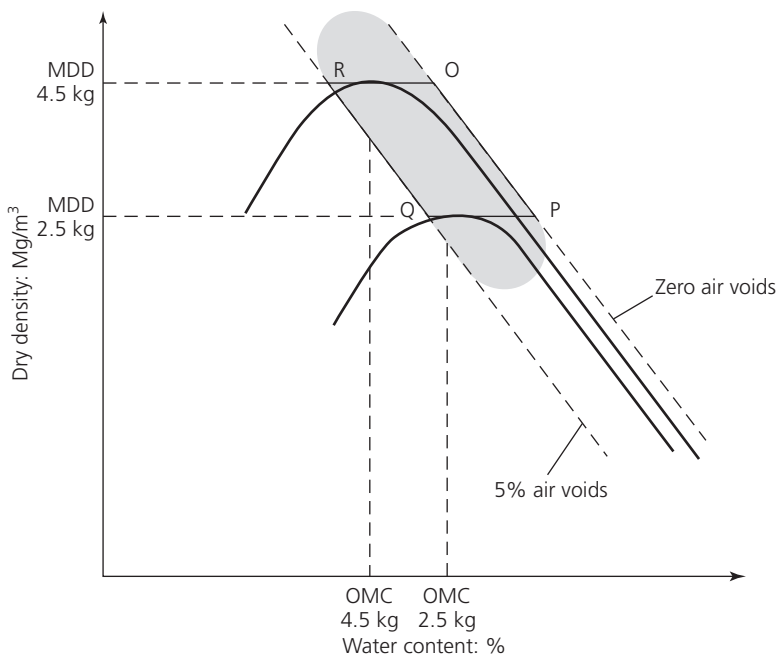
other than argillaceous rock, or durable clean crushed demolition rubble of similar particle size and free from any contaminants. Such fill may be employed as capping layers beneath structure foundations, beneath roads or as backfill to retaining walls.

A2. Selection of end product requirements

[A2.1] On the basis of the results of a site investigation that shall have been carried out, the Engineer shall, at the time of tender, provide the Contractor with the following:

- (a) The results of the tests shown below:
 - (i) natural water content (BS 1377:1990, Part 2: Test 3 (BSI, 1990))
 - (ii) liquid and plastic limits for cohesive soils (BS 1377: 1990, Part 2: Tests 4 and 5)
 - (iii) compaction tests to determine maximum dry density and optimum water content at the appropriate compactive effort (2.5 and 4.5 kg rammer) (BS 1377:1990, Part 4: Test 3)
 - (iv) particle density (specific gravity) to assist in evaluating the compaction test (BS 1377:1990, Part 2: Test 8).
- (b) A graph such as that illustrated in Figure A1.1.

Figure A1.1 Basis for design. Compaction curves for 2.5 and 4.5 kg rammer methods shown. Water contents and dry densities, when plotted, to fall within stippled area



[A2.2] The graph will show the dry density plotted against water content for the 2.5 kg and 4.5 kg rammer method compaction tests; the corresponding optimum water contents and maximum dry densities; and the 5% air voids line.

[A2.3] By reference to lines OPQR illustrated on Figure A1.1, the required level of compaction will be indicated by selecting appropriate water content and dry density values. This level of compaction shall form the basis of the compaction specification.

A3. Preparation of site

[A3.1] On side-long ground, drainage grips or trenches shall be excavated uphill of the area to be filled and compacted. Drainage shall be effected without causing siltation or erosion and water shall be disposed of in a manner to be agreed by the Engineer.

[A3.2] The area to be filled, whether an existing excavation or otherwise undisturbed ground, shall be graded to falls, and sump pumping or other suitable dewatering facilities shall be provided by the Contractor to keep the base of the excavation dry at all times.

[A3.3] Where the area to be filled comprises an existing excavation, the excavation shall be inspected and subsequently monitored by the Contractor, to ensure that there is no danger of its collapse during the works with consequences for safety, for existing buildings or for other construction adjoining.

[A3.4] All topsoil shall be stripped and, where required for further use, stockpiled in an area provided by the Contractor and agreed by the Engineer.

[A3.5] All soft and compressible soils or existing fill shall be removed and run to spoil in dumps provided by the Contractor and agreed by the Engineer (including licensed tips in the case of certain contaminated materials). The work shall be accomplished in such a way that there is no undercutting of the sides of existing excavations.

[A3.6] Existing foundations or ledges of hard rock, roots of trees or former pipelines or services at the base of the area to be filled shall be excavated and replaced with compacted general fill, which shall be compacted to the same specification as adopted for subsequent compaction works.

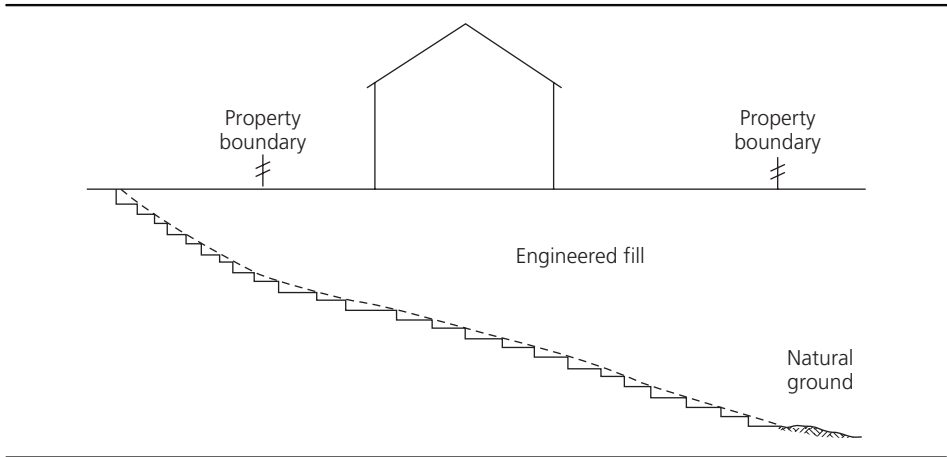
[A3.7] Where unsuitable material has been excavated, the underlying natural ground shall be compacted to the same specification as adopted for subsequent compaction works.

A4. Disposition of fill

[A4.1] Where construction is required on fill placed over sloping natural ground, and where fill thickness is less than 5 m, the natural ground shall be benched, with the maximum vertical height of each bench not exceeding 500 mm (Figure A1.2).

[A4.2] Where a development contains landscaped areas on which no structures will be built, the underlying fill shall be selected, placed and compacted in the same way as the

Figure A1.2 Benching of sloping natural ground. Benching within engineered fill area only

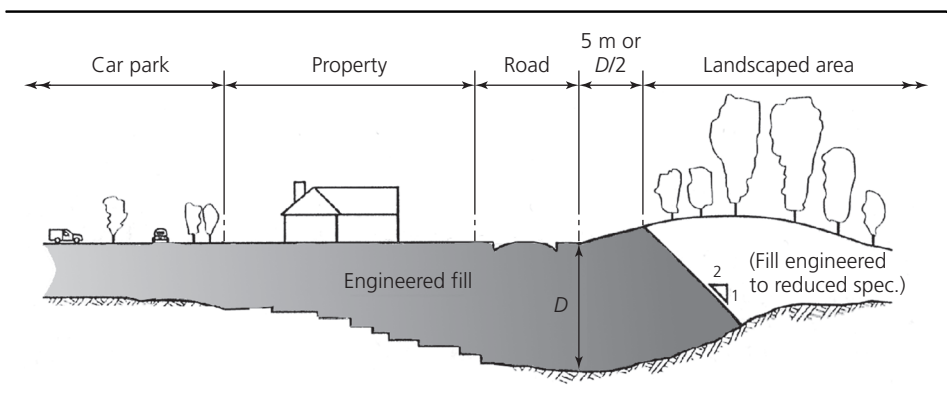


engineered fill, unless otherwise directed by the Engineer. Where some relaxation of the specification for fill compaction underlying landscaped areas is permitted, there shall be a transition zone between the fill underlying the landscaped area and the fill underlying the structure. The dimensions of the transition zone will depend on the degree to which fill compaction was relaxed for the fill in the landscaped area (Figure A1.3). The location and extent of fill placed to a reduced standard of compaction shall be recorded.

A5. Placing and compacting fill

[A5.1] Fill shall be placed and compacted in near-horizontal layers of the thicknesses required to achieve the specified end product and shall, as far as practicable, be brought up at a uniform rate so that all parts of the site reach finished (formation) level at the same time.

Figure A1.3 Construction beneath buildings, hardstanding and landscaped areas



[A5.2] The compaction plant selected, the number of passes made and the fill layer thickness and water content used shall have regard to the specified end product and the means and manner of control testing.

[A5.3] Where several different types of fill material (all meeting the requirements of Clause [A1] of this specification) are to be employed, they shall be deposited in such a way that all parts of the site receive roughly equal amounts of a given material, in roughly the same sequence, thus ensuring a uniform distribution of fill types over the whole fill thickness.

[A5.4] The Contractor shall take all necessary steps to ensure that the fill is placed at the water content necessary to achieve the specified level of compaction and shall, where necessary, add water to or dry the fill, in order to obtain this value. Where it is necessary to add water, this shall be done as a fine spray and in such a way that there is time for the water to be absorbed into the fill before being rolled by the plant.

[A5.5] Cobbles, boulders, rock or waste fragments whose largest dimension is greater than two-thirds of the loose layer thickness shall not be incorporated into the fill.

[A5.6] No fill shall be placed and left uncompacted at the end of a working day. Compacted fill shall be graded to falls to ensure free runoff of rainwater without ponding.

[A5.7] Compaction plant and compaction method shall be selected having regard to the proximity of existing trenches, excavations, retaining walls or other structures and all work shall be performed in such a way as to ensure that their existing stability is not impaired.

[A5.8] If weather conditions are such that the specified water content and density values cannot be achieved, the Contractor shall cease work until such time that the fill can be placed and compacted to meet specification requirements.

[A5.9] If the results of control tests (Clause [A6.3]) indicate that the fill is being placed and compacted in such a way that the desired level of compaction is not being achieved, the Contractor shall further compact or, if necessary, shall excavate the affected work and replace with new fill, compacted to meet the specification requirements.

[A5.10] If the results of control tests (Clause [A6.3]) indicate that antecedent weather conditions (such as frost or heavy rain) have caused deterioration of finished work such that the work no longer meets the specification, the Contractor shall at its own cost take such steps as are necessary to bring the fill to specification requirements.

A6. Control testing

[A6.1] The end product requirements selected in Clause [A2.1] shall be controlled by in situ and laboratory testing as follows

- (a) in situ dry density (BS 1377:1990, Part 9: Test 2) and water content determinations (BS 1377:1990, Part 2: Test 3)

- (b) where required these tests shall be augmented by water content–dry density relationships (BS 1377:1990, Part 4: Test 3) and particle density (BS 1377:1990, Part 2: Test 8).

[A6.2] At least 14 working days before the start of site work, the Contractor shall provide the Engineer, for approval, with a list of the equipment the Contractor proposes to use to undertake these tests.

[A6.3] Control tests shall be performed throughout the fill at such frequency and at such locations as shall be directed by the Engineer.

[A6.4] When requested by the Engineer, the Contractor shall make available a plot of in situ dry densities against in situ water content results on a graph such as illustrated in Figure A1.1, showing that the results lie within or above the area OPQR or such area as has been selected by the Engineer. Should any results lie outside the selected area, the Contractor shall provide the Engineer with proposals for rectifying the existing situation and for improving future performance.

[A6.5] The Engineer will, from time to time and with reasonable notice, request the Contractor to make available equipment to enable the Engineer to perform control tests. The results of these tests shall be used by the Engineer in assessing the Contractor's performance.

A7. Monitoring of fill performance

[A7.1] If instructed by the Engineer, the Contractor shall make arrangements for the performance of the fill, once placed, to be monitored. Monitoring may take one or more of the following forms:

- (a) optical levelling of surface monuments
- (b) standpipes or piezometers
- (c) load tests
- (d) other methods as directed by the Engineer.

[A7.2] The Contractor shall, within 21 working days of receiving notification of the Engineer's intention to monitor fill performance, arrange for the procurement and supply of the equipment to the Engineer's written specification and shall inform the Engineer of the date on which the equipment installation shall commence. The specification shall include:

- (a) a full description of the nature and type of instrument and the purpose it fulfils
- (b) the number required and the locations and or depths at which it is to be installed
- (c) the frequency, accuracy and duration for which any readings are to be taken.

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Appendix 2

Examples of plant commonly used for earthworks

The following narrative and images capture a selection of the most commonly used plant operating on earthworks construction projects at the present time. It expands on the text by Horner (1988), which was included in the first edition of this book, and Dumelow (2012). The details given for specific items of plant are typical only and for more accurate descriptions manufacturers' websites should be consulted.

Plant to excavate and load



360 degree excavator



360 degree excavator



Long reach excavator



Wheeled loader

Backacters comprise a bucket on the end of an articulated boom, set on a pneumatic-tired or crawler tractor unit. The boom, bucket arm and bucket are usually controlled by hydraulic rams. They operate by digging towards the machine in an arc from a small distance above the surface on which the machine stands to a position vertically below the outer edge of the machine. Loading is generally carried out by lifting the bucket, and swinging the boom away from the working face to the awaiting haulage vehicle. Alternatively, material can be dumped adjacent to the machine.

Table A2.1 Typical details of excavators

Machine type	Weight: tonnes	Typical dig depth: m
Mini excavator	1–8.5	2.0–4.0
Small excavator	14–18	4.0–6.0
Medium excavator	23–30	5.0–7.0
Large excavator	37–92	6.0–8.0

Long boom medium and large excavators can operate to an excavation depth of 12.0 m

Typical details of backacters are summarised in Table A2.1.

Buckets for backacters generally range in size between 0.45 m and 2.4 m wide with a corresponding capacity of 0.2 m³ to 6 m³. The smaller width buckets are for smaller excavators and trenching operations, with the larger buckets being used on larger machines.

Dragline equipment (not illustrated) is operated from cranes or similar plant with a long boom. Excavation is by pulling a bucket, suspended on a cable, towards the machine by a second cable. Thus draglines are especially suited for the excavation of soft and loose materials from a distance at a level beneath or slightly above their tracks, and may be used to excavate under water. Excavated material can be removed directly to a stockpile or loaded into haulage vehicles or conveyor hoppers by rotating the machine, with the bucket in the upright position. Draglines are much larger in weight and capacity than backacters, varying in weight between 1800 and 7500 tonnes with bucket capacities in the range 24–110 m³. They are not usually utilised on earthworks operations and are more commonly employed in mining and quarrying.

Face, front or loading shovels (not illustrated) are constructed in a similar manner to a backacter except that the boom, bucket arm and bucket operate in the opposite direction, that is, up and away from the machine. They are generally used for excavating faces up to about 8 m high and stockpiles. Loading is carried out in a similar manner to the backacter, although some machines have bottom dump buckets to increase the speed of loading. They are useful in excavating soils, weak rock and blasted rock from faces in cuttings and so on. Some larger excavators can be converted from backacters to face shovels. Typical machine weights and capacities are summarised in Table A2.2.

Table A2.2 Typical weights and capacities of face shovels

Machine type	Weight: tonnes	Bucket capacity: m ³
Compact	5–8.5	0.9–1.3
Small	11.5–15.5	1.7–5.0
Medium	18.5–30	2.5–8.0
Large	42.5–99.3	5.5–12.5

Forward loaders consist generally of a crawler tractor at the front of which is mounted a wide bucket that can be moved in a vertical plane. Excavation is carried out by driving the machine towards, and the bucket into, the material; the bucket is then turned and lifted upwards thus catching and excavating the material. The hauling vehicle is loaded by driving the loader to, and emptying the bucket into, the body of a vehicle. Loaders are generally used to excavate material at and for a distance above ground level (e.g. stock-piles, blasted rock) and can be used to push or haul material in the bucket over short distances. Modern loaders have hydraulically powered buckets and the smaller units may be equipped with a backacter (i.e. a backhoe loader). They typically vary in weight between 16 and 28 tonnes with a bucket capacity in the range 1.8–3.2 m³.

Earthmoving plant



Articulated dump truck



Rigid dump truck



Road tipper lorry



Motor scraper

Rigid dump trucks generally vary in size from about 40–90 tonnes capacity, are usually used in mines, quarries or opencast sites, although can be employed on large earthworks contracts such as the recent construction of the new TGV line in France between Tours and Bordeaux.

Articulated dump trucks are common for normal earthworks contracts as they are versatile, and are especially suitable for hauling on softer subgrades. Bodies of dump trucks may be heated by the exhaust from the engines and the sides are often splayed out to provide a large loading target. The speed of tipping is increased over a road lorry by the absence of a tailgate. Small dumper units are available for work on small sites and track-mounted dump trucks are also available with load capacities up to about 20 tonnes. Struck capacity of articulated dump trucks is typically 10.5–18.5 m³ (22.7–39.5 tonnes).

Road lorries are available in varying sizes up to about 38 tonnes gross vehicle weight and generally have steel or steel/aluminium sheeted bodies. Such vehicles require to be loaded by other plant but are generally unloaded by side or rear tipping.

Scrapers can excavate, load, haul and deposit material in one cycle and may be towed or self-propelled (motorised). They consist of a centrally mounted bowl, the bottom, leading edge of which can be controlled. Both towed and self-propelled scrapers are effectively articulated between the front motorised or towing unit and the bowl, and larger self-propelled scrapers may have a second engine mounted at the rear. During excavation and loading, as the machine moves forward, the cutting edge scrapes the top layer of soil into the bowl. On completion of loading the apron is lowered and the bowl raised. Deposition of the material is carried out by moving the ejector forward, with the apron raised and the bowl partially lowered. The thickness of the layer of deposited soil can be regulated by control of the ejector, apron and bowl. Ideally, haulage should take place over well maintained haul roads with minimum of steep gradients or sharp turns. Scrapers are less commonly used than some 30 years ago unless the earthworks are of sufficient volume. Earthworks on motorway widening contracts in the UK are generally carried out by the use of backacter and dump truck due to the volume of earthworks involved and the site access conditions.

Towed scrapers typically range in size from about 5 m³ to 17 m³ struck capacity, and are generally towed by crawler tractors. They are limited to economic hauls of about 400 m each way.

Single and double engine motorised scrapers are typically range in size from about 11 m³ to 30 m³ struck capacity (15 m³ to 40 m³ heaped). Motor scrapers can generally be economically operated on fast hauls of up to 2.6 km each way, with optimum performance on hauls of about 800 m.

Elevating scrapers are similar to conventional scrapers except that a rotating elevator moves the excavated material from the front of the bowl, breaks it down and deposits

it within the bowl, thus reducing the resistance on, and increasing the speed of, loading. Elevating scrapers generally range in capacity from 7 m^3 to 26 m^3 .

Conveyors (not illustrated) have been used with varying degrees of success on construction projects in the UK. They are built up with a number of units of endless flat belt conveyors placed in series, and major changes in direction can be made at transfer points, where material from one belt falls and is channelled onto the next. Loading is generally carried out by means of a hopper, which may be designed to screen out over-size material. The conveyor may end either above a stockpile or in a stacker, which allows the material to be spread over a wide area. The capital costs to set up a conveyor system are often large in comparison with conventional plant, but operating costs may be lower. Conveyors are generally used in quarries and pits, in areas of very steep or poor access.

Bulldozers and motor graders



Bulldozer (small)



Bulldozer (large)



Motor grader

Bulldozers are tractors equipped with a front pusher blade that can be raised and lowered by hydraulic rams. An angle dozer has a blade that is capable of being set at an angle to push material sideways while the tractor moves forward. The tractor unit is usually mounted on crawler tracks thus allowing it to travel over and push off a wide variety of ground conditions although wheel-mounted units are available. Blades are manufactured in a variety of styles, but are all of heavy duty construction with a hardened steel basal leading edge, driven into the ground to cut and push the material to be excavated. Dozers have a wide variety of roles including excavating soils and weak rocks, ripping, moving excavated material over short distances, spreading materials, trimming earthworks, and acting as a pusher to boost the effective power of scrapers and other plant.

Table A2.3 Typical weights and power capacities of bulldozers

Machine	Weight: tonnes	Power: kW
Small dozer	8–9.5	45–72
Medium dozer	13–25	95–180
Large dozer	38–104	225–630

A wide range of crawler units is available, ranging between about 45 kW and 630 kW. Dozer blades may be attached to other plant (such as self-propelled compactors) so that they can operate without the need for a dozer.

Typical operating weights and power capacities of bulldozers are summarised in Table A2.3.

Graders are used to spread fill and finely trim the subgrade. They consist of a blade that can rotate in a circular arc about a sub-horizontal axis and that is supported beneath a longitudinal frame joining the front steering wheels and the rear drive wheels. The front wheels are generally articulated, while the rear wheels are set in tandem beneath the motor and control units. The blade is used to trim and redistribute soil and therefore graders usually operate in the forward direction.

Earthworks processing



Mobile vibrating screen for use with sand and gravel
(shown here separating fines from a granular fill)



Water bowser and spray bar



Self-propelled soil stabilisation plant (direction of travel from right to left mixing the binder spread across the ground surface into the soil)



Tractor-towed stabiliser

Mobile screens are used to separate granular materials into multiple grades, depending on the particle size. Material is deposited into the hopper by a front-end loader or large 360-degree excavator. The vibratory action of the plant helps the soil to pass through a number of internal screens, which are selected according to the fraction(s) requiring separation. Separated fractions exit by means of conveyors into a series of separate stockpiles. Mobile screens are track mounted to provide agile on-site mobility and low ground-bearing pressures. The conveyors can usually be folded hydraulically and remain on board for transport, thus minimising costs. Hopper capacities typically range from 4.5 m³ to 7.5m³, and weights are generally in the order of 23–32 tonnes.

Water bowsers are mobile water tanks that can be fitted with spray bars to regulate discharge rates. Water bowsers have several uses on earthworks schemes, from increasing the water content of a fill ahead of compaction to suppression of dust when working in dry conditions.

Soil stabilisation plant may be deployed on sites where the in situ soil requires improvement through the use of binding agents, to improve bearing capacity, for example. The plant can take the form of large self-propelled machines or smaller tractor-towed units, depending on the scale of the earthworks required. Towed units typically weigh around 4.5 tonnes, with self-propelled units ranging from about 21 to 33.5 tonnes. Most soil stabilisers have a working depth of up to 500 mm, increasing to 560 mm for some of the

largest plant. As the machine traverses the ground surface, milling and mixing rotors mix the pre-spread binding agents (e.g. lime or cement) into the underlying soil to form a homogeneous soil–binder mix. The stabilised soil can then be compacted and trimmed as required.

Compaction plant



Self-propelled smooth-wheeled vibratory roller



Self-propelled tandem smooth-wheeled vibratory roller



Self-propelled padfoot/tamping vibratory roller



Tamping roller



Pneumatic-tyred roller



Towed vibratory padfoot/tamping roller



Towed vibratory roller



Towed grid roller



High energy impact compaction (HEIC) roller



Vibrating plate compactor



Remotely operated vibrating plate compactors
(three heavy duty compactors fixed in a line)



Excavator boom-mounted
vibrating plate compactor

Smooth-wheeled vibratory rollers are perhaps the most commonly used item of compaction plant for earthworks schemes in recent years as they are generally suited to most soil types. Most examples seen now are self-propelled machines, comprising larger single drums or smaller tandem arrangements. Towed examples are particularly suited to uniform fine sands, however, as self-propelled machines can have a tendency to 'dig in'. Units generally range in size from about 0.5 to 27 tonnes, and usually operate at speeds between 1.5 and 2.5 km/h. Repeated passes at high speed are generally less effective than fewer passes at lower speed. When tandem rollers are used, both rolls can be considered to be applying compaction; the input of compactive effort is generally

adjusted to that of the roll with the lesser mass per unit width; alternatively it can be considered as a single drum roller with higher mass per unit width. The frequency of vibration of a given machine is generally fixed by the manufacturer and typically varies from 20 to 35 Hz for larger rollers and from 45 to 75 Hz for smaller rollers. On some rollers the frequency can be varied. When the vibrating mechanism is not used the roller can be considered as a deadweight roller.

Tamping rollers, including sheepfoot and padfoot rollers, are generally similar to smooth-wheeled rollers in overall appearance except that the steel drums are fitted with projecting feet. The projections, which vary considerably in their geometry, may be circular, square or rectangular and are available in different lengths and tapers. The projections penetrate the soil layers and hence provide an element of horizontal compaction in addition to vertical compaction. They also deliver high localised pressures and generate an overall mixing and kneading action as the plant traverses the fill surface, ensuring that a good interlock is achieved between successive layers. These rollers are generally best suited to fine (cohesive) soils with relatively low water contents as the projections can cause unwanted disturbance to some granular materials on retraction (particularly with sheepfoot rollers, which have long, slender projections). Units may be self-propelled or towed, with single or multiple axles and the mass may be increased with ballast. Optimum speeds vary between 4 and 10 km/h.

Pneumatic-tyred rollers are most commonly self-propelled machines, although towed versions are still available. They may have one or two axles, on which are set a number of rubber-tyred wheels. On two-axle machines, one axle generally has one fewer wheel than the other and the tyres are offset so that the weight of the roller can generally be increased by applying ballast to a box or platform mounted over or between the axles. The compactive effort can be further modified by adjusting the inflation pressures of the tyres. Provision is usually made for a certain amount of vertical movement of individual or pairs of wheels, to enable the roller to negotiate uneven ground while exerting steady pressure on the soil. On the 'wobbly-wheel' roller, the wheels are set to wobble from side to side, providing more kneading action to the soil. These rollers are classified in terms of total mass of the roller divided by the number of wheels. Optimum speeds are generally between 1.6 and 24 km/h, with operating weights ranging from about 9 to 27 tonnes for self-propelled machines. Pneumatic-tyred rollers are generally rarely seen on UK earthworks schemes now.

Grid rollers have a cylindrical, heavy, open steel mesh roller and may be ballasted with concrete blocks. They are generally towed units and can operate at speeds between 5 and 24 km/h. Typical weights vary between 5.5 tonnes net and 15 tonnes ballasted. They are suited to breaking up lumps of stiff clay and weak rocks as the ballasted mesh delivers relatively high contact pressures.

High energy impact compaction (HEIC) rollers have non-circular drums that deliver a high compactive effort through a combination of vibration and a dynamic 'cam' effect as the drum rotates. Most rollers seen now have drums of truncated triangular or pentagonal cross-section, and may be self-propelled or towed by tractor units. Depths

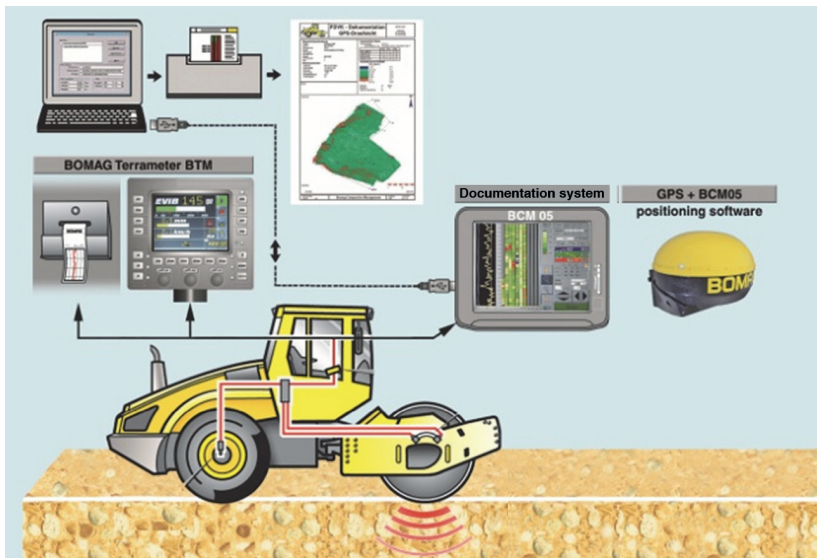
of effective compaction can reach 4 m in some instances, and these rollers are therefore generally reserved for major earthworks schemes, such as land reclamation projects.

Vibrating plate compactors vary in weight from 100 kg to 2 tonnes with plate areas between 0.16 m^2 and 1.6 m^2 . Smaller versions are manually guided and are therefore suitable for compacting small or awkwardly shaped areas. They usually travel at about 0.7 km/h. They are classified in terms of the mass divided by the area of the base in contact with the ground. Some modern units can be operated remotely, which has health and safety benefits, and also efficiency benefits when several compactors are joined together, as is common practice for compaction of railway ballast.

Vibro-tampers (not illustrated). Compaction is induced by vibrations set up in a base plate through a spring activated by an engine-driven reciprocating mechanism. They are usually manually guided and weigh between 50 kg and 100 kg. They are not suited to heavy compaction, but are frequently used in confined spaces. They are classified in terms of the static mass of the machine.

Power rammers (not illustrated) are manually driven machines suitable for the compaction of small or awkwardly shaped areas. Explosions in an internal combustion engine cause the machine to be driven upwards and the subsequent impact of the base plate on the ground results in compaction. They generally weigh about 100 kg, although special larger versions are available. They are classified in terms of the static mass of the machine.

Dropping-weight compactors (not illustrated) consist of a weight of around 500 kg or more that can be dropped from a controlled height, usually about 1–3 m.



Continuous compaction control (CCC):
schematic image of plant and electronic recording devices

Continuous compaction control (CCC) is a relatively recent technology that enables the performance of the fill to be monitored during compaction in real time. Specially adapted vibratory rollers are equipped with accelerometers and instrumentation that records and analyses the operation of the plant (e.g. travel speed, location, vibration frequency) and the vibration response. The increase in compaction of the fill is recorded as the work progresses, and weak areas can be identified in relative terms.

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Appendix 3

Planning and construction for earthworks projects

In order to establish, where practicable, a balance in cut and fill volume various vertical and horizontal alignments may be considered primarily based on topography along the route alignment and the likely nature of materials to be won from excavation to be utilised as engineering fill in embankment construction.

In current practice, once a vertical and horizontal alignment has been established it would be modelled in a computer program of which MXROAD and Auto Desk Civil 3d are the most commonly used in the UK.

The determination of the earthworks quantities for each individual earthwork will usually be established at 20 metre chainage intervals along the route corridor taking into account both the vertical alignment and the likely earthworks side slopes.

Ground level information for use in the modelling can be determined by topographic survey along the route corridor or by remote sensing techniques such as LIDAR (light detection and ranging) or GIS (geographic information system) data. It is common that for the early stages of a scheme the latter information would be utilised to be confirmed by a topographic survey prior to the detailed design of a scheme.

Using both sets of information a model is established from which the volumes are calculated geometrically within the computer program and cross-sections are produced at 20 metre intervals as setting out drawings for earthworks construction.

With the aid of GPS survey equipment and GPS equipment mounted on earthworks plant data produced from the computer model can be uploaded directly into field instrumentation to aid the setting out and the formation of final earthworks slopes. This method has largely replaced the use of 'batter boards' at regular chainage intervals to set out the final cut slope or embankment profile.

The first edition of this publication included a mathematical determination of earthworks quantities. Hand calculation is now rarely used but the method is included below.

In the following, simplified examples of the planning of earthmoving and compaction projects are presented, together with brief reviews of some contracts.

Earthmoving

Example of the planning of an earthmoving scheme

A site 150 m wide and 250 m long is to be developed for warehousing units. This involves the stripping of topsoil, reducing the site level and removing the material off site. The following materials are to be excavated and disposed of:

clay	18 125 m ³
topsoil	<u>10 000 m³</u>
total	28 125 m ³

The site is reasonably flat and has good access. Tipping space is available 3.0 km distant. The work is to be carried out in the early summer and approximately 10 weeks are available on the provisional programme; production is limited to a 40 hour week.

During excavation, bulking of the soils will occur and this must be allowed for. Thus

$$\text{loose volume} = \text{bulking factor} \times \text{bank volume}$$

Therefore

$$\text{loose volume clay} = 18\,125 \times 1.3 = 23\,562 \text{ m}^3$$

$$\text{loose volume topsoil} = 10\,000 \times 1.45 = 14\,500 \text{ m}^3$$

$$\text{total loose volume} = 38\,062 \text{ m}^3$$

The estimator has chosen to investigate excavation using tracked forward loaders with 14 m³ (20 tonne) road lorries. For a 1.53 m³ capacity loader

$$\text{bucket fill factor} = 0.8 \text{ (80\% capacity)}$$

$$\text{load per cycle} = 0.8 \times 1.53 \text{ m}^3 = 1.22 \text{ m}^3$$

$$\text{number of loads per 14 m}^3 \text{ lorry} = 14 \div 1.22 = 11.5 \text{ (approx. 12)}$$

$$\text{loading time of loader} = 0.8 \text{ min/load (from manufacturer's tables)}$$

$$\text{total load time/lorry} = 12 \times 0.8 \text{ min} = 9.6 \text{ min}$$

Add 2.0 min for lorry to position. Therefore

$$\text{total load time} = 11.6 \text{ min}$$

$$\text{output per loader per 40 hour week working 50 min/h}$$

$$= 40 \times 50 / 11.6 \text{ loads/week} = 172 \text{ loads/week}$$

$$= 2408 \text{ m}^3/\text{week}$$

$$\text{total time required for one loader}$$

$$= 38\,062 / 2408 \text{ weeks}$$

$$= 15.8 \text{ weeks}$$

In order that the work can be completed within the allocation time period, two loaders will be required for 8 weeks each (thus allowing time for the establishment of the plant on site, etc.).

The lorries have the following working cycle:

positioning at the loader	2.0 min
loading	9.6 min
moving on/off site	1.0 min
travel 2.5 km each way at 30 km/h	10.0 min
tipping time	<u>3.0 min</u>
total	25.6 min

Number of lorries required per loader = $25.6 \div 11.6 = 2.2$

Total number of lorries required = 4.4

Allowing for break downs and so on five lorries will be required; thus the total plant required will be two loaders and five lorries.

Example of an earthmoving project

An example of a large earthmoving project can be well illustrated by a motorway that was constructed in South Wales, in which both soils and rock had to be excavated. Because of the scale of the project the example has been simplified without altering the fundamental pattern of the work.

The contract involved the excavation of three major and one minor cutting, the construction of six embankments and the disposal of unsuitable and surplus suitable materials in off-site tips, as is shown in Figure A3.1. In the middle of the works lay a large bridge, which effectively limited the transport of fill along the line of motorway.

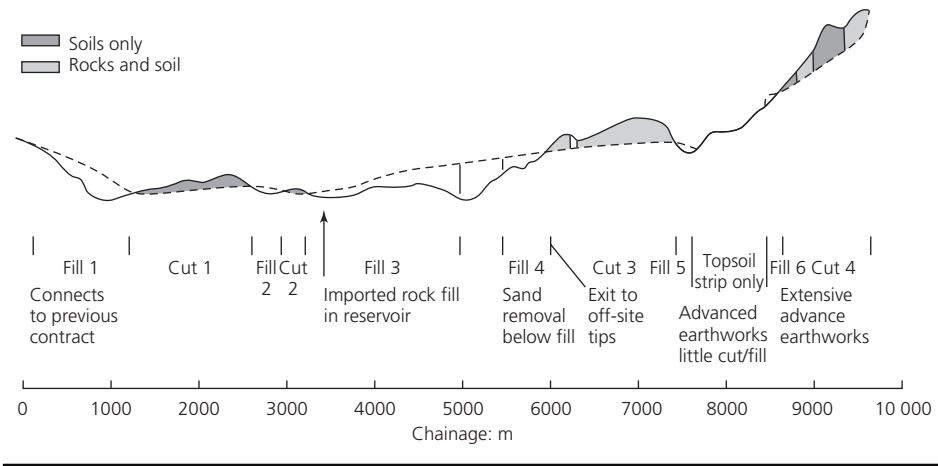
The mass haul indicated that short to medium hauls of up to 3 km and long hauls of up to 10 km would be required and that a mixed fleet of motor scrapers, dump trucks and lorries would be necessary. Rock was known to be present within two of the cuttings and would need dump trucks or lorries for its transportation. This could be only partly accommodated by the requirements of the mass haul and consequently tended to reduce the amount of work that could be carried out by motor scrapers. In the event more rock was encountered than anticipated at the east end of the contract and further reduced the amount of work carried out by motor scrapers. In Table A3.1 the more important items of plant used to excavate, load and transport the rock and soil are shown together with the major hauls.

Compaction

Example of the planning of a compaction scheme

A series of embankments are to be constructed of rock fill and well-graded granular soils. After the initial build up of plant, it is anticipated that between 15 000 m³ and 22 000 m³ of material is to be compacted per week. The types of rollers that should be used have to

Figure A3.1 Example of a large earthmoving project in South Wales



be determined. Rock fill and well-graded granular soils can be treated similarly for compaction purposes and are generally best compacted by vibrating rollers. Therefore a study into whether a towed or self-propelled roller would be more suited is to be carried out.

For both types of roller the specification defines the number of passes for a given layer thickness of fill at an operational speed of 1.5–2.5 km/h (average 2.0 km/h), and an overlap of 0.15 m each side of the roll is to be taken on each pass. The sizes are as given in Table A3.2.

The maximum thickness of compacted soil and minimum number of passes can be determined from the specification on compaction as shown in Table A3.3.

For towed rollers

$$\text{area covered/h} = \text{effective width of roller} \times \text{travel speed}$$

$$= (2.08 - 0.3) \times 2.0 \times 1000 \text{ m}^2/\text{h}$$

$$= 3560 \text{ m}^2/\text{h}$$

$$\text{total production/h} = \frac{3560 \times 0.275}{4} \text{ m}^3/\text{h} = 244.75 \text{ m}^3/\text{h}$$

$$\text{total production/week} = 244.75 \times 54 = 13\,217 \text{ m}^3/\text{week}$$

This makes no allowance for working at less than peak efficiency, starting, turning, break downs and so on. A correction is required to allow for 60% working efficiency

$$\text{corrected weekly production} = 13\,217 \times 0.6 \text{ m}^3/\text{week} = 7930 \text{ m}^3/\text{week}$$

Table A3.2

	Towed roller	Self-propelled double drum roller
Roll width	2.08 m	2.03 m
Weight/m width of roller	5600 kg/m	3000 kg/m

For self-propelled rollers

$$\begin{aligned}\text{area covered/h} &= (2.03 - 0.3) \times 2.0 \times 1000 \text{ m}^2/\text{h} \\ &= 3460 \text{ m}^2/\text{h}\end{aligned}$$

$$\text{total production/h} = \frac{3640 \times 0.2}{2} \text{ m}^3/\text{h} = 2.44.75 \text{ m}^3/\text{h} = 18\,684 \text{ m}^3/\text{week}$$

Allowing for the efficiency correction as before

$$\text{corrected weekly production} = 18\,684 \times 0.6 \text{ m}^3/\text{week} = 11\,210 \text{ m}^3/\text{week}$$

To achieve peak production either two self-propelled tandem rollers or towed vibrating rollers would be required. In this case the unit cost of operating a towed roller (including the crawler tractor) was 1.4 times that of the tandem roller and a significant saving could be made if two self-propelled tandem rollers were used. However, if the work had been spread over a number of embankments such that the rollers were required to be moved from area to area, it may have been more economical to use the three towed rollers.

Example on compaction practice

The example taken to illustrate compaction in practice concerns a motorway contract in south east England. The contract consisted of the construction of 2.25 km of motorway together with two roundabouts, their associated slip roads and a trunk road crossing the motorway. This involved the placing and compaction of over 700 000 m³ of bulk fill, selected backfill to structures, drainage blankets and fill for surcharging embankments and for landscaping purposes. The compaction of the fill was carried out with a number of different types of plant, which were selected to suit the materials being compacted, the volumes and rate of placement of fill requiring compaction, and the environment in which compaction was being carried out. A summary of the plant used is given in Table A3.4.

Table A3.3

	Towed roller	Self-propelled double drum roller
Maximum thickness of compacted soil	275 mm	200 mm
Minimum number of passes	4	2 ^a

^a For double drum rollers of equal mass/metre width the number of passes stated can be divided by 2

Table A3.4 Description of features, soil conditions and plant for earthmoving project in south of England

Area	Fill type	Deposited by	Compacted by	Layer thickness: mm	Number of passes
Roundabout 1					
Slip roads	Thanet Sand	Lorries/dump trucks	140 HP crawler tractor towing vibrating roller (3260 kg/m width)	275	8
Old chalk pits	Coarse sand–fine gravel (free draining)	Lorries	As above failed – then saturated material and compacted with tracks of 140 HP crawler tractor	As required	As required
Landscaping	Chalk, clay, rubble	Lorries	140 HP crawler tractor towing vibrating roller (3260 kg/m width)	200	2 ^a
			As above without vibration on roller	150	2–4 ^a
			140 HP crawler tractor towing 8000 kg smooth-wheeled roller	150	2–4 ^a
Motorway and side road embankment					
Start to roundabout 1	Thanet Sand	Motor scrapers	300 HP crawler tractor towing two vibrating rollers in tandem (both 5600 kg/m width)	100	1
Roundabout 1 to viaduct	Thanet Sand	Motor scrapers	300 HP crawler tractor towing two vibrating rollers in tandem (5600 kg/m plus 3260 kg/m)	100	1–2
			30 740 kg self-propelled tamping roller	250	4
	Brickearth	Tractor and box	140 HP crawler tractor towing 5500 kg towed tamping roller	225	8
Old chalk pits	Thanet Sand	Motor scrapers	140 HP crawler tractor towing 5600 kg/m width vibrating roller	300	4
Surcharge to embankments	Thanet Sand	Dump trucks	140 HP crawler tractor towing 3260 kg/m width vibrating roller	275	8

Table A3.4. Continued

Area	Fill type	Deposited by	Compacted by	Layer thickness: mm	Number of passes
Roundabout 2					
Slip roads	Thanet Sand	Tractor and box	140 HP crawler tractor towing 5600 kg/m width vibrating roller	300	4
		Dump trucks	170 HP special rubber-tyred tractor towing 3260 kg/m vibrating roller	275	8
	Gravelly clay	Tractor and box	140 HP crawler tractor towing 3260 kg/m vibrating roller	200	4
		Motor scraper	30 740 kg self-propelled tamping roller	225/150	4/12
Clay seal to contaminated area	London Clay	Lorries	140 HP crawler tractor towing 5500 kg tamping roller	225	4
Trunk road embankments	Thanet Sand	Dump trucks	170 HP special rubber-tyred tractor towing 3260 kg/m vibrating roller	275	8
Structures					
Granular backfill	Sandy gravel	Lorries	140 HP crawler tractor towing 3260 kg/m vibrating roller	200	4
			Manual 664 kg/m vibrating roller	75	12

^a Specification required compaction to be half of normal number of passes for landscaping fill

The majority of the fill used consisted of Thanet Sand. Where motor scrapers were employed to deliver the sand at high output rates, towed vibrating rollers in tandem or self-propelled tamping rollers were used, whereas when lorries, dump trucks or tractor and boxes were employed, single towed vibrating rollers were adopted. For other materials towed vibrating, smooth-wheeled (deadweight) and towed or self-propelled tamping rollers were used, depending on the nature of the soil and rate of production required.

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