Concise Eurocodes: Geotechnical design

BS EN 1997-1: Eurocode 7, Part 1

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Preface

1. Purpose

This book is intended to assist geotechnical designers by providing a reduced version of BS EN 1997-1 (EC7) together with added commentary by the author (which is differentiated from the Eurocode text by means of grey shading), giving explanations where the text could be found difficult to understand or to apply. It has been developed particularly for the use of engineers who are experienced in geotechnical design but want a quicker way to understand the procedures required by EC7. It is intended to aid use of EC7, rather than to provide background explanation.

Starting from the original text of BS EN 1997-1, material that is not needed in common design situations has been omitted. Omissions are marked by ** and the headings and numbering of EC7 is retained throughout. Cross-references have been added to other Euronorms, and to other documents, including those published by BSI, quoting text or tables of values from other Euronorms where this is helpful. All additions to EC7 are shown in shaded boxes.

The text of EC7 contains many checklists. For ease of use, these have been removed to Appendix 2 and arranged as forms for practical use.

This book also contains the most commonly needed content of the United Kingdom National Annex (UKNA) to EC7. In particular, Annex A of the European (CEN) version of EC7, which provides the values of partial factors, is replaced by the equivalent annex taken from the UKNA.

Designs for common situations based on this book will generally conform to EC7. However, **full conformance to the Eurocode can only be ensured by careful use of the complete version of BS EN 1997-1 and its National Annex.**

2. Geotechnical design

The subject of EC7 is ‘Geotechnical design’. In this context, a good definition of ‘design’ is ‘the process of decisions that determines what will be built’. The Eurocodes are particularly concerned with final design, which governs safety, and not so much with scheme designs and the preliminary designs used for the purposes of costing and tendering. Thus the Eurocodes provide a series of tests that the structure that is finally to be built is required to pass.

3. Safety format in EC7 calculations

Like the other Eurocodes, EC7 uses the approach of limit state design, generally with the method of partial factors as its format for prescribing safety. The fundamentals and basic terminology of these can be found in BS EN 1990. The following paragraphs provide a brief introduction for geotechnical designers.

The essence of limit state design is to set up ‘design situations’, which in some cases might be quite severe, and then to show that relevant limit states would not be exceeded if those situations were to occur.

For the typical designs considered by this book, the designer is required to consider ultimate limit state (ULS) and serviceability limit state (SLS) situations.

BS EN 1990 provides the following definitions:

1.5.2.12
limit states

states beyond which the structure no longer fulfils the relevant design criteria

1.5.2.13
ultimate limit states

states associated with collapse or with other similar forms of structural failure

1.5.2.14
serviceability limit states

states that correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met

These definitions are essentially non-technical. Ultimate limit states generally involve danger of casualties or severe economic loss. Serviceability limit states are less serious, involving inconvenience or disappointment, and any damage is often repairable. More detailed descriptions of ultimate and serviceability limit states are given in BS EN 1990, clauses 3.3 and 3.4. EC7 clause 2.4.7.1 notes five types of ULS relevant to geotechnical design.

Calculations for both ULS and SLS are carried out using *design values* of the parameters defining actions, materials, resistances and geometry. These are generally pessimistic values, incorporating all necessary factors or margins of safety, so that no further overall factor of safety is required. The design values for ULS and SLS will usually be different, generally more pessimistic for ULS since the occurrence of such a limit state would be a more serious event. The term *design value* as used in the Eurocodes is quite different from the traditional British usage of, for example, a design load, which would usually mean an unfactored load.

Most commonly, *design values* are derived by applying partial factors γ to *characteristic* material strengths or resistances, and to *representative* actions. In principle, this applies to both ULS and SLS calculations but, generally, all the values of γ for SLS are 1.0. Details of the application of γ factors can be found in 2.4.6 and 2.4.7. *Direct assessment of design values* is also allowed, in which the designer assesses a suitably pessimistic value without the use of factoring, as discussed under 2.4.6.

Characteristic values of geotechnical parameters are discussed further under 2.4.5.2. Representative values of actions are derived from characteristic values by applying load combination factors, as explained in BS EN 1990, Section 4.

The partial factor method was originally introduced in order to provide more consistent safety for a wide range of situations in which various parameters – actions (loads) and the way they combine, material properties, geometry – could be the dominant uncertainties in terms of their effects on overall safety. It is also thought that the method gives more opportunity for rational variation and development in situations where these uncertainties are unusually large or small.

In the author's understanding, the purposes of the partial factors include:

- Allowance for severe variations of leading parameters.
- Allowance for variation of secondary parameters that have not been factored. These often include small geometric variations.
- Some degree of control over displacements and the degree to which material strength is mobilised, affecting strains and distortions. Although these might be thought of as serviceability aspects, the factors adopted for ULS inevitably perform this role to some extent, and since they have generally been derived by calibration to previous practice, their values are affected by this requirement.
- Although not usually stated explicitly, the factors inevitably provide some degree of protection in case of errors in design or construction. This additional role should not be overlooked in cases where reduction in factor values is allowed or where direct assessment of design values is used.

The values of partial factors have *not* been chosen to allow for major uncertainty or systematic inaccuracy of calculation methods. Rather, it is required that all calculation methods are either accurate or err on the side of safety [2.4.1(6) to (9)].

4. Values of partial factors

The values of partial factors, and some other numerical parameters, are termed 'nationally determined parameters (NDP)'. The Eurocodes (as published by CEN and reprinted nationally) allow these values to be specified in national annexes but they also provide what are termed 'recommended values' for the partial factors. It should not be understood that these values are recommended for use in design. Rather, they are recommended by the original drafters of the Eurocodes for the consideration of national standards bodies. In this book, the 'recommended values' published by CEN are referred to as 'the CEN default values'.

Eurocodes are implemented as standards by national standards bodies. When using them, it is therefore important to use the appropriate national annex, which will usually be the national annex of the nation in which the construction site is located. Alternatively, the national annex to be used could be specified by contracts, depending on the legal constraints of each nation. The United Kingdom National Annex, published by BSI, will be referred to as the UKNA.

For the UK, a letter sent by the Department for Communities and Local Government to building control offices and others can be seen at http://www.communities.gov.uk/documents/planningandbuilding/pdf/1454859.pdf. This gives formal notification that Eurocodes are available for building control purposes and that the UKNA is to be used for construction in the UK. The letter also announces that some British Standards are withdrawn from 31 March 2010. Annex A states correctly that BS 8002 and BS 8004 are withdrawn but also states that BS 8006 and BS 8081 are withdrawn, which is not correct. The term 'withdrawn' means that BS 8002 and BS 8004 are no longer current British Standards and they will not be maintained further. It does not prevent their use in design but this could be open to the objection that they are no longer current.

If Eurocodes are used in non-European countries, it is important to establish which national annex is to be applied. Use of the CEN 'recommended values' (CEN default values) is strongly discouraged, especially for pile design, for which more conservative values have been specified by most countries.

5. Structural design in geotechnical situations

EC7 interfaces with other Eurocodes that give provision for design of structural elements of concrete or steel including foundations, retaining structures, pre-stressed members such as anchors, etc. The Eurocodes have been written so that the same partial factors on actions and on ground properties and resistances are applied to both geotechnical and structural design, consistently in EC7 and in the other Eurocodes. A further note relevant to the UK use of 'Design Approach 1' is provided under 2.4.7.3.4.2.

Contents

Section 1 General

1.1 Scope

1.1.1 Scope of EN 1997

(1) EN 1997 is intended to be used in conjunction with EN 1990:2002, which establishes the principles and requirements for safety and serviceability, describes the basis of design and verification and gives guidelines for related aspects of structural reliability.

(2) EN 1997 is intended to be applied to the geotechnical aspects of the design of buildings and civil engineering works.

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1.1.2 Scope of EN 1997-1

(1) EN 1997-1 is intended to be used as a general basis for the geotechnical aspects of the design of buildings and civil engineering works.

(2) The following subjects are dealt with in EN 1997-1:

Section 1: General

Section 2: Basis of geotechnical design

Section 3: Geotechnical data

Section 4: Supervision of construction, monitoring and maintenance

Section 5: Fill, dewatering, ground improvement and reinforcement

Section 6: Spread foundations

Section 7: Pile foundations

Section 8: Anchorages

Section 9: Retaining structures

Section 10: Hydraulic failure

Section 11: Overall stability

Section 12: Embankments

(3) EN 1997-1 is accompanied by Annexes A to J, which provide:

- in A: recommended partial factor values; different values of the partial factors may be set by the National annex;
- in B to J: supplementary informative guidance such as internationally applied calculation methods.

1.1.3 Further Parts of EN 1997

(1) EN 1997-1 is supplemented by EN 1997-2 that provides requirements for the performance and evaluation of field and laboratory testing.

1.2 Normative references

This clause provides references to the other Eurocodes and some of the geotechnical execution standards.

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1.3 Assumptions

The Eurocodes assume that data collection, design, construction and maintenance will all be carried out consistently with good practice. The codes do not state who carries responsibility for each of these aspects but it should be noted that they are addressed to designers. Adequate documentation of all these aspects is essential to good practice.

It is clearly impossible for designers to control construction and maintenance. However, besides ensuring that design is undertaken 'by appropriately qualified and experienced personnel', designers should demand that data collection is adequately carried out, which, in the geotechnical context, includes ground investigation. Guidance on investigations is provided by BS EN 1997-2 and by Uff and Clayton (1986, 1991).

Designers should also take responsibility for adequate specification of construction and the preparation and communication of manuals and other documents related to maintenance.

It is assumed that 'adequate continuity and communication exist between the personnel involved in data-collection, design and construction'. This can be challenging in situations in which structural and geotechnical design are carried out by different individuals or different companies. For example, clear communication of the actions to be applied to foundations is essential, requiring that all the engineers involved in the design understand and comply with the Eurocode system.

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1.4 Distinction between Principles and Application Rules

(1) Depending on the character of the individual clauses, distinction is made in EN 1997-1 between Principles and Application Rules.

(2) The Principles comprise:

— general statements and definitions for which there is no alternative;
— requirements and analytical models for which no alternative is permitted unless specifically stated.

(3) The Principles are preceded by the letter P.

(4) The Application Rules are examples of generally recognised rules, which follow the Principles and satisfy their requirements.

The Principles in EC7 are, in general, statements of essential good practice, on which the Application Rules expand.

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1.5 Definitions

1.5.1 Definitions common to all Eurocodes

(1) The definitions common to all Eurocodes are given in EN 1990:2002, 1.5.

1.5.2 Definitions specific for EN 1997-1

1.5.2.1
geotechnical action

action transmitted to the structure by the ground, fill, standing water or ground-water

The meaning of the term 'action' in the context of geotechnical design has been much debated. See comments in 2.4.2 for further information.

**

1.5.2.2
comparable experience

documented or other clearly established information related to the ground being considered in design, involving the same types of soil and rock and for which similar geotechnical behaviour is expected, and involving similar structures. Information gained locally is considered to be particularly relevant

Geotechnical design depends critically on the proper use of accumulated experience. However, this can become an impenetrable and irrational black box, sometimes termed 'engineering judgement'. EC7 attempts to define 'comparable experience' so as to distinguish experience that is of genuine relevance and also recorded in an objective fashion, so that it can be scrutinised and applied in an appropriate manner.

1.5.2.3
ground

soil, rock and fill in place prior to the execution of the construction works;

1.5.2.4
structure

organised combination of connected parts, including fill placed during execution of the construction works, designed to carry loads and provide adequate rigidity

Generally, the term 'structure' has to be understood in its context. This definition shows that it may include structural fill as, for example, in an embankment.

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1.5.2.5
derived value

value of a geotechnical parameter obtained by theory, correlation or empiricism from test results

1.5.2.6
stiffness

material resistance against deformation

1.5.2.7
resistance

The term 'resistance' is used to denote the capacity of a component or cross-section, such as bending resistance, buckling resistance, bearing resistance (or bearing capacity), pile resistance/capacity or passive resistance. It should be seen as distinct from the strength of material. Partial factors are applied either to the strength of material or to 'resistance', depending on the approach adopted.

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1.6 Symbols

In this text, symbols and abbreviations will be defined in context as they are needed. For the full list, see BS EN 1997-1.

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Section 2 Basis of geotechnical design

2.1 Design requirements

(1)P For each geotechnical design situation it shall be verified that no relevant limit state, as defined in EN 1990:2002, is exceeded.

The essence of limit state design is to set up 'design situations' which, in some cases, might be quite severe, and then to show that relevant limit states would not be exceeded if those situations were to exist.

(2) When defining the design situations and the limit states, the following factors should be considered:

- site conditions with respect to overall stability and ground movements;
- nature and size of the structure and its elements, including any special requirements such as the design life;
- conditions with regard to its surroundings (e.g.: neighbouring structures, traffic, utilities, vegetation, hazardous chemicals);
- ground conditions;
- ground-water conditions;
- regional seismicity;
- influence of the environment (hydrology, surface water, subsidence, seasonal changes of temperature and moisture).

This is one of many checklists provided in EC7. Although the items listed may sometimes seem obvious, they represent issues that have repeatedly caused failures when they have been overlooked in construction projects. A quick check against these lists is therefore strongly recommended.

To facilitate this, selected lists have been formatted as pro formas in Appendix 2. Users are encouraged to add their own items to these lists and blank lines are included for this purpose. If significant omissions are noted, readers are asked to propose them to BSI for incorporation in a future revision.

(3) Limit states can occur either in the ground or in the structure or by combined failure in the structure and the ground.

(4) Limit states should be verified by one or a combination of the following:

- use of calculations as described in 2.4;
- adoption of prescriptive measures, as described in 2.5;
- experimental models and load tests, as described in 2.6;
- an observational method, as described in 2.7.

(5) In practice, experience will often show which type of limit state will govern the design and the avoidance of other limit states may be verified by a control check.

(6) Buildings should normally be protected against the penetration of ground-water or the transmission of vapour or gases to their interiors.

(7) If practicable, the design results should be checked against comparable experience.

Remember that 'comparable experience' is specifically defined in 1.5.2.2.

(8) to (21) –these paragraphs introduce the concept of 'Geotechnical Categories'. The intention of these is to identify the degree of complexity of each design, varying from simple and conventional to unusual and difficult, so that suitably experienced and qualified engineers can be assigned to it and appropriate procedures can be used in design. It is important to assess complete designs, as well as each of their components in this way. The system of classification has not been used much in BS EN 1997-1 but it is used slightly more in BS EN 1997-2, in order to propose the amount of ground investigation and types of test to be undertaken.

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2.2 Design situations

(1)P Both short-term and long-term design situations shall be considered.

Paragraph (2) constitutes a checklist of items relevant to design situations, which is provided in a ready-to-use form in Appendix 2.

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2.3 Durability

(1)P At the geotechnical design stage, the significance of environmental conditions shall be assessed in relation to durability and to enable provisions to be made for the protection or adequate resistance of the materials.

Paragraph (2) constitutes a checklist of items relevant to durability, which is provided in a ready-to-use form in Appendix 2.

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(3) Reference should be made to durability provisions in construction materials standards.

2.4 Geotechnical design by calculation

2.4.1 General

(1)P Design by calculation shall be in accordance with the fundamental requirements of EN 1990:2002 and with the particular rules of this standard. Design by calculation involves:

— actions, which may be either imposed loads or imposed displacements, e.g. from ground movements;
— properties of soils, rocks and other materials;
— geometrical data;
— limiting values of deformations, crack widths, vibrations, etc.;
— calculation models.

BS EN 1990 indicates that calculations will normally be carried out using partial factor methods. This requires that parameters are first represented by 'characteristic' values and then factored to obtain 'design' values that are entered into calculations. As the factors have already been applied, no further overall 'factor of safety' is required. In principle, the process is the same for both ultimate and serviceability limit states, though in practice all the partial factors γ are generally set to 1.0 for serviceability calculations.

In this process, the partial factors may be applied to actions, action effects, material properties or resistances (i.e. capacities), as detailed later.

(2) It should be considered that knowledge of the ground conditions depends on the extent and quality of the geotechnical investigations. Such knowledge and the control of workmanship are usually more significant to fulfilling the fundamental requirements than is precision in the calculation models and partial factors.

(3)P The calculation model shall describe the assumed behaviour of the ground for the limit state under consideration.

(4)P If no reliable calculation model is available for a specific limit state, analysis of another limit state shall be carried out using factors to ensure that exceeding the specific limit state considered is sufficiently improbable. Alternatively, design by prescriptive measures, experimental models and load tests, or the observational method, shall be performed.

Paragraph (4) is mainly relevant to calculations for serviceability limit states. Displacements are often very difficult to calculate, so it is common practice to use a calculation for a plastic mechanism with a factor of safety applied to ensure that displacement will be 'small enough'. These ideas are developed in 2.4.8(4) and used in 6.6.2(16), for settlement of spread foundations on clays. Bolton has proposed that this approach can be put onto a rational framework for both foundations and retaining structures, and this thinking influenced the drafting of BS 8002 in 1994 (Bolton et al, 1990a, 1990b; Osman et al, 2004, 2006).

(5) The calculation model may consist of any of the following:

— an analytical model;
— a semi-empirical model;
— a numerical model.

(6)P Any calculation model shall be either accurate or err on the side of safety.

(7) A calculation model may include simplifications.

It is an underlying assumption of EC7 that any error in calculation methods is on the conservative side. The partial factor values provided are not intended to allow for any optimism in the calculation method. (See 'Safety format in EC7 calculations' in the Preface to this book for a discussion of the purposes of the factors.)

Paragraphs (8) and (9) note that where the calculation method is by nature optimistic (such as an upper bound method), it may be adjusted to become conservative by application of a 'model factor', used together with the other partial factors required by EC7. The calculation methods specifically recommended in EC7 do not fall into this category. In other situations in which the only calculation available is potentially optimistic, the value of model factor required should be judged by the designer; in some cases, reference to previous practice could inform this design.

The UKNA requires the use of model factors in pile design, since calculation of pile resistance from the results of ground tests is considered to be an uncertain process. As a further example, Figure X12 shows a raft foundation to be designed to accept a heavy load. The ground surface is irregular and soil strata, with markedly different properties, lie within a potential slip surface for bearing capacity. Since a simple calculation was not readily available, the designer wanted to use a method of slices slip circle calculation. However, comparisons with uniform material showed that this could be unconservative, so a model factor was introduced to prevent this inaccuracy having an unsafe effect.

(8) If needed, a modification of the results from the model may be used to ensure that the design calculation is either accurate or errs on the side of safety.

(9) If the modification of the results makes use of a model factor, it should take account of the following:

— the range of uncertainty in the results of the method of analysis;
— any systematic errors known to be associated with the method of analysis.

(10)P If an empirical relationship is used in the analysis, it shall be clearly established that it is relevant for the prevailing ground conditions.

As an example, one of the most commonly used empirical relationships in UK practice is the adhesion factor, α, used in pile design. It is important to be able to demonstrate that the value adopted has been calibrated in 'the prevailing ground conditions'.

(11) Limit states involving the formation of a mechanism in the ground should be readily checked using a calculation model. For limit states defined by deformation considerations, the deformations should be evaluated by calculation as described in 2.4.8, or otherwise assessed.

Limit states such as a bearing capacity failure or ultimate pile failure involve 'the formation of a mechanism in the ground' and these are usually ultimate limit states. Calculations can usually allow development of some plasticity but see the following note and paragraph (13) on the subject of ductility and brittleness.

Serviceability limit states such as excess settlement are ideally checked by making an estimate of the settlement, either by calculation or by other means, such as recorded observations. For example, observations of settlements related to embedded retaining walls are provided by CIRIA Report 580 (Gaba et al, 2003). Calculations often assume linear behaviour up to SLS but this is not a requirement of EC7 and may sometimes be an unnecessary restriction.

NOTE Many calculation models are based on the assumption of a sufficiently ductile performance of the ground/structure system. A lack of ductility, however, will lead to an ultimate limit state characterised by sudden collapse.

Ductility is of great value and should be a feature of good design as far as possible. Where it is impractical to have ductile structures or where the ground, by nature, is brittle, extra caution should be employed. This point is taken up again in paragraph (13) in the context of using numerical analysis.

(12) Numerical methods can be appropriate if compatibility of strains or the interaction between the structure and the soil at a limit state are considered.

(13) Compatibility of strains at a limit state should be considered. Detailed analysis, allowing for the relative stiffness of structure and ground, may be needed in cases where a combined failure of structural members and the ground could occur. Examples include raft foundations, laterally loaded piles and flexible retaining walls. Particular attention should be paid to strain compatibility for materials that are brittle or that have strain-softening properties.

In non-ductile situations, results of analysis may depend critically on relative stiffnesses of structural elements and the ground, which are difficult to prescribe accurately. Ductile structures can be analysed more reliably, especially for ULS.

(14) In some problems, such as excavations supported by anchored or strutted flexible walls, the magnitude and distribution of earth pressures, internal structural forces and bending moments depend to a great extent on the stiffness of the structure, the stiffness and strength of the ground and the state of stress in the ground.

(15) In these problems of ground-structure interaction, analyses should use stress-strain relationships for ground and structural materials and stress states in the ground that are sufficiently representative, for the limit state considered, to give a safe result.

2.4.2 Actions

(1)P The definition of actions shall be taken from EN 1990:2002. The values of actions shall be taken from EN 1991, where relevant.

'Actions' is the term used in Eurocodes for loads. They can be represented as forces, pressures, stresses or imposed displacements or strains. In the author's understanding, actions were originally defined such that their values were known at the start of the calculation and so factors could be applied to them. However, this definition has been eroded and factors are now placed on both known quantities and those derived during calculation. Opinions are divided as to the benefits of this more flexible approach.

As in his third law, Newton would have called the known values 'actions' and the derived values 'reactions'. In Eurocodes, 'reactions' are generally called 'action effects'.

In geotechnical calculations, the forces that pass between the structure and the ground are variously termed either 'actions' or 'action effects', and the differing 'Design Approaches' (see 2.4.7.3.4) specify whether factors are to be applied to them. Most importantly, paragraph (3) notes that ground-structure interaction should be considered in determining the values of such forces.

Paragraph (4) provides a checklist of actions that might have to be considered in geotechnical design. This is provided in a ready-to-use form in Appendix 2.

Paragraphs (5) to (9) warn that particularly careful consideration should be given to: variable actions in combination; actions that are time-dependent, applied repeatedly or rapidly enough to create a dynamic response; and actions due to water pressure.

An important note is added to paragraph (9) about the 'single source principle'. For further discussion of this, see the comments on 2.4.7.2.

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NOTE Unfavourable (or destabilising) and favourable (or stabilising) permanent actions may in some situations be considered as coming from a single source. If they are considered so, a single partial factor may be applied to the sum of these actions or to the sum of their effects.

2.4.3 Ground properties

EC7 anticipates that ground properties will be assessed by competent personnel, aiming to obtain values that represent the actual situation in the ground, which might be different from the values obtained directly in tests. This is discussed further below under 2.4.5.2. As various limit states are considered, such as settlements or gross failures, the values of ground properties selected for calculations should be relevant to the prevention of those particular limit states being exceeded.

Paragraph (4) constitutes a checklist of items that might cause differences between actual ground properties and test results. This is provided in a ready-to-use form in Appendix 2.

Paragraph (5) makes it clear that no legitimate source of information about ground parameters should be disregarded. Evaluation of parameters should not be restricted to sampling and testing at the construction site but should take account of all relevant information. This is developed further in 2.4.5.2.

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(5) When establishing values of geotechnical parameters, the following should be considered:

— published and well recognised information relevant to the use of each type of test in the appropriate ground conditions;
— the value of each geotechnical parameter compared with relevant published data and local and general experience;
— the variation of the geotechnical parameters that are relevant to the design;
— the results of any large scale field trials and measurements from neighbouring constructions;
— any correlations between the results from more than one type of test;
— any significant deterioration in ground material properties that may occur during the lifetime of the structure.

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2.4.4 Geometrical data

(1)P The level and slope of the ground surface, water levels, levels of interfaces between strata, excavation levels and the dimensions of the geotechnical structure shall be treated as geometrical data.

2.4.5 Characteristic values

Characteristic values are the initial 'unfactored', though generally cautious, values by which parameters are represented.

2.4.5.1 Characteristic and representative values of actions

(1)P Characteristic and representative values of actions shall be derived in accordance with EN 1990:2002 and the various parts of EN 1991.

EC7 does not discuss issues related to load combinations but refers the reader to BS EN 1990. This shows how combination factors ψ are applied to characteristic values of actions to obtain 'representative values', as noted below at equation (2.1b).

2.4.5.2 *Characteristic values of geotechnical parameters*

The selection of characteristic values for geotechnical parameters is critical to design and has been much debated. The problem is not new: representation of ground properties by numerical values for calculation has always been difficult and potentially contentious. The intention of the code drafters here was to define a value that could be used in a manner compatible with characteristic values in structural design, while being suited to the geotechnical context. The derivation of the value is not a simple statistical process and depends heavily on the knowledge and expertise of the designer.

The main points noted in this sub-clause are:

- Assessment of characteristic values should use available information, including test results, published data, results of field tests and monitoring exercises.
- The characteristic value is to be a 'cautious estimate' [paragraph (2)] of the value actually available in the ground to prevent the occurrence of a limit state. This will generally be more pessimistic than the 'most probable' value, the difference between the two depending on the level of uncertainty. Caution will usually require strengths lower than the most probable but, for some limit states, higher strengths may be more adverse for some of the parameters.
- The characteristic value has to be relevant to the limit state being considered, noting the extent of the ground involved. This means that a single soil stratum might have different characteristic values for different limit states or failure modes [see further under paragraph (7) below].
- All available information should be properly considered. Where there are other sources of relevant information, such as from previous experience or publications, the characteristic value is not based on test results alone [paragraph (4)].
- In some cases it might be helpful to use statistical methods, at least to give an initial suggestion of the characteristic value. For these, it is proposed [paragraph (11)] that the probability of a worse value occurring in such a way as to allow a limit state to occur should be not greater than 5%. Simpson et al (2009) summarise publications showing how this can be related to the standard deviation of the data. They also compare the use of a 'conservatively assessed mean' proposed by American authorities, which typically is defined to require a 75% exceedance by the test results. These approaches are only suitable when there are sufficient data points available and they require significant understanding of statistical methods on the part of the designer. It should be re-emphasised that the data used in this way are assumed to be representative of the real behaviour in the ground, which may require considerable processing of directly measured test results.

Characteristic angle of shearing resistance

In relation to angle of shearing resistance, the question has been asked: 'Which value is the characteristic value?' It is sometimes necessary to choose from one of the following, depending on circumstances:

— peak, critical state or residual shear strength;
— ultimate strength or a 'mobilised' value;
— strength of intact material or strength on joints;
— strength at first loading or after repeated loading.

In all cases, the answer of Eurocode 7 is: 'The one that is relevant to the prevention of the limit state under consideration.' EC7 does not differ in this respect from conventional practice. For some particular situations, the code is able to specify which of these values is relevant. For example, where concrete is to be cast against ground, which might therefore be disturbed, the critical state (or constant volume) value for the angle of shearing resistance, φ'_{cv}, is required (EC7, 6.5.3(10), 9.5.1(6)). In considering rocks, a study of the joint patterns will determine whether intact or joint strength is relevant (EC7, 3.3.8).

This answer to the question is not the same as: 'The one which would become relevant if the limit state was not prevented.' For example, in most plastic clays, if a slip occurred, the angle of shearing resistance would eventually fall to the residual value. Nevertheless, it is not necessary to design for residual strength in clays which have not previously slipped. Similarly, it may be unnecessary to design for critical state values, though brittleness and ductility should be considered, as noted in 2.4.1(11, 13).

Generally the strength to be used in Eurocode 7 is the maximum available to prevent collapse, not a mobilised value.

Because this sub-clause is particularly critical, it is retained here in full.

(1)P The selection of characteristic values for geotechnical parameters shall be based on results and derived values from laboratory and field tests, complemented by well-established experience.

(2)P The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state.

(3)P The greater variance of c' compared to that of $\tan\varphi'$ shall be considered when their characteristic values are determined.

Comments related to paragraph (3) are provided under Table A.NA.4.

(4)P The selection of characteristic values for geotechnical parameters shall take account of the following:

— geological and other background information, such as data from previous projects;
— the variability of the measured property values and other relevant information, e.g. from existing knowledge;
— the extent of the field and laboratory investigation;
— the type and number of samples;
— the extent of the zone of ground governing the behaviour of the geotechnical structure at the limit state being considered;
— the ability of the geotechnical structure to transfer loads from weak to strong zones in the ground.

(5) Characteristic values can be lower values, which are less than the most probable values, or upper values, which are greater.

(6)P For each calculation, the most unfavourable combination of lower and upper values of independent parameters shall be used.

(7) The zone of ground governing the behaviour of a geotechnical structure at a limit state is usually much larger than a test sample or the zone of ground affected in an in situ test. Consequently the value of the governing parameter is often the mean of a range of values covering a large surface or volume of the ground. The characteristic value should be a cautious estimate of this mean value.

As an example, Figure X13 shows a proposed building near the top of a slope. A single stratum is involved, consisting of sand with some occasional lenses of clay, considered to be weaker than the sand. The characteristic strength (or angle of shearing resistance) of the soil deposit is to be assessed for two ULS conditions: overall slope failure and local bearing failure of a footing. The overall failure will pass through a large amount of soil, so it is reasonable to take an average of the strengths of the sand and clay for this calculation (though it would be necessary to consider whether a failure mechanism could occur predominantly in the weaker material). For each footing, however, it is necessary to consider that it could be underlain by a clay lens, which would dominate the bearing resistance available to it. Thus a lower characteristic value for the soil should be taken for the footing than for the slope stability, despite the fact that both are in the same variable stratum. (There is a further possible alternative: probing could be carried out beneath each footing and any clay removed, ensuring good material for which a higher characteristic value could be used.)

(8) If the behaviour of the geotechnical structure at the limit state considered is governed by the lowest or highest value of the ground property, the characteristic value should be a cautious estimate of the lowest or highest value occurring in the zone governing the behaviour.

(9) When selecting the zone of ground governing the behaviour of a geotechnical structure at a limit state, it should be considered that this limit state may depend on the behaviour of the supported structure. For instance, when considering a bearing resistance ultimate limit state for a building resting on several footings, the governing parameter should be the mean strength over each individual zone of ground under a footing, if the building is unable to resist a local failure. If, however, the building is stiff and strong enough, the governing parameter should be the mean of these mean values over the entire zone or part of the zone of ground under the building.

(10) If statistical methods are employed in the selection of characteristic values for ground properties, such methods should differentiate between local and regional sampling and should allow the use of a priori knowledge of comparable ground properties.

(11) If statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%.

NOTE In this respect, a cautious estimate of the mean value is a selection of the mean value of the limited set of geotechnical parameter values, with a confidence level of 95%; where local failure is concerned, a cautious estimate of the low value is a 5% fractile.

(12)P When using standard tables of characteristic values related to soil investigation parameters, the characteristic value shall be selected as a very cautious value.

2.4.5.3 Characteristic values of geometrical data

(1)P Characteristic values of the levels of ground and ground-water or free water shall be measured, nominal or estimated upper or lower levels.

(2) Characteristic values of levels of ground and dimensions of geotechnical structures or elements should usually be nominal values.

2.4.6 Design values

Design values are the values actually entered into calculations. They are derived from the characteristic values (or, in the case of actions, from 'representative values') by applying partial factors γ. Thus 'design values' are factored values. This usage should not be confused with traditional UK usage in which 'design values' usually were more akin to the 'characteristic values' of the Eurocodes.

Design values are required for both ultimate and serviceability limit states. However, for SLS the factor γ is usually 1.0, making the design value numerically equal to the characteristic value or representative action.

'Direct assessment' of design values is also permitted. If this is done, the level of safety required should be respected and the code says that the values of the partial factors recommended in Annex A should be used as a guide to the required level of safety. The author recommends that, if direct assessment is used, the designer should be very confident that a worse value could not occur in practice. In fact, the design value should be even more severe than this, incorporating a margin to accommodate typical unplanned occurrences, i.e. it must have a degree of 'robustness' (Simpson et al, 2011). This issue is noted further below under 2.4.6.1(2) and 2.4.6.2(1).

2.4.6.1 Design values of actions

(1)P The design value of an action shall be determined in accordance with EN 1990:2002.

Particular care should be given to the concept of 'favourable' and 'unfavourable' actions, both permanent and variable. This classification may not be the same for both structural and geotechnical design. In particular, soil gains its strength mainly through friction, so an increase in direct stress may lead to an increase in strength and so would be 'favourable'. In some cases, it will not be obvious whether a particular action is favourable or unfavourable to the limit state considered, and it will then be necessary to consider both possibilities in forming load combinations for design.

(2)P The design value of an action (F_d) shall either be assessed directly or shall be derived from representative values using the following equation:

$$F_d = \gamma_F \cdot F_{rep} \tag{2.1a}$$

with

$$F_{rep} = \psi \cdot F_k \tag{2.1b}$$

For use of the ψ factor, see the note under 2.4.5.1(1), above.

For construction in the UK, values of γ_F and ψ are given in the UK National Annex to BS EN 1990:2002. For simple situations, values of γ_F for buildings are provided in Annex A of this book.

An alternative to the use of partial factors is allowed here. The designer may choose to select design values 'directly' without the process of multiplying representative values by partial factors γ. This approach is discussed above under 2.4.6. Paragraph (5) cautions that the level of safety required should be respected. Paragraph (6) invites use of this approach for water pressures. Attention is drawn to the discussion of the purpose of partial factors in the Preface to this book.

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(6)P When dealing with ground-water pressures for limit states with severe consequences (generally ultimate limit states), design values shall represent the most unfavourable values that could occur during the design lifetime of the structure. For limit states with less severe consequences (generally serviceability limit states), design values shall be the most unfavourable values which could occur in normal circumstances.

For ultimate limit states, the designer is asked to imagine the worst water pressures that are credible. For serviceability limit states, the requirement is the worst that is reasonably likely. These water pressures are then to be taken as 'design' values, meaning that no further factors are applied to them.

Paragraph (7) notes that some imaginable water pressures may be so unlikely that they could be treated as 'accidental' values. This implies that they are used in combination with less severe values of other variables, as noted in BS EN 1990:2002 (e.g. 6.4.3.3) and in 2.4.7.1 below.

Paragraph (8) states that design values of water pressures could be derived by factoring characteristic values. However, the UKNA advises against this (A2.1 and A3.1), preferring the use of paragraphs (2) and (6) or the use of a safety margin on water level as in paragraph (8). In effect, this approach treats the water level as a geometric parameter, consistently with 2.4.6.3(2) below. Similarly, PD 6694-1 states: 'because of the site-specific nature of uncertainty in water levels and the associated difficulties in calibration, no partial factor is given for ground-water pressure in the UK National Annex to BS EN 1990 for the design of bridges.'

These issues are discussed further in Simpson et al (2011).

(7) In some cases extreme water pressures complying with 1.5.3.5 of EN 1990:2002, may be treated as accidental actions.

(8) Design values of ground-water pressures may be derived either by applying partial factors to characteristic water pressures or by applying a safety margin to the characteristic water level in accordance with 2.4.4(1)P and 2.4.5.3(1)P.

(9) The following features, which may affect the water pressures should be considered:

— the level of the free water surface or the ground-water table;
— the favourable or unfavourable effects of drainage, both natural and artificial, taking account of its future maintenance;
— the supply of water by rain, flood, burst water mains or other means;
— changes of water pressures due to the growth or removal of vegetation.

The list in paragraph (9) is provided in a ready-to-use form in Appendix 2.

(10) Consideration should be given to unfavourable water levels that may be caused by changes in the water catchment and reduced drainage due to blockage, freezing or other causes.

(11) Unless the adequacy of the drainage system can be demonstrated and its maintenance ensured, the design ground-water table should be taken as the maximum possible level, which may be the ground surface.

2.4.6.2 Design values of geotechnical parameters

(1)P Design values of geotechnical parameters (X_d) shall either be derived from characteristic values using the following equation:

$$X_d = X_k / \gamma_M \qquad (2.2)$$

or shall be assessed directly.

As for actions in 2.4.6.1, an alternative to the use of partial factors is allowed here. The designer may choose to select design values 'directly' without the process of multiplying characteristic values by partial factors γ. This approach is discussed above under 2.4.6. Paragraph (3) cautions that the level of safety required should be respected.

For construction in the UK, values of γ_M are given in the UK National Annex to BS EN 1997-1:2004, Annex A (see pages 114–127).

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(3) If design values of geotechnical parameters are assessed directly, the values of the partial factors recommended in Annex A should be used as a guide to the required level of safety.

2.4.6.3 Design values of geometrical data

(1) The partial action and material factors (γ_F and γ_M) include an allowance for minor variations in geometrical data and, in such cases, no further safety margin on the geometrical data should be required.

(2)P In cases where deviations in the geometrical data have a significant effect on the reliability of a structure, design values of geometrical data (a_d) shall either be assessed directly or be derived from nominal values using the following equation (see 6.3.4 of EN 1990:2002):

$$a_d = a_{nom} \pm \Delta a \qquad (2.3)$$

for which values of Δa are given in 6.5.4(2) and 9.3.2.2

This sub-clause is taken up in relation to water pressure (2.4.6.1(8)), highly eccentric loading on spread foundations (6.5.4(2)) and the level of soil providing passive restraint to retaining walls (9.3.2.2).

2.4.6.4 Design values of structural properties

(1)P The design strength properties of structural materials and the design resistances of structural elements shall be calculated in accordance with EN 1992 to EN 1996 and EN 1999.

2.4.7 Ultimate Limit States

2.4.7.1 General

(1)P Where relevant, it shall be verified that the following limit states are not exceeded:

— loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance (EQU);
— internal failure or excessive deformation of the structure or structural elements, including e.g. footings, piles or basement walls, in which the strength of structural materials is significant in providing resistance (STR);
— failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance (GEO);
— loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions (UPL);
— hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients (HYD).

NOTE Limit state GEO is often critical to the sizing of structural elements involved in foundations or retaining structures and sometimes to the strength of structural elements.

For discussion of limit state EQU, see under 2.4.7.2.

In geotechnical design, it is often the case that a structure and the ground can only fail when both fail together, so limit states STR and GEO merge. In practice, the same factors are quoted for both.

Figures that clarify the definition of UPL and HYD are presented in Section 10, which is not included in this book.

Values of partial factors γ for construction in the UK are provided in Annex A of this book. Paragraphs (4) and (5) indicate that more, or less, severe values should be considered in exceptional circumstances. This issue is discussed in Annex B of BS EN 1990:2002, where the possibility of increasing or reducing factors on actions by 10% is suggested. Alternatives also noted are more thorough checking of design (e.g. third-party checking) and varying levels of supervision of design and inspection of construction. The UKNA adds no specific instruction but says that the values should be agreed with the client and relevant authorities.

A particular issue is the value of factors to be used in design for temporary situations, including 'temporary works'. Paragraph (5) states that factors can only be reduced 'where the likely consequences justify it'. In some cases, during site works the consequences of a failure may be less than in the finished project or an incipient failure may be preventable by the vigilance of site workers. In such situations, a reduction in values of partial factors may be acceptable but this is not on the basis of the temporary nature of the situation alone. This was discussed at length by Magnus et al (2005), as noted by Simpson et al (2008).

For accidental situations, the UKNA (Table NA.1) requires that the partial factor on all actions or the effects of actions is 1.0. The table is not fully clear what values are to be taken for resistances (and, by implication, material strengths) but the author understands it to mean that the values are to be the same as for STR and GEO.

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2.4.7.2 Verification of static equilibrium

Limit state EQU was devised to ensure safety in situations where equilibrium depends on a delicate balance of permanent loads. It is particularly relevant if the dominant permanent action comes from a 'single source', but could be divided into different parts. In such cases, the single permanent action has to be divided into parts to which differing partial factors are applied.

An example is shown in Figure X10. The balanced beam is a 'single source' represented as two equal forces: W. Ideally, this causes no bending moment in the column and equal compressive forces in the two piles. However, if some small variation in the load from the beam were to occur, it could cause significant bending moments in the column and variations of force in the piles, possibly putting one of them into tension.

In some cases, calculations that use the partial factors of EQU will lead the designer to ensure equilibrium by using the strength of some structural or ground material. For example, in Figure X10, EQU could create a demand, not otherwise needed, for bending strength in the column or for additional compressive or tensile resistance in the piles. Design for such situations is still under discussion among the developers of the Eurocodes.

EQU is called by BS EN 1990 a 'limit state'. However, in the view of the author it is better to understand EQU simply as an additional set of load factors. *In principle*, all designs must be able to accommodate the EQU load factors, though in practice it will be obvious by inspection in most cases that they are not critical.

Some geotechnical examples of EQU are discussed by Schuppener et al (2009) and by Simpson et al (2009). The values of partial factors for EQU in the UKNA (A.2 in the UKNA Annex A on pages 114–116 of this book) have been chosen with the aim of preventing EQU appearing to be critical when it was not intended to be so.

(1)P When considering a limit state of static equilibrium or of overall displacements of the structure or ground (EQU), it shall be verified that:

$$E_{dst;d} \leq E_{stb;d} + T_d \tag{2.4}$$

with

$$E_{dst;d} = E\{\gamma_F F_{rep} ; X_k / \gamma_M ; a_d\}_{dst} \tag{2.4a}$$

and

$$E_{stb;d} = E\{\gamma_F F_{rep} ; X_k / \gamma_M ; a_d\}_{stb} \tag{2.4b}$$

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2.4.7.3 Verification of resistance for structural and ground limit states in persistent and transient situations

2.4.7.3.1 General

(1)P When considering a limit state of rupture or excessive deformation of a structural element or section of the ground (STR and GEO), it shall be verified that:

$$E_d \leq R_d \tag{2.5}$$

Equation 2.5 is the basic check required for most design calculations. It simply states that at critical points in the ground/structural system, it shall be checked that the available design (i.e. factored) resistance is greater than the design effect of all the loading (including partial load factors). No further factor of safety is required.

'R_d' could represent design resistances such as the bearing capacity of a spread foundation, the capacity of a pile or pile group or the bending resistance of a retaining wall. E_d would then be the design value of the combined action effects exploiting that particular resistance.

Requirement 2.5 has to hold at *all* points in the ground/structural system – the available strength can never be exceeded.

When all possible partial factors are incorporated, equation 2.5 could be expanded as:

$$\gamma_E\, E\{\gamma_F\, F_{rep};\, X_k/\gamma_M;\, a_d\} = E_d \leq R_d = R\{\gamma_F\, F_{rep};\, X_k/\gamma_M;\, a_d\}/\gamma_R \qquad (2.5^*)$$

where $E\{\,\}$ indicates a calculation deriving action effects from actions (e.g. a load take-down) and $R\{\,\}$ shows a calculation deriving resistance from material strengths.

Requirement 2.5* is never used in this form and simplified versions are presented with fewer γ factors. This form of the requirement shows a factor γ_F applied to the actions F together with γ_E applied to the derived action effect, E. Both of these are allowed by EC7 but, in practice, only one of these factors is applied. Similarly, when determining R_d, only one of γ_M and γ_R is used.

However, a notable feature of equation 2.5* is important to geotechnical design: mainly because soil is a frictional material, resistance is sometimes a function of actions and action effects are sometimes a function of material strengths. Examples include sliding resistance, which is dependent on the loads acting normally to the sliding surface, and earth pressures, which are dependent on soil strength.

The values of factors to be used in these equations are given in Annex A of this book (A.3).

2.4.7.3.2 Design effects of actions

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(2) In some design situations, the application of partial factors to actions coming from or through the soil (such as earth or water pressures) could lead to design values, which are unreasonable or even physically impossible. In these situations, the factors may be applied directly to the effects of actions derived from representative values of the actions.

In geotechnical calculations, confusion often arises if load factors are applied to the weight, or density, of the ground or ground-water. Paragraph (2) provides a way of avoiding this by factoring the resulting action effects. This is important in the design of retaining structures, for example, where this paragraph makes it permissible to factor resulting bending moments, shear forces and strut forces after they have been calculated, for example, rather than pre-factoring the density of the ground before the calculation. It is the author's recommendation that this approach be used with Design Approach 1, Combination 1. In Combination 2, permanent actions have a load factor of 1.0, so the issue does not arise.

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2.4.7.3.3 Design resistances

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2.4.7.3.4 Design Approaches

2.4.7.3.4.1 General

The way in which requirement 2.5* is applied is governed by the choice of one of three available 'Design Approaches' (DA1, DA2, DA3). The UKNA specifies that DA1 shall be used for construction in the UK. The main reasons for this choice are discussed by Simpson (2007). Simpson et al (2009) summarise them as follows.

- It facilitates consistent analysis of combined problems, which are very common, involving, for example, a slope, loaded by a structure, supported by a retaining wall, itself supported by anchors and foundations.
- Because the strength of soil is derived from friction, non-linear or disproportionate relationships between soil parameters and resistances are common. In these circumstances, it is important to check designs with partial factors applied to the basic strength parameters of the soil, as explained fully in BS EN 1990 (6.3.2). Such non-linear relationships occur, for example in the derivation of bearing capacity factors (e.g. N_γ) or of coefficient of passive resistance (K_p) from angle of shearing resistance, as illustrated in Figure X14. (K_p is plotted for $\delta/\phi=⅔$.)
- It can be used readily with both simple hand calculations and more complex finite element calculations. Introduction of resistance factors into finite element computations, which essentially require overall equilibrium, has proved to be difficult. Demonstration of equilibrium of complete systems is fundamental to good design practice.

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2.4.7.3.4.2 Design Approach 1

DA1 requires that, for ULS, designs are checked, in principle, by two separate calculations involving different values for partial factors γ. In 'Combination 1' (C1), actions are factored, much as for structural design, and the strength of the ground is not factored. In 'Combination 2' (C2), the strength of the ground is factored and the factor on actions is 1.0, except that a small value (γ_Q = 1.3) is applied to unfavourable variable actions, and favourable variable actions are omitted – i.e. γ_Q = 0. For piles and anchors, the material (or resistance) factor is effectively applied to the soil at the interface between the structure and the ground and, for this, special values are provided.

If it is obvious that one of the two combinations governs the design, calculations for the other combination need not be carried out. In many cases, a little experience will soon show which one is likely to be critical.

Annexes A of EC7, the UKNA and this book refer to sets of partial factors to be used in various combinations: A1, A2, etc. for actions, M1, M2 for materials and R1, R4 for resistances. (Sets R2 and R3 are not used in DA1.) The 'A' factors are sometimes applied directly to actions (as γ_F in requirement 2.5*) and sometimes to action effects (as γ_E). The values of the factors required for design of buildings by the UKNA are summarised in Figure X1.

In general, the factors on actions in C1 are the same as used in structural design, ensuring that foundations can always accept the ULS design loads from supported structures, albeit with no further factor of safety in the ground. In most cases, the check on the strength of the ground is more critical in C2 than in C1. Forces or stress distributions are derived from the ground in retaining structures and as reactions in foundations. These are obtained from both C1 and C2 and should be treated as ULS *design* values for checking structures, implying that no further load factor is to be applied to them. The more critical design loading for structures may come from either C1 or C2.

Because DA1 is the UK choice, the full text of 2.4.7.3.4.2 is retained here.

(1)P Except for the design of axially loaded piles and anchors, it shall be verified that a limit state of rupture or excessive deformation will not occur with either of the following combinations of sets of partial factors:

Combination 1: *A1* "+" *M1* "+" *R1*

Combination 2: *A2* "+" *M2* "+" *R1*

where "+" implies: "to be combined with".

NOTE In Combinations 1 and 2, partial factors are applied to actions and to ground strength parameters.

(2)P For the design of axially loaded piles and anchors, it shall be verified that a limit state of rupture or excessive deformation will not occur with either of the following combinations of sets of partial factors:

Combination 1: *A1* "+" *M1* "+" *R1*

Combination 2: *A2* "+" (*M1* or *M2*) "+" *R4*

NOTE 1 In Combination 1, partial factors are applied to actions and to ground strength parameters. In Combination 2, partial factors are applied to actions, to ground resistances and sometimes to ground strength parameters.

NOTE 2 In Combination 2, set *M1* is used for calculating resistances of piles or anchors and set *M2* for calculating unfavourable actions on piles owing e.g. to negative skin friction or transverse loading.

(3) If it is obvious that one of the two combinations governs the design, calculations for the other combination need not be carried out. However, different combinations may be critical to different aspects of the same design.

2.4.7.3.4.3 Design Approach 2

DA2 requires a single calculation, in which factors are applied to actions and to resistances, such as bearing capacity and passive resistance. It has been found that it is difficult to apply this approach to slope stability problems and for finite element analysis, so most of the countries that have adopted DA2 use DA3 for these.

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2.4.7.3.4.4 Design Approach 3

In DA3, factors are applied to both actions and material strengths in the same calculation. This was avoided in the UKNA because, if the factors are sufficient to cover all situations, it tends to be too conservative in many cases.

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2.4.7.4 Verification procedure and partial factors for uplift

(1)P Verification for uplift (UPL) shall be carried out by checking that the design value of the combination of destabilising permanent and variable vertical actions ($V_{dst;d}$) is less than or equal to the sum of the design value of the stabilising permanent vertical actions ($G_{stb;d}$) and of the design value of any additional resistance to uplift (R_d):

$$V_{dst,d} \leq G_{stb;d} + R_d \tag{2.8}$$

where

$$V_{dst,d} = G_{dst;d} + Q_{dst;d}$$

Like EQU, UPL is about the balance of forces, generally with little involvement of material strength but, in this case, one of the main forces comes from water pressure. Figure 10.1 of EC7 clarifies the definition of the limit state. Although not explicitly stated, it is understood that the partial factors are to be applied to representative actions or action effects. That is:

$$V_{dst,d} = \gamma_{G;dst}\, G_{dst;rep} + \gamma_{Q;dst}\, Q_{dst;rep}$$

and $G_{stb;d} = \gamma_{G;stb}\, G_{stb;rep}$

In the UKNA (Table A.NA.15), $\gamma_{G;dst} = 1.1$, $\gamma_{Q;dst} = 1.5$ and $\gamma_{G;stb} = 0.9$. The UKNA requires that any resistance involved, R_d, is calculated using the same factors as for STR/GEO (Table A.NA.16 – see Figure X1). This is contrary to paragraph (2), below, which gives an alternative approach that is not recommended.

(2) Additional resistance to uplift may also be treated as a stabilising permanent vertical action ($G_{stb;d}$).

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2.4.7.5 *Verification of resistance to failure by heave due to seepage of water in the ground*

Limit state HYD refers to hydraulic uplift. Figure 10.2 of EC7 clarifies the definition of the limit state.

(1)P When considering a limit state of failure due to heave by seepage of water in the ground (HYD, see 10.3), it shall be verified, for every relevant soil column, that the design value of the destabilising total pore water pressure ($u_{dst;d}$) at the bottom of the column, or the design value of the seepage force ($S_{dst;d}$) in the column is less than or equal to the stabilising total vertical stress ($\sigma_{stb;d}$) at the bottom of the column, or the submerged weight ($G'_{stb;d}$) of the same column:

$$u_{dst;d} \leq \sigma_{stb;d} \tag{2.9a}$$

$$S_{dst;d} \leq G'_{stb;d} \tag{2.9b}$$

With consistent design values, these two formulations are physically equivalent, but Orr (2005) demonstrated that very different design results could result, depending on how factors are applied. Partial factor values are provided: the UKNA (A.5) gives $\gamma_{G;dst} = 1.35$ and $\gamma_{G;stb} = 0.9$. Unfortunately, the code is not explicit about where the factors are to be applied and this has led to much debate. Should factors be applied to water pressures, effective (or buoyant) weights, total stresses, total weights, seepage forces ... ?

Unreasonable results tend to emerge if factors are applied to the density of water. The following approach is recommended.

a) *Design* ground-water pressures, piezometric levels or seepage forces are better assessed directly, rather than by applying partial factors [see comments on 2.4.6.1(6)].
b) If fixed safety elements are proposed for these, additive margins are preferable to multiplying factors [see comments on 2.4.6.1(8)].
c) Similar approaches could also be used for γ or γ' (noting that additive margins produce the same result when applied to either parameter) but this is not too important as the value proposed is close to unity – 0.9.
d) If it is required to apply partial factors to γ or γ', using γ may provide a more appropriate and consistent margin of safety.

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2.4.8 Serviceability limit states

In the context of EC7, serviceability limit states relate either to ground displacement (settlement, lateral displacement of retaining walls, etc.) or to serviceability damage in structures (cracking, etc.). In terms of calculation, they are to be prevented either by a cautious calculation of likely displacements [paragraph (1)] or by ensuring that a sufficiently low fraction of the ground strength is mobilised [paragraph (4)].

Although not stated explicitly in the code, serviceability may also be justified by using 'prescriptive measures' as defined in 2.5. The 'indirect method' noted in 6.4(5) is one example of this, related to settlement of spread foundations.

(1)P Verification for serviceability limit states in the ground or in a structural section, element or connection, shall either require that:

$$E_d \leq C_d, \tag{2.10}$$

or be done through the method given in 2.4.8(4).

(2) Values of partial factors for serviceability limit states should normally be taken equal to 1,0.

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(3) Characteristic values should be changed appropriately if changes of ground properties e.g. by ground-water lowering or desiccation, may occur during the life of the structure.

(4) It may be verified that a sufficiently low fraction of the ground strength is mobilised to keep deformations within the required serviceability limits, provided this simplified approach is restricted to design situations where:

— a value of the deformation is not required to check the serviceability limit state;
— established comparable experience exists with similar ground, structures and application method.

(5)P A limiting value for a particular deformation is the value at which a serviceability limit state, such as unacceptable cracking or jamming of doors, is deemed to occur in the supported structure. This limiting value shall be agreed during the design of the supported structure.

2.4.9 Limiting values for movements of foundations

(1)P In foundation design, limiting values shall be established for the foundation movements.

NOTE Permitted foundation movements may be set by the National annex.

The UKNA does not provide limiting values for movements but refers the reader to Annex H of EC7. In practice, limiting values are dependent on the type of supported structure and should be agreed with building designers and owners. Guidance is given by CIRIA Report C580 section 2.5.4 (Gaba et al, 2003).

(2)P Any differential movements of foundations leading to deformation in the supported structure shall be limited to ensure that they do not lead to a limit state in the supported structure.

(3)P The selection of design values for limiting movements and deformations shall take account of the following:

— the confidence with which the acceptable value of the movement can be specified;
— the occurrence and rate of ground movements;
— the type of structure;
— the type of construction material;
— the type of foundation;
— the type of ground;
— the mode of deformation;
— the proposed use of the structure;
— the need to ensure that there are no problems with the services entering the structure.

The checklists of paragraphs (3) and (4) are provided in ready-to-use forms in Appendix 2.

(4)P Calculations of differential settlement shall take account of:

— the occurrence and rate of settlements and ground movements;
— random and systematic variations in ground properties;
— the loading distribution;
— the construction method (including the sequence of loading);
— the stiffness of the structure during and after construction.

NOTE In the absence of specified limiting values of structural deformations of the supported structure, the values of structural deformation and foundation movement given in Annex H may be used.

2.5 Design by prescriptive measures

(1) In design situations where calculation models are not available or not necessary, exceeding limit states may be avoided by the use of prescriptive measures. These involve conventional and generally conservative rules in the design, and attention to specification and control of materials, workmanship, protection and maintenance procedures.

NOTE Reference to such conventional and generally conservative rules may be given in the National annex.

(2) Design by prescriptive measures may be used where comparable experience, as defined in 1.5.2.2, makes design calculations unnecessary. It may also be used to ensure durability against frost action and chemical or biological attack, for which direct calculations are not generally appropriate.

The use of prescriptive measures is illustrated in EC7 by:

- 6.4(5) and 6.5.2.4 – design of spread foundations;
- 10.5(1) – design against piping failure.

The concept has been developed in practice by Wong et al (1999) for slopes and retaining walls in Hong Kong.

No additional information is given by the UKNA.

2.6 Load tests and tests on experimental models

Load testing of structural elements is particularly important for piles and ground anchors, so these principles are expanded in sections 7 and 8. Full scale tests or smaller scale models are sometimes used in the design of embankments.

(1)P When the results of load tests or tests on large or small scale models are used to justify a design, or in order to complement one of the other alternatives mentioned in 2.1(4), the following features shall be considered and allowed for:

— differences in the ground conditions between the test and the actual construction;
— time effects, especially if the duration of the test is much less than the duration of loading of the actual construction;
— scale effects, especially if small models are used. The effects of stress levels shall be considered, together with the effects of particle size.

(2) Tests may be carried out on a sample of the actual construction or on full scale or smaller scale models.

2.7 Observational method

Reference is often made to the 'observational method' (OM) during geotechnical design and construction; some recent texts call the process 'Interactive design'. Too often, this involves monitoring without any ability to react to the results, if adverse, or even without proper scrutiny of them (e.g. Health and Safety Executive, 2000; Magnus et al, 2005; Simpson et al, 2008). The method can be used to great advantage, especially in the common situation where there is a wide range of uncertainty in design parameters but the possibility exists to amend the design during construction if observations suggest this would be beneficial to improve either safety or economy (Peck, 1969; Nicholson et al, 1999).

The clauses provided here give the essential framework for a successful application of the method, particularly in situations where a design is implemented that is less cautious than it could have been without the OM.

(1) When prediction of geotechnical behaviour is difficult, it can be appropriate to apply the approach known as "the observational method", in which the design is reviewed during construction.

(2)P The following requirements shall be met before construction is started:

— acceptable limits of behaviour shall be established;
— the range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits;
— a plan of monitoring shall be devised, which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully;
— the response time of the instruments and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system;
— a plan of contingency actions shall be devised, which may be adopted if the monitoring reveals behaviour outside acceptable limits.

(3)P During construction, the monitoring shall be carried out as planned.

(4)P The results of the monitoring shall be assessed at appropriate stages and the planned contingency actions shall be put into operation if the limits of behaviour are exceeded.

(5)P Monitoring equipment shall either be replaced or extended if it fails to supply reliable data of appropriate type or in sufficient quantity.

2.8 Geotechnical Design Report

It is a code requirement that a design report is produced, cross-referring to the Ground Investigation Report (see 3.4). It is assumed, though not explicitly stated, that responsibility for producing this report will rest with the party taking responsibility for design.

(1)P The assumptions, data, methods of calculation and results of the verification of safety and serviceability shall be recorded in the Geotechnical Design Report.

(2) The level of detail of the Geotechnical Design Reports will vary greatly, depending on the type of design. For simple designs, a single sheet may be sufficient.

Paragraph (3) constitutes a checklist of items to be included in the Geotechnical Design Report, which is provided in a ready-to-use form in Appendix 2.

**

(4)P The Geotechnical Design Report shall include a plan of supervision and monitoring, as appropriate. Items, which require checking during construction or, which require maintenance after construction shall be clearly identified. When the required checks have been carried out during construction, they shall be recorded in an addendum to the Report.

Paragraph (5) constitutes a checklist of items relevant to supervision and monitoring, which is provided in a ready-to-use form in Appendix 2.

**

(6)P An extract from the Geotechnical Design Report, containing the supervision, monitoring and maintenance requirements for the completed structure, shall be provided to the owner/client.

Section 3 Geotechnical data

Section 3 contains basic requirements for good practice in the process of collecting data and deriving values for parameters. The requirements it states are important but they are not particular to EC7. They might be regarded as well known and obvious by most engineers with some geotechnical training. Failure to satisfy these requirements would, of course, constitute non-conformance with the Eurocode.

Section 3 is considerably supplemented by BS EN 1997-2 and may be revised in the future to acknowledge this more clearly, probably reducing the section itself.

For ground investigation in the UK, BS 5930 was extensively revised and republished in 2009. This provides a much more detailed treatment of the subject of Section 3.

**

3.4 Ground Investigation Report

It is a code requirement that a Ground Investigation Report is produced, cross-referring to the Geotechnical Design Report (see 2.8). As for other requirements, the code does not assume a particular contract set-up, so it does not prescribe who is to produce this report.

3.4.1 Requirements

(1)P The results of a geotechnical investigation shall be compiled in a Ground Investigation Report, which shall form a part of the Geotechnical Design Report described in 2.8.

(2)P Reference shall be made to EN 1997-2 for information on the use of laboratory and field tests for geotechnical parameters.

(3) The Ground Investigation Report should normally consist of:

— a presentation of all available geotechnical information including geological features and relevant data;
— a geotechnical evaluation of the information, stating the assumptions made in the interpretation of the test results.

The information may be presented as one report or as separate parts.

3.4.2 Presentation of geotechnical information

This sub-clause constitutes checklists of items relevant to presentation of factual data, which are provided in a ready-to-use form in Appendix 2.

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3.4.3 Evaluation of geotechnical information

This sub-clause constitutes checklists of items relevant to presentation of interpretative data, which are provided in a ready-to-use form in Appendix 2.

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Section 4 Supervision of construction, monitoring and maintenance

Supervision of construction, monitoring of performance and provision of long-term maintenance are intrinsic parts of geotechnical design. In this context, a good definition of 'design' is 'the process of decisions that determines what will be built'. Under this definition, design often continues during construction as ground conditions are revealed, leading to adjustments of what is built. Performance monitoring may lead to further changes, especially if the *observational method*, as defined in 2.7, is adopted.

Paragraph 4.1(2) requires that necessary supervision and monitoring are specified in the Geotechnical Design Report (see 2.8), making this specification the responsibility of the designer. In addition to supervision and monitoring, it is essential to have the facility, both contractual and practical, to respond to adverse observations.

Overall, the emphasis is on doing what is appropriate in particular circumstances and keeping good records. The term 'as appropriate' is frequently used. Reference is made to the Geotechnical Categories but distinctions between the actions noted are fairly imprecise and not mandatory.

Because it is fairly brief but sets out important principles, Section 4 is retained here in full.

4.1 General

(1)P To ensure the safety and quality of a structure, the following shall be undertaken, as appropriate:

— the construction processes and workmanship shall be supervised;
— the performance of the structure shall be monitored during and after construction;
— the structure shall be adequately maintained.

(2)P Supervision of the construction process, including workmanship, and any monitoring of the performance of the structure during and after construction, shall be specified in the Geotechnical Design Report.

(3) Supervision of the construction process, including workmanship, should involve the following, as appropriate:

— checking the validity of the design assumptions;
— identifying the differences between the actual ground conditions and those assumed in the design;
— checking that the construction is carried out according to the design.

(4) Observations and measurements of the behaviour of the structure and its surroundings should be made, as appropriate:

— during construction, to identify any need for remedial measures or alterations to the construction sequence, for example;
— during and post construction, to evaluate the long-term performance.

(5)P Design decisions, which are influenced by the results of the supervision and monitoring shall be clearly identified.

(6) The amount of construction supervision and the quantity of field and laboratory testing required to control and monitor performance should be planned during the design stage.

(7)P In the case of unexpected events, the methods, extent and frequency of monitoring shall be reviewed.

(8)P The level and quality of supervision and monitoring shall be at least equal to those assumed in the design and shall be consistent with the values selected for the design parameters and partial factors.

NOTE Annex J gives a checklist for construction supervision and performance monitoring.

4.2 Supervision

4.2.1 Plan of supervision

(1)P The plan included in the Geotechnical Design Report shall state acceptable limits for the results to be obtained by the supervision.

(2) The plan should specify the type, quality and frequency of supervision, which should be commensurate with:

— the degree of uncertainty in the design assumptions;
— the complexity of the ground and loading conditions;
— the potential risk of failure during construction;
— the feasibility of implementing design modifications or corrective measures during construction.

4.2.2 Inspection and control

(1)P The construction work shall be inspected on a continuous basis and the results of the inspection shall be recorded.

(2) For Geotechnical Category 1, the supervision programme may be limited to inspection, simple quality controls and a qualitative assessment of the performance of the structure.

(3) For Geotechnical Category 2, measurements of ground properties or the behaviour of structures should often be required.

(4) For Geotechnical Category 3, additional measurements should be required during each significant stage of construction.

(5)P Records shall be maintained of the following, as appropriate:

- significant ground and ground-water features;
- sequence of works;
- quality of materials;
- deviations from design;
- as-built drawings;
- results of measurements and of their interpretation;
- observations of the environmental conditions;
- unforeseen events.

(6) Records of temporary works should also be kept. Interruptions to the works, and their condition on re-commencement, should be recorded.

(7)P The results of the inspection and control shall be made available to the designer before any changes are decided.

(8) In general, the design documents and records of what was constructed should be stored for 10 years, unless agreed otherwise. More important documents should be stored for the lifetime of the relevant structure.

4.2.3 Assessment of the design

(1)P The suitability of the construction procedures and the sequence of operations shall be reviewed in the light of the ground conditions, which are encountered; the predicted behaviour of the structure shall be compared with the observed performance. The design shall be assessed on the basis of the results of the inspection and supervision.

(2) The assessment of the design should include a careful review of the most unfavourable conditions, which occur during construction with regard to:

- ground conditions;
- ground-water conditions;
- actions on the structure;
- environmental impacts and changes including landslides and rockfalls.

4.3 Checking ground conditions

4.3.1 Soil and rock

(1)P The descriptions and geotechnical properties of the soils and rocks in or on which the structure is founded or located shall be checked during construction.

(2) For Geotechnical Category 1, the descriptions of the soils and rocks should be checked by:

— inspecting the site;
— determining the types of soil and rock within the zone of influence of the structure;
— recording descriptions of the soil and rock exposed in excavations.

(3) For Geotechnical Category 2, the geotechnical properties of the soil or rock in or on which the structure is founded or located should also be checked. Additional site investigation may be needed. Representative samples should be recovered and tested to determine the index properties, strength and deformability.

(4) For Geotechnical Category 3, additional requirements should include further investigations and examination of details of the ground or fill conditions, which may have important consequences for the design.

(5) Indirect evidence of the geotechnical properties of the ground (for example, from pile driving records) should be recorded and used to assist in interpreting the ground conditions.

(6)P Deviations from the ground type and properties assumed in the design shall be reported without delay.

NOTE Normally these deviations are reported to the designer.

(7)P The principles used in design shall be checked to ensure that they are appropriate for the geotechnical features of the ground, which are encountered.

4.3.2 Ground-water

(1)P As appropriate, the ground-water levels, pore-water pressures and ground-water chemistry encountered during execution shall be compared with those assumed in the design.

(2) More thorough checks should be performed for sites on which significant variations of ground type and permeability are known or believed to exist.

(3) For Geotechnical Category 1, checks should usually be based on previously documented experience in the area or on indirect evidence.

(4) For Geotechnical Categories 2 and 3, direct observations should normally be made of the ground-water conditions if these greatly affect either the method of construction or the performance of the structure.

(5) Ground-water flow characteristics and the pore-water pressure regime should be obtained by means of piezometers, which preferably should be installed before the start of construction operations. It may sometimes be necessary to install piezometers at large distances from the site as part of the monitoring system.

(6) If pore-water pressure changes occur during construction that may affect the performance of the structure, pore-water pressures should be monitored until construction is complete or until the pore-water pressures have dissipated to safe values.

(7) For structures below ground-water level, which may be subject to uplift, pore-water pressures should be monitored until the weight of the structure is sufficient to rule out the possibility of uplift.

(8) Chemical analysis of mobile water should be performed when any part of the permanent or temporary works may be significantly affected by chemical attack.

(9)P The effect of construction operations (including processes such as dewatering, grouting and tunnelling) on the ground-water regime shall be checked.

(10)P Deviations from the ground-water features assumed in the design shall be reported without delay.

(11)P The principles used in design shall be checked to ensure that they are appropriate for the ground-water features, which are encountered.

4.4 Checking construction

(1)P Site operations shall be checked for compliance with the method of construction assumed in the design and stated in the Geotechnical Design Report. Observed differences between the design assumptions and the site operations shall be reported without delay.

(2)P Deviations from the methods of construction assumed in the design and stated in the Geotechnical Design Report shall be explicitly and rationally considered and implemented.

(3)P The principles followed in design shall be checked to ensure that they are appropriate for the sequence of construction operations, which are used.

(4) For Geotechnical Category 1, a formal construction schedule need not normally be included in the Geotechnical Design Report.

NOTE The sequence of construction operations is normally decided by the contractor.

(5) For Geotechnical Categories 2 and 3, the Geotechnical Design Report may give the sequence of construction operations envisaged in the design.

NOTE Alternatively, the Geotechnical Design Report can state that the sequence of construction is to be decided by the contractor.

4.5 Monitoring

Some readers have understood 4.5(1) to mean that all constructions conforming to EC7 have to be monitored, presumably by measuring displacements and other parameters. However, the intention of this paragraph is really only to note what monitoring should be expected to achieve, on those occasions when it is appropriate to use it.

The text of EC7 concentrates on monitoring that involves measurement. However, it is emphasised here that the vigilance and common sense observations of site staff may often be equally valuable, especially in noticing the development of an adverse situation.

(1)P Monitoring shall be applied:

— to check the validity of predictions of performance made during the design;
— to ensure that the structure will continue to perform as required after completion.

(2)P The monitoring programme shall be carried out in accordance with the Geotechnical Design Report (see 2.8(3)).

(3) Records of the actual performance of structures should be made in order to collect databases of comparable experience.

(4) Monitoring should include measurement of the following:

— deformations of the ground affected by the structure;
— values of actions;
— values of contact pressure between ground and structure;
— pore-water pressures;
— forces and displacements (vertical or horizontal movements, rotations or distortions) in structural members.

(5) Results of measurements should be integrated with qualitative observations including architectural appearance.

(6) The length of any post-construction monitoring period should be altered as a result of observations made during construction. For structures that may impact unfavourably on appreciable parts of the surrounding physical environment, or for which failure may involve abnormal risks to property or life, monitoring should be required for more than ten years after construction is complete, or throughout the life of the structure.

(7)P The results obtained from monitoring shall always be evaluated and interpreted and this shall normally be done in a quantitative manner.

(8) For Geotechnical Category 1, the evaluation of performance may be simple, qualitative and based on inspection.

(9) For Geotechnical Category 2, the evaluation of performance may be based on measurements of movements of selected points on the structure.

(10) For Geotechnical Category 3, the evaluation of performance should normally be based on the measurement of displacements and analyses, which take account of the sequence of construction operations.

(11)P For structures that may have an adverse effect on ground or ground-water conditions, the possibility of leakage or of alterations to the pattern of ground-water flow, especially when fine grained soils are involved, shall be taken into account when planning the monitoring programme.

(12) Examples of this type of structure are:

— water retaining structures;
— structures intended to control seepage;

— tunnels;
— large underground structures;
— deep basements;
— slopes and earth retaining structures;
— ground improvements.

4.6 Maintenance

(1)P The maintenance required to ensure the safety and serviceability of the structure shall be specified.

NOTE Normally this is specified to the owner/client.

(2) The maintenance specifications should provide information on:

— critical parts of the structure, which require regular inspection;
— works prohibited without a design review of the structure prior to their execution;
— frequency of the inspection.

Section 5 Fill, dewatering, ground improvement and reinforcement

Section 5 gives very basic provisions for 'unnatural ground', i.e. situations in which fill is placed or ground is improved by geotechnical processes.

Although the provisions are sound, they do not provide enough information to define the design process. More detailed and generally more helpful information can be found in the following references:

BS 6031, *Code of practice for earthworks* (revised and republished in 2009)

BS 8006-1, *Soil reinforcement* (currently being revised for republication)

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Section 6 Spread foundations

Section 6 is the first of a series of sections dealing with the major elements of construction that require geotechnical design. Each of these has the same basic list of clauses, with further explanation shown here in *italics*:

- General *(scope)*;
- Limit states *(that could occur and so should be considered)*;
- Actions and design situations *(that the structure should be able to accommodate)*;
- Design and construction considerations *(including buildability)*;
- Ultimate limit state design;
- Serviceability limit state design;
- Structural design;
- Other particular clauses are added at the end or earlier.

Because it deals with a very common form of foundation, most of the text of Section 6 is included here.

6.1 General

(1)P The provisions of this Section apply to spread foundations including pads, strips and rafts.

The term *spread foundations* is preferred to *shallow foundations* because there is no restriction on the depth below ground level at which these foundations may operate. The essential point of their definition is that they transfer load to the ground as a direct stress spread over an adequate area. Thus spread foundations could be several metres 'deep' or take the form of pad foundations in the bottom of a basement many metres below the original ground surface.

(2) Some of the provisions may be applied to deep foundations such as caissons.

6.2 Limit states

(1)P The following limit states shall be considered and an appropriate list shall be compiled:

— loss of overall stability; [1]
— bearing resistance failure, punching failure, squeezing; [2]
— failure by sliding; [3]
— combined failure in the ground and in the structure; [4]
— structural failure due to foundation movement; [5]
— excessive settlements;

— excessive heave due to swelling, frost and other causes;
— unacceptable vibrations.

[1] See Figure X2a.

[2] See Figure X2b.

[3] See Figure X2c.

[4] An example of combined failure in the ground and in the structure could be the structural failure in bending of a pad or raft foundation because weak, failing ground in the bearing stratum requires the bearing pressures to be spread further, increasing bending moments. See Figure X2d.

[5] Examples of structural ‘failure’ could be cracking due to foundation settlement (SLS) or collapse of the structure (ULS) due to foundation displacement, even though the ground has not reached an ultimate failure. Further information can be found in 6.5.5.

6.3 Actions and design situations

(1)P Design situations shall be selected in accordance with 2.2.

(2) The actions listed in 2.4.2(4) should be considered when selecting the limit states for calculation.

(3) If structural stiffness is significant, an analysis of the interaction between the structure and the ground should be performed in order to determine the distribution of actions.

It can be debated whether stresses found in an interaction analysis should be called *actions* or *action effects*. See comments on 2.4.2(1).

6.4 Design and construction considerations

Paragraphs (1) and (2) constitute checklists of items relevant to design and construction considerations. These are provided in a ready-to-use form in Appendix 2.

(1)P When choosing the depth of a spread foundation the following shall be considered:

— reaching an adequate bearing stratum;
— the depth above which shrinkage and swelling of clay soils, due to seasonal weather changes, or to trees and shrubs, may cause appreciable movements;
— the depth above which frost damage may occur;
— the level of the water table in the ground and the problems, which may occur if excavation for the foundation is required below this level;

— possible ground movements and reductions in the strength of the bearing stratum by seepage or climatic effects or by construction procedures;
— the effects of excavations on nearby foundations and structures;
— anticipated excavations for services close to the foundation;
— high or low temperatures transmitted from the building;
— the possibility of scour;
— the effects of variation of water content due to long periods of drought, and subsequent periods of rain, on the properties of volume-unstable soils in arid climatic areas;
— the presence of soluble materials, e.g. limestone, claystone, gypsum, salt rocks;

(2) Frost damage will not occur if:

— the soil is not frost-susceptible;
— the foundation level is beneath frost-free depth;
— frost is eliminated by insulation.

(3) EN-ISO 13793 may be applied for frost protecting measures for building foundations.

EN ISO 13793 provides helpful generic information and worked examples. To avoid problems from frost, UK Building Regulations (Clause 2E4) require a minimum depth of foundations below ground level of 450 mm. The same figure is also given by BS 8002 and BS 8103-1:1995.

(4)P In addition to fulfilling the performance requirements, the design foundation width shall take account of practical considerations such as economic excavation, setting out tolerances, working space requirements and the dimensions of the wall or column supported by the foundation.

Paragraph (4) is about practical issues of buildability. Paragraph (5) considers alternative approaches to the design calculations.

(5)P One of the following design methods shall be used for spread foundations:

— a direct method, in which separate analyses are carried out for each limit state. When checking against an ultimate limit state, the calculation shall model as closely as possible the failure mechanism, which is envisaged. When checking against a serviceability limit state, a settlement calculation shall be used;

The method above is fairly easily defined: bearing capacity calculations will be needed for ULS and settlement calculations for SLS. Commonly, the dimensions of a spread foundation will be governed by DA1-C2 or SLS.

The two methods described below may be essentially the same.

— an indirect method using comparable experience and the results of field or laboratory measurements or observations, and chosen in relation to serviceability limit state loads so as to satisfy the requirements of all relevant limit states;
— a prescriptive method in which a presumed bearing resistance is used (see 2.5).

(6) Calculation models for ultimate and serviceability limit state design of spread foundations on soil given in 6.5 and 6.6 respectively should be applied. Presumed bearing pressures for the design of spread foundations on rock should be applied according to 6.7.

6.5 Ultimate limit state design

6.5.1 Overall stability

Whatever is to be built, it is necessary to check that the site itself is stable, both with and without the proposed additional loading. On sloping ground, this may include considering the stability of the slope well beyond the boundaries of the site itself, both up and down slope.

(1)P Overall stability, with or without the foundations, shall be checked particularly in the following situations:

— near or on a natural or man-made slope;
— near an excavation or a retaining wall;
— near a river, a canal, a lake, a reservoir or the sea shore;
— near mine workings or buried structures.

(2)P For such situations, it shall be demonstrated using the principles described in Section 11, that a stability failure of the ground mass containing the foundation is sufficiently improbable.

6.5.2 Bearing resistance

6.5.2.1 General

(1)P The following inequality shall be satisfied for all ultimate limit states:

$$V_d \leq R_d \tag{6.1}$$

This requirement, reflecting requirement 2.5, simply means that the *design* ultimate bearing capacity, R_d, (expressed in a vertical direction) shall be no less than the *design* vertical load on the foundation, V_d. The design resistance will be affected by the inclination and eccentricity of the load, as well as by the soil properties.

(2)P R_d shall be calculated according to 2.4.

(3)P V_d shall include the weight of the foundation, the weight of any backfill material and all earth pressures, either favourable or unfavourable. Water pressures not caused by the foundation load shall be included as actions.

The comparison between V_d and R_d is made at the interface between the structure and the ground, at the underside of the foundation. So V_d includes all vertical forces coming from above the interface.

In undrained clay, the water pressure may change as a result of loading. This is regarded as a reaction. Existing water pressures, not caused by the loading, are classed as actions and, in some design approaches, may be subject to factoring. For a discussion of the treatment of water pressures in EC7 and, in particular, the approach taken to this in the UKNA, see 2.4.6.1.

6.5.2.2 Analytical method

(1) A commonly recognized analytical method should be used.

NOTE The sample analytical calculation for bearing resistance given in Annex D may be used.

Annex D provides values for bearing capacity factors, considering also inclination and eccentricity of the loads, shape of foundation and inclination of the structure/ground interface. For spread foundations at significant depth, compared to their width, depth correction factors are needed but they are not provided in Annex D because a common formulation has not been agreed across Europe. Reference to Chen and McCarron (1991) or Tomlinson and Boorman (2001) is recommended for these.

Various values for the bearing capacity factors can be found in the literature. The formulation provided in Annex D is not the most conservative, so it would be unwise to adopt values more optimistic than those of Annex D.

(2)P An analytical evaluation of the short-term and long-term values of R_d shall be considered, particularly in fine-grained soils.

(3)P Where the soil or rock mass beneath a foundation presents a definite structural pattern of layering or other discontinuities, the assumed rupture mechanism and the selected shear strength and deformation parameters shall take into account the structural characteristics of the ground.

Calculation of bearing capacity in jointed rock is difficult and specialist advice is needed.

(4)P When calculating the design bearing resistance of a foundation on layered deposits, the properties of which vary greatly between one another, the design values of the ground parameters shall be determined for each layer.

(5) Where a strong formation underlies a weak formation, the bearing resistance may be calculated using the shear strength parameters of the weak formation. For the reverse situation, punching failure should be checked.

A slightly less conservative approach to this problem, developed by Vesic, is presented by Chen and McCarron (1991).

(6) Analytical methods are often not applicable to the design situations described in 6.5.2.2(3)P, 6.5.2.2(4)P and 6.5.2.2(5). Numerical procedures should then be applied to determine the most unfavourable failure mechanism.

(7) The overall stability calculations described in Section 11 may be applied.

In general, use of slope stability methods for calculations of bearing capacity is not recommended because it can give significantly unsafe results. If well performed, finite element or finite difference computations are preferable.

6.5.2.3 Semi-empirical method

(1) A commonly recognized semi-empirical method should be used.

NOTE The sample semi-empirical method for bearing resistance estimation using pressuremeter test results given in Annex E is recommended.

It should be remembered that this text applies to the derivation of bearing resistance for ULS calculations. Other available empirical calculations for spread foundations are generally concerned with settlement rather than ultimate bearing capacity.

6.5.2.4 Prescriptive method using presumed bearing resistance

(1) A commonly recognized prescriptive method based on presumed bearing resistance should be used.

It is questionable whether this approach, and the reference to Annex G, belong under the heading of ULS. The charts in Annex G were taken from BS 8004, where they were intended for use with unfactored loads, giving settlement less than 0.5% of the width of the foundation. They could be used, quite reasonably, with ULS, DA1-C2, since the factors on actions in this combination, in the UKNA, are close to unity.

NOTE The sample method for deriving the presumed bearing resistance for spread foundations on rock given in Annex G is recommended. When such a method is applied, the design result should be evaluated on the basis of comparable experience.

6.5.3 Sliding resistance

(1)P Where the loading is not normal to the foundation base, foundations shall be checked against failure by sliding on the base.

(2)P The following inequality shall be satisfied:

$$H_d \leq R_d + R_{p;d} \tag{6.2}$$

In this requirement, H_d is the design horizontal force potentially causing sliding, R_d is the design base resistance to sliding and $R_{p;d}$ is the design value of any passive resistance that is available. As these are all *design* values for ULS, partial factors are incorporated in deriving them.

(3)P H_d shall include the design values of any active earth forces imposed on the foundation.

(4)P R_d shall be calculated according to 2.4.

(5) The values of R_d and $R_{p;d}$ should be related to the scale of movement anticipated under the limit state of loading considered. For large movements, the possible relevance of post-peak behaviour should be considered. The value of $R_{p;d}$ selected should reflect the anticipated life of the structure.

The final sentence of (5) relates to the paragraphs that follow.

For most purposes, $R_{p;d}$ can be calculated using the peak strength of the soil, with a characteristic value (cautiously selected, by definition) reduced by partial factors as specified in the UKNA. Dense granular soils may exhibit very high angles of shearing resistance which are rarely used in practice because of a fear of progressive failure in a brittle material; this is a sound approach. The paragraphs that follow have more about R_d.

(6)P For foundations bearing within the zone of seasonal movements of clay soils, the possibility that the clay could shrink away from the vertical faces of foundations shall be considered.

(7)P The possibility that the soil in front of the foundation may be removed by erosion or human activity shall be considered.

(8)P For drained conditions, the design shear resistance, R_d, shall be calculated either by factoring the ground properties or the ground resistance as follows;

$$R_d = V'_d \tan \delta_d \tag{6.3a}$$

A variant of this equation, (6.3b), has been removed here because it is thought to cause confusion when the UKNA is being used. The value of V'_d will incorporate the partial factors for beneficial actions (usually 1.0 for permanent actions and 0.0 for variable actions).

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(9)P In determining V'_d, account shall be taken of whether H_d and V'_d are dependent or independent actions.

(10) The design friction angle δ_d may be assumed equal to the design value of the effective critical state angle of shearing resistance, $\varphi'_{cv;d}$, for cast-in-situ concrete foundations and equal to 2/3 $\varphi'_{cv;d}$ for smooth precast foundations. Any effective cohesion c' should be neglected.

(11)P For undrained conditions, the design shearing resistance, R_d, shall be calculated either by factoring the ground properties or the ground resistance as follows:

$$R_d = A'c_{u;d} \tag{6.4a}$$

or

$$R_d = (A'c_{u;k}) / \gamma_{R;h} \tag{6.4b}$$

In Design Approach 1, adopted by the UKNA, only 6.4a is used.

(12)P If it is possible for water or air to reach the interface between a foundation and an undrained clay subgrade, the following check shall be made:

$$R_d \leq 0,4\ V_d \tag{6.5}$$

Paragraphs (12) and (13) apply in the special case where the vertical pressure from the foundation is fairly small relative to the strength of the soil or rock. Here, the possibility of the foundation sliding over the asperities of the surface should be considered (Mortensen, 1983).

(13) Requirement (6.5) may only be disregarded if the formation of a gap between the foundation and the ground will be prevented by suction in areas where there is no positive bearing pressure.

6.5.4 Loads with large eccentricities

(1)P Special precautions shall be taken where the eccentricity of loading exceeds 1/3 of the width of a rectangular footing or 0,6 of the radius of a circular footing.

Such precautions include:

- careful review of the design values of actions in accordance with 2.4.2;
- designing the location of the foundation edge by taking into account the magnitude of construction tolerances.

(2) Unless special care is taken during the works, tolerances up to 0,10 m should be considered.

There is no ‘middle third’ requirement in EC7. Special care is called for if the ULS design resultant forces pass outside the middle two-thirds of a rectangular footing. The ‘tolerance’ required by paragraph (2) means, in effect, that the footing should be designed to be adequate even if it is built 0.1 m short at the critical location; that is, a construction tolerance of 0.1 m should be added to the design width calculated for bearing capacity when the design loading is highly eccentric. The comments on 6.8(2) regarding structural design are also relevant.

6.5.5 Structural failure due to foundation movement

(1)P Differential vertical and horizontal foundation displacements shall be considered to ensure that they do not lead to an ultimate limit state occurring in the supported structure.

In some circumstances, settlement [paragraph (1)] or heave [paragraph (3)] can lead to sufficient deformation in a supported building that collapse occurs. This could relate, for example, to loss of bearing area on a supporting wall for a floor slab or roof. Paragraph (2) is similar to 6.5.2.4 and seems to be out of place here.

(2) A presumed bearing pressure may be adopted (see 2.5) provided displacements will not cause an ultimate limit state in the structure.

(3)P In ground that may swell, the potential differential heave shall be assessed and the foundations and structure designed to resist or accommodate it.

6.6 Serviceability limit state design

6.6.1 General

(1)P Account shall be taken of displacements caused by actions on the foundation, such as those listed in 2.4.2(4).

(2)P In assessing the magnitude of foundation displacements, account shall be taken of comparable experience, as defined in 1.5.2.2. If necessary, calculations of displacements shall also be carried out.

EC7 intends not to encourage misleading or unnecessary calculation. So, in paragraph (2), assessment of foundation displacement (settlement, heave or horizontal displacement) may be based on ‘comparable experience’ (see 1.5.2.2) and calculations are only used ‘if necessary’, which could be taken to mean if comparable experience does not lead to a confident view. This is taken up in 6.6.2(16), specifically for settlement of foundations on clays. Whichever approach is used, precise prediction is not necessary and usually not possible *N* paragraph (6). What is required is to check that estimated displacements will not exceed tolerable limits (see 2.4.8 and 2.4.9).

For foundations on soft clay (fairly unusual), paragraph (3) (a Principle) makes displacement calculations mandatory. For foundations on stiff and firm clays, paragraph (4) advises that calculations 'should usually be undertaken', though this should be read with paragraph (2), which makes calculations unnecessary where there is clear, comparable experience.

Unfactored values of actions are used for serviceability calculations. Paragraphs (7) and (9) state that total and differential settlements, and relative rotations of foundations, shall all be considered. More information on settlement calculations is provided in 6.6.2.

(3)P For soft clays, settlement calculations shall always be carried out.

It is advisable to calculate or consider settlements very carefully if the undrained strength of the clay is less than about 50 kPa or if the new loading exceeds the clay's preconsolidation pressure.

(4) For spread foundations on stiff and firm clays in Geotechnical Categories 2 and 3, calculations of vertical displacement (settlement) should usually be undertaken. Methods that may be used to calculate settlements caused by loads on the foundation are given in 6.6.2.

(5)P The serviceability limit state design loads shall be used when calculating foundation displacements for comparison with serviceability criteria.

(6) Calculations of settlements should not be regarded as accurate. They merely provide an approximate indication.

(7)P Foundation displacements shall be considered both in terms of displacement of the entire foundation and differential displacements of parts of the foundation.

(8)P The effect of neighbouring foundations and fills shall be taken into account when calculating the stress increase in the ground and its influence on ground compressibility.

(9)P The possible range of relative rotations of the foundation shall be assessed and compared with the relevant limiting values for movements discussed in 2.4.9.

6.6.2 Settlement

(1)P Calculations of settlements shall include both immediate and delayed settlement.

Application rules in this section describe the basic principles of conventional calculation of settlement, as found in many textbooks, without giving details of calculations. They are therefore omitted here. Part 2 of Eurocode 7 (BS EN 1997-2) contains in its annexes several methods of calculating settlement on the basis of in situ tests.

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(8)P Any possible additional settlement caused by self-weight compaction of the soil shall be assessed.

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(10)P Either linear or non-linear models of the ground stiffness shall be adopted, as appropriate.

Non-linear models of calculation include oedometric calculations in clays (e.g. Burland et al, 1977; Tomlinson and Boorman, 2001), empirical relationships such as Burland and Burbidge (1985) and use of more complex models involving stiffness degradation with strain or shear failure, typically in finite element programs.

(11)P To ensure the avoidance of a serviceability limit state, assessment of differential settlements and relative rotations shall take account of both the distribution of loads and the possible variability of the ground.

(12) Differential settlement calculations that ignore the stiffness of the structure tend to be over-predictions. An analysis of ground-structure interaction may be used to justify reduced values of differential settlements.

(13) Allowance should be made for differential settlement caused by variability of the ground unless it is prevented by the stiffness of the structure.

(14) For spread foundations on natural ground, it should be taken into account that some differential settlement normally occurs even if the calculation predicts uniform settlement only.

(15) The tilting of an eccentrically loaded foundation should be estimated by assuming a linear bearing pressure distribution and then calculating the settlement at the corner points of the foundation, using the vertical stress distribution in the ground beneath each corner point and the settlement calculation methods described above.

Paragraph (15) offers a method of calculating the tilt of a rigid foundation, implicitly assuming the adoption of a linear elastic model of the ground. When the stiffness of the ground is relatively constant over the depth concerned beneath the foundation, an alternative approach is offered by the formulae for rotation of rigid plates on the surfaces of half spaces. These can be found in Lambe and Whitman (1979) or Poulos and Davis (1974), for example.

(16) For conventional structures founded on clays, the ratio of the bearing capacity of the ground, at its initial undrained shear strength, to the applied serviceability loading should be calculated (see 2.4.8(4)). If this ratio is less than 3, calculations of settlements should always be undertaken. If the ratio is less than 2, the calculations should take account of non-linear stiffness effects in the ground.

Paragraph (16) allows the use of a conventional 'overall factor of safety' approach as a means of limiting settlements of spread foundations on clays, avoiding the need for calculation. In effect, it is an application of the 'comparable experience' approach of 6.6.1(2), restricting strain by mobilising only a small component of the soil's strength; Osman et al (2004) discuss this approach. It should be read with 6.6.1(3), which requires calculations of settlements for foundations on soft clays. If overall factors of safety are to be reduced, calculations become necessary, being more sophisticated if relatively low overall factors are to be used.

6.6.3 Heave

(1)P The following causes of heave shall be distinguished:

— reduction of effective stress;
— volume expansion of partly saturated soil;
— heave due to constant volume conditions in fully saturated soil, caused by settlement of an adjacent structure.

(2)P Calculations of heave shall include both immediate and delayed heave.

6.6.4 Vibration analysis

(1) P Foundations for structures subjected to vibrations or to vibrating loads shall be designed to ensure that vibrations will not cause excessive settlements.

Advice on design for vibrations can be found in CP 2012-1:1974 and ISO 4866:2010 and in many textbooks such as O'Reilly and Brown (1991) or Arya et al (1979).

(2) Precautions should be taken to ensure that resonance will not occur between the frequency of the dynamic load and a critical frequency in the foundation-ground system, and to ensure that liquefaction will not occur in the ground.

(3)P Vibrations caused by earthquakes shall be considered using EN 1998.

6.7 Foundations on rock; additional design considerations

(1)P The design of spread foundations on rock shall take account of the following features:

— the deformability and strength of the rock mass and the permissible settlement of the supported structure;
— the presence of any weak layers, for example solution features or fault zones, beneath the foundation;
— the presence of bedding joints and other discontinuities and their characteristics (for example filling, continuity, width, spacing);

— the state of weathering, decomposition and fracturing of the rock;
— disturbance of the natural state of the rock caused by construction activities, such as, for example, underground works or slope excavation, being near to the foundation.

(2) Spread foundations on rock may normally be designed using the method of presumed bearing pressures. For strong intact igneous rocks, gneissic rocks, limestones and sandstones, the presumed bearing pressure is limited by the compressive strength of the concrete foundation.

NOTE The recommended method for deriving presumed bearing resistances for spread foundations on rock is given in Annex G.

Annex G presents not so much a method as a series of charts derived from BS 8004.

(3) The settlement of a foundation may be assessed on the basis of comparable experience related to rock mass classification.

6.8 Structural design of spread foundations

(1)P Structural failure of a spread foundation shall be prevented in accordance with 2.4.6.4.

The comments under 2.4.7.3.4.2 consider the application of ULS Design Approach 1 in structural design. Commonly, the dimensions of a spread foundation may be governed by DA1-C2 or SLS. However, once the dimensions have been fixed, the structural strength of the foundations should be checked using factored structural loads, which are consistent with DA1-C1.

(2) The bearing pressure beneath a stiff foundation may be assumed to be distributed linearly. A more detailed analysis of soil-structure interaction may be used to justify a more economic design.

A linear distribution of bearing pressure is allowed for the purpose of calculating structural stresses due to ULS design actions; examples are shown in Figure X3. This should not be confused with the more plastic approach taken for verification of the ULS of bearing failure, as shown in Annex D of BS EN 1997-1. It is possible, and allowable, that the resultant of the factored loads could pass outside the middle third of the foundation (e.g. in DA1-C1; see 6.5.4). In this case, it should not be assumed that tension can exist between the footing and the ground, and a distribution as shown in Figure X3(c) should be used.

(3) The distribution of bearing pressure beneath a flexible foundation may be derived by modelling the foundation as a beam or raft resting on a deforming continuum or series of springs, with appropriate stiffness and strength.

The springs or other model should not allow tension.

(4)P The serviceability of strip and raft foundations shall be checked assuming serviceability limit state loading and a distribution of bearing pressure corresponding to the deformation of the foundation and the ground.

(5) For design situations with concentrated loads acting on a strip or raft foundation, forces and bending moments in the foundation may be derived from a subgrade reaction model of the ground, using linear elasticity. The moduli of subgrade reaction may be assessed by a settlement analysis with an appropriate estimate of the bearing pressure distribution. The moduli may be adjusted so that the computed bearing pressures do not exceed values for which linear behaviour may be assumed.

(6) Total and differential settlements of the structure as a whole should be calculated in accordance with 6.6.2. For this purpose, subgrade reaction models are often not appropriate. More precise methods, such as finite element computations, should be used when ground-structure interaction has a dominant effect.

6.9 Preparation of the subsoil

(1)P The subsoil shall be prepared with great care. Roots, obstacles and enclosures of weak soil shall be removed without disturbing the ground. Any resulting holes shall be filled with soil (or other material) to replicate the stiffness of the undisturbed ground.

(2) In soils susceptible to disturbance, such as clay, the sequence of excavation for a spread foundation should be specified to minimise disturbance. Usually it is sufficient to excavate in horizontal slices. In cases where heave is to be controlled, excavation should be in alternate trenches, the concrete being cast in each trench before excavating intermediate ones.

Section 7 Pile foundations

Section 7 considers the design of axially loaded piles in both compression (the most common case) and tension, laterally loaded piles and the effects of ground movements such as those that cause downdrag (negative skin friction). However, in this book only piles in compression will be considered.

Design may be based on calculation or load testing. Although it is acknowledged in 7.4.1(1)P that all pile design requires elements of both of these, they are considered separately in the text and it is not obvious how they are to be combined. For this reason, the UKNA has developed the procedures to some extent, so as to be relevant to the common UK practice of basing design on calculation from ground strength tests, supported by reference to loading tests in similar ground conditions on the same site or in published references. Commonly, the dimensions of piled foundations will be governed by DA1-C2 or SLS; usually the structural design will be governed by DA1-C1. Whatever approach is adopted, an adequate ground investigation is a prerequisite.

Most nations have found it necessary to change the CEN default values for the resistance factors of piles, generally increasing them (see the note on 'recommended values' in 'Values of partial factors' in the Preface). In the UK, studies have been undertaken to compare the sizes of piles designed using EC7 with those designed using more traditional methods. As would be expected, some differences have been noted but, on balance, the different approaches have been found to be comparable (e.g. Bond and Simpson 2009–10). Structural design of piles should conform to EC2, for which some details are still under debate, following reports that results may show greater deviations from past practice in some cases.

7.1 General

(1)P The provisions of this Section apply to end-bearing piles, friction piles, tension piles and transversely loaded piles installed by driving, by jacking, and by screwing or boring with or without grouting.

(2) The provisions of this Section should not be applied directly to the design of piles that are intended as settlement reducers, such as in some piled raft foundations.

Despite 7.1(2), the UKNA is intended to include design of piled rafts. The definition of the characteristic ultimate resistance of piles has been set up so that it could be used with a partial factor of unity for SLS design, following the normal procedure.

(3)P The following standards shall apply to the execution of piles:

— EN 1536:1999, for bored piles
— EN 12063:2000, for sheet pile walls,
— EN 12699:2000, for displacement piles,
— EN 14199:2005, for micropiles.

7.2 Limit states

Paragraph (1) constitutes a checklist of items relevant to design situations, which is provided in a ready-to-use form in Appendix 2.

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7.3 Actions and design situations

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7.3.2 Actions due to ground displacement

This sub-clause deals with design for downdrag (negative skin friction) and other ground movements that might affect piles. These special cases are not included in this book.

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7.4 Design methods and design considerations

7.4.1 Design methods

(1)P The design shall be based on one of the following approaches:

— the results of static load tests, which have been demonstrated, by means of calculations or otherwise, to be consistent with other relevant experience;
— empirical or analytical calculation methods whose validity has been demonstrated by static load tests in comparable situations;
— the results of dynamic load tests whose validity has been demonstrated by static load tests in comparable situations;
— the observed performance of a comparable pile foundation, provided that this approach is supported by the results of site investigation and ground testing.

The essential message of this paragraph is that all pile design has to be based on static load testing, directly or indirectly. However, design based on testing alone is also not allowed: supporting calculations or ‘other relevant experience’ are required, in order to eliminate reliance on unrepresentative test results.

In many cases, a combination of static load tests and calculations will be used or, at least, the designer should be aware of published or other static load testing in ‘comparable situations’ that supports the calculations or dynamic tests on which the design relies. In judging ‘comparable situations’, pile geometry, installation techniques and ground strata should be considered.

It is also permitted to design on the basis of the observed performance of existing piles, provided the comparison is shown to be directly relevant, including the installed geometry of the pile, the actual applied loads and an assessment of the displacements arising from them.

(2) Design values for parameters used in the calculations should be in general accordance with Section 3, but the results of load tests may also be taken into account in selecting parameter values.

Besides being consistent with the general guidance of Section 3, design values should also be in accordance with Section 2 and, for construction in the UK, with the UKNA.

(3) Static load tests may be carried out on trial piles, installed for test purposes only, before the design is finalised, or on working piles, which form part of the foundation.

7.4.2 Design considerations

(1)P The behaviour of individual piles and pile groups and the stiffness and strength of the structure connecting the piles shall be considered.

(2)P In selecting calculation methods and parameter values and in using load test results, the duration and variation in time of the loading shall be considered.

(3)P Planned future placement or removal of overburden or potential changes in the ground-water regime shall be considered, both in calculations and in the interpretation of load test results.

Paragraphs 7.4.2 4(P) and (5) constitute checklists of items to be considered in choosing an appropriate type of pile. These are provided in a ready-to-use form in Appendix 2.

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7.5 Pile load tests

7.5.1 General

(1)P Pile load tests shall be carried out in the following situations:

— when using a type of pile or installation method for which there is no comparable experience;
— when the piles have not been tested under comparable soil and loading conditions;
— when the piles will be subject to loading for which theory and experience do not provide sufficient confidence in the design. The pile testing procedure shall then provide loading similar to the anticipated loading;
— when observations during the process of installation indicate pile behaviour that deviates strongly and unfavourably from the behaviour anticipated on the basis of the site investigation or experience, and when additional ground investigations do not clarify the reasons for this deviation.

7.5.1(1)P lists situations in which it is a code requirement that load testing shall be carried out, though this is qualified slightly by 7.5.1(3). The arrangement of the sub-clause headings implies that the tests need not necessarily be static load tests but the author recommends that this should normally be the case.

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(3) Where load tests are not practical due to difficulties in modelling the variation in the load (e.g. cyclic loading) very cautious design values for the material properties should be used.

Paragraph (3) could be misleading if taken literally, since the designer does not have direct control over *design* values. Like paragraph (4), it should be understood to remind the designer that the *characteristic* value of the pile resistance is intended to be cautious (2.4.5.2(2)), so special care will be needed if there is a lack of relevant load testing. This issue underlies the approach taken in the UKNA to derivation of characteristic resistance by calculation (A.3.3.2 of the UKNA).

(4)P If one pile load test is carried out, it shall normally be located where the most adverse ground conditions are believed to occur. If this is not possible, an allowance shall be made when deriving the characteristic value of the compressive resistance.

(5)P If load tests are carried out on two or more test piles, the test locations shall be representative of the site of the pile foundation and one of the test piles shall be located where the most adverse ground conditions are believed to occur.

The qualification stated for paragraph (4) in cases where it is not possible to test a pile located in the most adverse ground conditions also applies to paragraph (5).

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7.5.2 Static load tests

A ‘static’ load test is one in which the load is applied sufficiently slowly that no dynamic effects are involved. This would not exclude a ‘constant rate of penetration’ (CRP) test in which the pile is pushed slowly, but continuously, into the ground. However, the CRP test is excluded in the UKNA by the requirement of a ‘maintained load’ test.

The sub-clauses below refer to tests on trial piles and working piles. In this book, this terminology is discussed further in the comments on A.3.3.2 of the UKNA.

7.5.2.1 Loading procedure

(1)P The pile load test procedure[5], particularly with respect to the number of loading steps, the duration of these steps and the application of load cycles, shall be such that conclusions can be drawn about the deformation behaviour, creep and rebound of a piled foundation from the measurements on the pile. For trial piles, the loading shall be such that conclusions can also be drawn about the ultimate failure load.

Load testing in the UK should normally be carried out in accordance with the ICE Specification for piling and embedded retaining walls ('SPERWALL', Institution of Civil Engineers, 2007).

Paragraph (1) says that, for trial piles, it shall be possible to draw conclusions about the ultimate failure load, but it does not say that the pile shall be loaded to failure; it appears that a cautious extrapolation of results that do not reach failure could be allowable. In contrast, paragraph (4) says that, for tensile load tests, no extrapolation is allowed. The methods of Chin (1970) or Fleming (1992) could be considered for this extrapolation but it is recommended that the maximum limit of the extrapolation should be a settlement of 10% of the diameter of the pile, consistent with 7.6.1.1(3).

The UKNA allows a reduction in the value of the 'model factor' to be applied to calculations if a maintained load test has been taken to 'the calculated, unfactored ultimate resistance' (UKNA, A.3.3.2). This does not allow extrapolation but does not require the pile to be loaded beyond the value required to verify the design, even if this has not reached failure.

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(4) Pile load tests for the purpose of designing a tensile pile foundation should be carried out to failure. Extrapolation of the load-displacement graph for tension tests should not be used.

7.5.2.2 Trial piles

The UKNA (A.3.3.2) gives values for the model factor and partial resistance factors dependent on the number of pile loading tests undertaken on the site. Nevertheless, the following paragraphs are mandatory principles of EC7 and should therefore be applied.

(1)P The number of trial piles required to verify the design shall depend on the following:

— the ground conditions and their variability across the site;
— the Geotechnical Category of the structure, if appropriate;
— previous documented evidence of the performance of the same type of pile in similar ground conditions;
— the total number and types of pile in the foundation design.

[5] See: ISSMFE Subcommittee on Field and Laboratory Testing, Axial Pile Loading Test, Suggested Method. ASTM Journal, June 1985, pp. 79-90.

(2)P The ground conditions at the test site shall be investigated thoroughly. The depth of borings or field tests shall be sufficient to ascertain the nature of the ground both around and beneath the pile tip. All strata likely to contribute significantly to pile behaviour shall be investigated.

(3)P The method used for the installation of the trial piles shall be fully documented in accordance with 7.9.

7.5.2.3 Working piles

(1)P It shall be specified that the number of working pile load tests shall be selected on the basis of the recorded findings during installation.

(2)P The test load applied to working piles shall be at least equal to the design load for the foundation.

It is unclear what is meant in paragraph (2) by 'design load'. In this book, the comments on A.3.3.2 of the UKNA relate to typical UK practice.

7.5.3 Dynamic load tests

The use of dynamic load testing is allowed provided the method has been carefully calibrated against static load tests.

(1) Dynamic load tests[6] may be used to estimate the compressive resistance provided an adequate site investigation has been carried out and the method has been calibrated against static load tests on the same type of pile, of similar length and cross-section, and in comparable soil conditions, (see 7.6.2.4 to 7.6.2.6).

(2)P If more than one type of dynamic test is used, the results of different types of dynamic test shall always be considered in relation to each other.

(3) Dynamic load tests may also be used as an indicator of the consistency of the piles and to detect weak piles.

7.5.4 Load test report

Sub-clause 7.5.4 constitutes a checklist of items to be included in a load test report. This is provided in a ready-to-use form in Appendix 2. More comprehensive lists are provided by the Institution of Civil Engineers (2007).

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[6] See: ASTM Designation D 4945, Standard Test Method for High-Strain Dynamic Testing of Piles.

7.6 Axially loaded piles

In contrast to the requirements of 7.4.1, sub-clauses 7.6.2.2 and 7.6.2.3 consider separately design based on testing and design based on calculation. Typical UK design uses a combination of these two. For piles in compression, the UKNA accommodates this by starting from 7.6.2.3, 'Ultimate compressive resistance [calculated] from ground test results' and amending the factors required in the light of the amount of testing carried out – see comments on A.3.3.2 of the UKNA.

7.6.1 General

7.6.1.1 Limit state design

(1)P The design shall demonstrate that exceeding the following limit states is sufficiently improbable:

— ultimate limit states of compressive or tensile resistance failure of a single pile;
— ultimate limit states of compressive or tensile resistance failure of the pile foundation as a whole;
— ultimate limit states of collapse or severe damage to a supported structure caused by excessive displacement or differential displacements of the pile foundation;
— serviceability limit states in the supported structure caused by displacement of the piles.

(2) Normally the design should consider the margin of safety with respect to compressive or tensile resistance failure, which is the state in which the pile foundation displaces significantly downwards or upwards with negligible increase or decrease of resistance (see 7.6.2 and 7.6.3).

(3) For piles in compression it is often difficult to define an ultimate limit state from a load settlement plot showing a continuous curvature. In these cases, settlement of the pile top equal to 10% of the pile base diameter should be adopted as the "failure" criterion.

(4)P For piles that undergo significant settlements, ultimate limit states may occur in supported structures before the resistance of the piles is fully mobilised. In these cases a cautious estimate of the possible range of the settlements shall be adopted in design.

NOTE Settlement of piles is considered in 7.6.4

7.6.1.2 Overall stability

(1)P Failure due to loss of overall stability of foundations involving piles in compression shall be considered in accordance with Section 11.

(2) Where there is a possibility of instability, failure surfaces both passing below the piles and intersecting the piles should be considered.

(3)P Failure due to uplift of a block of soil containing piles shall be checked in accordance with 7.6.3.1(4)P.

7.6.2 Compressive ground resistance

7.6.2.1 General

(1)P To demonstrate that the pile foundation will support the design load with adequate safety against compressive failure, the following inequality shall be satisfied for all ultimate limit state load cases and load combinations:

$$F_{c;d} \leq R_{c;d} \tag{7.1}$$

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(3)P For piles in groups, two failure mechanisms shall be taken into account:

— compressive resistance failure of the piles individually;
— compressive resistance failure of the piles and the soil contained between them acting as a block.

The design resistance shall be taken as the lower value caused by these two mechanisms.

The second of the mechanisms in paragraph (3) is particularly relevant for large groups of piles in situations where the strength of the ground beneath the group is less than that of the founding stratum.

(4) The compressive resistance of the pile group acting as a block may be calculated by treating the block as a single pile of large diameter.

(5)P The stiffness and strength of the structure connecting the piles in the group shall be considered when deriving the design resistance of the foundation.

(6) If the piles support a stiff structure, advantage may be taken of the ability of the structure to redistribute load between the piles. A limit state will occur only if a significant number of piles fail together; therefore a failure mode involving only one pile need not be considered.

(7) If the piles support a flexible structure, it should be assumed that the compressive resistance of the weakest pile governs the occurrence of a limit state.

Paragraphs (5) to (7) assume a co-ordinated approach to the design of the structure and the piled foundation. In situations where this is not available, pile designers may have no choice but to design each pile in isolation, without reliance on stiffness in the structure. Paragraph (6) is most readily applied to groups of piles supporting a stiff pile cap but can also apply to other structures.

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7.6.2.2 Ultimate compressive resistance from static load tests

(1)P The manner in which load tests are carried out shall be in accordance with 7.5 and shall be specified in the Geotechnical Design Report.

(2)P Trial piles to be tested in advance shall be installed in the same manner as the piles that will form the foundation and shall be founded in the same stratum.

Paragraphs (3) to (6) consider piles of different sizes and downdrag.

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(8)P For structures, which do not exhibit capacity to transfer loads from "weak" piles to "strong" piles, as a minimum, the following equation shall be satisfied:

$$R_{c;k} = \mathrm{Min}\left\{\frac{(R_{c;m})_{mean}}{\xi_1};\frac{(R_{c;m})_{min}}{\xi_2}\right\} \tag{7.2}$$

where ξ_1 and ξ_2 are correlation factors related to the number of piles tested and are applied to the mean $(R_{c;m})_{mean}$ and the lowest $(R_{c;m})_{min}$ of $R_{c;m}$ respectively.

This paragraph allows derivation of the characteristic resistance of piles direct from results of load tests, without mention of any calculations. It is, however, subject to the general requirement in 7.4.1(1) that requires calculations as a check. In practice, it may be possible to revise initial calculations in the light of test results. The values of ξ_1 and ξ_2 to be used for construction in the UK are provided in the UKNA, Table A.NA.9.

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(9) For structures having sufficient stiffness and strength to transfer loads from "weak" to "strong" piles, the values of ξ_1 and ξ_2 may be divided by 1,1, provided that ξ_1 is never less than 1,0.

(10)P The systematic and random components of the variations in the ground shall be recognised in the interpretation of pile load tests.

(11)P The records of the installation of the test pile(s) shall be checked and any deviation from the normal execution conditions shall be accounted for.

(12) The characteristic compressive resistance of the ground, $R_{c;k}$, may be derived from the characteristic values of the base resistance, $R_{b;k}$, and of the shaft resistance, $R_{s;k}$, such that:

$$R_{c;k} = R_{b;k} + R_{s;k} \tag{7.3}$$

(13) These components may be derived directly from static load test results, or estimated on the basis of ground test results or dynamic load tests.

(14)P The design resistance, $R_{c;d}$, shall be derived from either:

$$R_{c;d} = R_{c;k}/\gamma_t \tag{7.4}$$

or

$$R_{c;d} = R_{b;k}/\gamma_b + R_{s;k}/\gamma_s \tag{7.5}$$

**

The values of γ_b and γ_s to be used for construction in the UK are provided in the UKNA, Tables A.NA.6 to A.NA.8.

7.6.2.3 Ultimate compressive resistance from ground test results

(1)P Methods for assessing the compressive resistance of a pile foundation from ground test results shall have been established from pile load tests and from comparable experience as defined in 1.5.2.*2.*

(2) A model factor may be introduced as described in 2.4.1(9) to ensure that the predicted compressive resistance is sufficiently safe.

(3)P The design compressive resistance of a pile, $R_{c;d}$, shall be derived from:

$$R_{c;d} = R_{b;d} + R_{s;d} \tag{7.6}$$

(4)P For each pile, $R_{b;d}$ and $R_{s;d}$ shall be obtained from:

$$R_{b;d} = R_{b;k}/\gamma_b \text{ and } R_{s;d} = R_{s;k}/\gamma_s \tag{7.7}$$

The values of γ_b and γ_s to be used for construction in the UK are provided in the UKNA, Tables A.NA.6 to A.NA.8.

Paragraphs (5) to (7) of this sub-clause explain how piles may be designed from ground test results using a similar procedure to that for design from load tests, as given in 7.6.2.2. This approach is mainly relevant to tests such as cone penetration tests that give a semi-continuous profile at each tested location and it is not expected to be used much in the UK. The UKNA therefore concentrates on the 'alternative' approach given in paragraph (8).

**

(8) The characteristic values may be obtained by calculating:

$$R_{b;k} = A_b\, q_{b;k} \text{ and } R_{s;k} = \sum_i A_{s;i} \cdot q_{s;i;k} \tag{7.9}$$

where $q_{b;k}$ and $q_{s;i;k}$ are characteristic values of base resistance and shaft friction in the various strata, obtained from values of ground parameters.

NOTE If this alternative procedure is applied, the values of the partial factors γ_b and γ_s recommended in Annex A may need to be corrected by a model factor larger than 1,0. The value of the model factor may be set by the National annex.

Much of the pile design carried out in the UK follows the procedure of paragraph (8). This is developed in more detail in the UKNA and further explanation, with examples, can be found in Bond and Simpson (2009–10). It is a weakness of paragraph (8) that the means by which the characteristic base and shaft resistances are to be established from ground test results is not stated. This issue is considered more carefully in the UKNA, sub-clause A.3.3.2. The reader is referred to that sub-clause and the notes on it in this book.

**

7.6.2.4 Ultimate compressive resistance from dynamic impact tests

Use of dynamic impact tests is a specialist procedure beyond the scope of this book.

**

7.6.2.5 Ultimate compressive resistance by applying pile driving formulae

Use of pile driving formulae is a specialist procedure beyond the scope of this book.

**

7.6.2.6 Ultimate compressive resistance from wave equation analysis

Use of wave equation analysis is a specialist procedure beyond the scope of this book.

**

7.6.2.7 Re-driving

(1)P In the design, the number of piles to be re-driven shall be specified. If re-driving gives lower results, these shall be used as the basis for ultimate compressive resistance assessment. If re-driving gives higher results, these may be considered.

(2) Re-driving should usually be carried out in silty soils, unless local comparable experience has shown it to be unnecessary.

Clearly, 7.6.2.7 only applies to driven piles. Re-driving is also necessary for any base controlled piles that have been lifted by the subsequent driving of other piles.

**

7.6.3 Ground tensile resistance

Tension piles are not included in this book.

**

7.6.4 Vertical displacements of pile foundations (Serviceability of supported structure)

7.6.4.1 General

(1)P Vertical displacements under serviceability limit state conditions shall be assessed and checked against the requirements given in 2.4.8 and 2.4.9.

(2) When calculating the vertical displacements of a pile foundation, the uncertainties involved in the calculation model and in determining the relevant ground properties should be taken into account. Hence it should not be overlooked that in most cases calculations will provide only an approximate estimate of the displacements of the pile foundation.

NOTE For piles bearing in medium-to-dense soils and for tension piles, the safety requirements for the ultimate limit state design are normally sufficient to prevent a serviceability limit state in the supported structure.

The note above is very significant. Whereas the Eurocodes generally aim to consider ultimate and serviceability limit states separately, this note states that for pile design the partial factors used in the ULS calculation 'are normally sufficient to prevent a serviceability limit state'. This requirement has been influential in the choice of partial factors for the UKNA. Nevertheless, for each particular application, the designer with overall responsibility for the design of the structure should consider carefully whether the proposed piles will provide adequately stiff foundations.

7.6.4.2 Pile foundations in compression

(1)P The occurrence of a serviceability limit state in the supported structure due to pile settlements shall be checked, taking into account downdrag, where probable.

NOTE When the pile toe is placed in a medium-dense or firm layer overlying rock or very hard soil, the partial factors for ultimate limit state conditions are normally sufficient to satisfy serviceability limit state conditions.

For the note above, see the comment on the note of 7.6.4.1(2). Paragraphs (2) to (4) make it clear that an assessment of the likely settlement of the piles under load should be made. This need not be precise in cases where it is readily established that it will be small enough to be no problem.

(2)P Assessment of settlements shall include both the settlement of individual piles and the settlement due to group action.

(3) The settlement analysis should include an estimate of the differential settlements that may occur.

(4) When no load test results are available for an analysis of the interaction of the piled foundation with the superstructure, the load-settlement performance of individual piles should be assessed on the basis of empirically established safe assumptions.

7.6.4.3 Pile foundations in tension

Tension piles are not included in this book.

**

7.7 Transversely loaded piles

Transversely loaded piles are not included in this book.

**

7.8 Structural design of piles

The note on structural design in the Preface to this book is relevant here. Often the structural design of piles will be governed by DA1-C1.

(1)P Piles shall be verified against structural failure in accordance with 2.4.6.4.

(2)P The structure of piles shall be designed to accommodate all the situations to which the piles will be subjected. These include:

- the circumstances of their use e.g. corrosion conditions;
- the circumstances of their installation e.g. adverse ground conditions such as boulders, steeply inclined bedrock surfaces;
- other factors influencing driveability, including quality of joints;
- for precast piles, the circumstances of their transportation to site and installation.

(3)P During structural design, construction tolerances as specified for the type of pile, the action components and the performance of the foundation shall be taken into account.

(4)P Slender piles passing through water or thick deposits of extremely low strength fine soil shall be checked against buckling.

(5) Normally a check for buckling is not required when the piles are contained by soils with a characteristic shear strength, c_u, that exceeds 10 kPa.

7.9 Supervision of construction

(1)P A pile installation plan shall form the basis for the piling works.

Paragraph (2) constitutes a checklist of items that should be included in the installation plan. This is provided in a ready-to-use form in Appendix 2.

**

(3)P It shall be specified that the installation of all piles is monitored and records are made as the piles are installed.

(4) The record for each pile should include aspects of construction covered in the relevant execution standards, EN 1536:1999, EN 12063:1999, EN 12699:2000, EN 14199:2005, such as the following:

The remainder of paragraph (4) constitutes a checklist of items that should be included in pile construction records. This is provided in a ready-to-use form in Appendix 2.

**

(5) Records should be kept for at least a period of five years after completion of the works. As-built records should be compiled after completion of the piling and kept with the construction documents.

(6)P If site observations or inspection of records reveal uncertainties about the quality of installed piles, investigations shall be carried out to determine their condition and if remedial measures are necessary. These investigations shall include either performing a static pile load or integrity test, installing a new pile or, in the case of a displacement pile, re-driving the pile, in combination with ground tests adjoining the suspect pile.

(7)P Tests shall be used to examine the integrity of piles for which the quality is sensitive to the installation procedures if the procedures cannot be monitored in a reliable way.

(8) Dynamic low strain integrity tests may be used for a global evaluation of piles that might have severe defects or that may have caused a serious loss of strength in the soil during construction. Defects such as insufficient quality of concrete and thickness of concrete cover, both of which can affect the long term performance of a pile, often cannot be found by dynamic tests and other tests, such as sonic tests, vibration tests or coring, may be needed in supervising the execution.

Section 8 Anchorages

Design of anchorages is a specialist activity outside the scope of this book. Section 8 is currently being revised. BS 8081, *Code of practice for ground anchorages*, has been retained for the time being as a current British Standard.

**

Section 9 Retaining structures

The UKNA to BS EN 1997-1 notes two documents as NCCI relevant to retaining structures, to which reference is recommended:

PD 6694-1: *Recommendations for the design of structures subject to traffic loading to BS EN 1997-1*.

and

CIRIA Report C580: Gaba et al (2003).

9.1 General

9.1.1 Scope

(1)P The provisions of this Section shall apply to structures, which retain ground comprising soil, rock or backfill and water. Material is retained if it is kept at a slope steeper than it would eventually adopt if no structure were present. Retaining structures include all types of wall and support systems in which structural elements have forces imposed by the retained material.

(2)P Pressure from granular material stored in silos shall be calculated using EN 1991-4.

9.1.2 defines gravity walls, embedded walls and 'composite' walls composed of elements from both gravity and embedded walls.

**

9.2 Limit states

(1)P A list shall be compiled of limit states to be considered. As a minimum the following limit states shall be considered for all types of retaining structure:

- loss of overall stability;
- failure of a structural element such as a wall, anchorage, wale or strut or failure of the connection between such elements;
- combined failure in the ground and in the structural element;
- failure by hydraulic heave and piping;
- movement of the retaining structure, which may cause collapse or affect the appearance or efficient use of the structure or nearby structures or services, which rely on it;
- unacceptable leakage through or beneath the wall;
- unacceptable transport of soil particles through or beneath the wall;
- unacceptable change in the ground-water regime.

(2)P In addition, the following limit states shall be considered for gravity walls and for composite retaining structures:

— bearing resistance failure of the soil below the base;
— failure by sliding at the base;
— failure by toppling;

and for embedded walls:

— failure by rotation or translation of the wall or parts thereof;
— failure by lack of vertical equilibrium.

(3)P For all types of retaining structure, combinations of the above mentioned limit states shall be taken into account, if relevant.

(4) Design of gravity walls often requires solution of the same types of problem encountered in the design of spread foundations and embankments and slopes. When considering the limit states, the principles of Section 6 should therefore be applied, as appropriate. Special care should be taken to account for bearing resistance failure of the ground below the base of the wall under loads with large eccentricities and inclinations (see 6.5.4).

9.3 Actions, geometrical data and design situations

9.3.1 Actions

9.3.1.1 Basic actions

(1) The actions listed in 2.4.2(4) should be considered.

The text omitted here provides comments on:

- weight of backfill material;
- surcharges;
- weight of water;
- wave and ice forces;
- seepage forces;
- collision forces;
- temperature effects.

More detailed information on design for traffic loading can be found in PD 6694-1.

**

9.3.2 Geometrical data

9.3.2.1 Basic data

(1)P Design values for geometrical data shall be derived in accordance with the principles stated in 2.4.6.3.

9.3.2.2 Ground surfaces

(1)P Design values for the geometry of the retained material shall take account of the variation in the actual field values. The design values shall also take account of anticipated excavation or possible scour in front of the retaining structure.

(2) In ultimate limit state calculations in which the stability of a retaining wall depends on the ground resistance in front of the structure, the level of the resisting soil should be lowered below the nominally expected level by an amount Δa. The value of Δa should be selected taking into account the degree of site control over the level of the surface. With a normal degree of control, the following should be applied:

— for a cantilever wall, Δa should equal 10 % of the wall height above excavation level, limited to a maximum of 0,5 m;
— for a supported wall, Δa should equal 10 % of the distance between the lowest support and the excavation level, limited to a maximum of 0,5 m.

(3) Smaller values of Δa, including 0, may be used when the surface level is specified to be controlled reliably throughout the appropriate design situation.

(4) Larger values of Δa should be used where the surface level is particularly uncertain.

The requirements of 9.3.2.2 are similar to those of BS 8002, as illustrated in Figure X4. They are introduced because some embedded retaining walls are very sensitive to small amounts of over-excavation, scour or erosion. This is particularly the case for walls penetrating into dense soils and using 'free earth' support, for which calculations may show small penetrations (Simpson, 1992).

The additional 10% allowance is a safety margin, akin to a partial factor. It is required for ULS calculations, which, for DA1, include both Combinations 1 and 2. As is the case with load factors, this safety margin is not a licence to infringe the characteristic values. It is provided for *unforeseen* conditions and should be additional to any reasonably expected erosion or over-excavation, such as for service trenches.

The allowance is not required for SLS. However, if finite element methods are in use, it may sometimes be convenient to use the computations for DA1 also for SLS. The 10% allowance should then be included in the analysis and this will mean that the SLS computations are slightly more pessimistic than they need to be to satisfy the code.

9.3.2.3 Water levels

(1)P The selection of design or characteristic values for the positions of free water and phreatic surfaces shall be made on the basis of data for the hydraulic and hydrogeological conditions at the site.

(2)P Account shall be taken of the effects of variation in permeability on the ground-water regime.

(3)P The possibility shall be considered of adverse water pressures due to the presence of perched or artesian water tables.

Reference should be made to the comments on 2.4.6.1(7) and (8).

9.3.3 Design situations

Paragraph (1)P constitutes a checklist of design situations that shall be considered. This is provided in a ready-to-use form in Appendix 2.

**

9.4 Design and construction considerations

9.4.1 General

(1)P Both ultimate and serviceability limit states shall be considered using the procedures described in 2.4.7 and 2.4.8.

(2)P It shall be demonstrated that vertical equilibrium can be achieved for the assumed pressure distributions and actions on the wall.

Much of the commonly available software for analysis of embedded walls checks horizontal and moment equilibrium but not vertical equilibrium. However, an imbalance of vertical forces may lead to collapse (Simpson, 1994).

(3) The verification of vertical equilibrium may be achieved by reducing the wall friction parameters.

(4) As far as possible, retaining walls should be designed in such a way that there are visible signs of the approach of an ultimate limit state. The design should guard against the occurrence of brittle failure, e.g. sudden collapse without conspicuous preliminary deformations.

Avoidance of brittleness is always important in design. This is particularly true in geotechnics, where the actual distribution of forces in a supporting structure is very difficult to predict. Collapses due to brittle behaviour of strutting and retaining walls are of great concern (e.g. the Nicoll Highway collapse, Magnus et al, 2005, Simpson et al, 2008).

(5) For many earth retaining structures, a critical limit state should be considered to occur if the wall has displaced enough to cause damage to nearby structures or services. Although collapse of the wall may not be imminent, the degree of damage may considerably exceed a serviceability limit state in the supported structure.

Paragraph (5) considers a condition that ‘may considerably exceed a serviceability limit state’, which seems to be somewhere between serviceability and ultimate limit states. Paragraph (6) says that the Eurocodes methods and factors ‘are usually sufficient to prevent the occurrence of *ultimate* [italics added] limit states in nearby structures’. This may be contrasted with the note on 7.6.4.1(2) which says, for pile design: ‘the safety requirements for the ultimate limit state design are normally sufficient to prevent a *serviceability* limit state in the supported structure’. For retaining structures, an assessment of displacement is required if there are supported structures susceptible to serviceability limit states, and reliance on the ULS design is not sufficient. The process of assessing displacement is considered in 9.8.

(6) The design methods and partial factor values recommended by this standard are usually sufficient to prevent the occurrence of ultimate limit states in nearby structures, provided that the soils involved are of at least medium density or firm consistency and that adequate construction methods and sequences are adopted. Special care should be taken, however, with some highly over-consolidated clay deposits in which large at rest horizontal stresses may induce substantial movements in a wide area around excavations.

(7) The complexity of the interaction between the ground and the retaining structure sometimes makes it difficult to design a retaining structure in detail before the actual execution begins. In this case use of the observational method for the design (see 2.7) should be considered.

Paragraph (8)P constitutes a checklist of items relevant to design of retaining structures, which is provided in a ready-to-use form in Appendix 2.

**

9.4.2 Drainage systems

(1)P If the safety and serviceability of the designed structure depend on the successful performance of a drainage system, the consequences of its failure shall be considered, having regard for both safety and cost of repair. One of the following conditions (or a combination of them) shall apply:

— a maintenance programme for the drainage system shall be specified and the design shall allow access for this purpose;
— it shall be demonstrated both by comparable experience and by assessment of any water discharge, that the drainage system will operate adequately without maintenance.

Inadequate drainage is a frequent cause of failures of retaining walls. The design should facilitate maintenance of the system, unless there is a very high level of certainty that this is not needed.

(2) The quantities, pressures and eventual chemical content of any water discharge should be taken into account.

9.5 Determination of earth pressures

The determination of appropriate earth pressures is critical to the design of retaining structures. For walls supporting traffic, additional guidance is provided by PD 6694-1. For embedded retaining walls, reference to CIRIA Report C580 is recommended.

9.5.1 General

(1)P Determination of earth pressures shall take account of the acceptable mode and amount of any movement and strain, which may occur at the limit state under consideration.

In retaining wall design, this requirement is generally followed either by choosing appropriate values of earth pressure coefficients or by a ground-structure interaction analysis such as a finite element model. In either case, partial factors may be incorporated into the parameters adopted if the analysis is for ULS.

(2) In the following context the words "earth pressure" should also be used for the total earth pressure from soft and weathered rocks and should include the pressure of ground-water.

Paragraph (3)P constitutes a checklist of items relevant to calculations of the magnitudes of earth pressures and directions of forces resulting from them, which is provided in a ready-to-use form in Appendix 2.

**

(4) The amount of mobilised wall friction and adhesion should be considered as a function of:

— the strength parameters of the ground;
— the friction properties of the wall-ground interface;
— the direction and amount of movement of the wall relative to the ground;
— the ability of the wall to support any vertical forces resulting from wall friction and adhesion.

(5) The amount of shear stress, which can be mobilised at the wall-ground interface should be determined by the wall-ground interface parameter δ.

(6) A concrete wall or steel sheet pile wall supporting sand or gravel may be assumed to have a design wall ground interface parameter $\delta_d = k.\varphi_{cv;d}$. k should not exceed 2/3 for precast concrete or steel sheet piling.

In drained soil, the angle of wall friction is related to the 'constant volume' or 'critical state' angle of shearing resistance of the soil because it is assumed that construction of the wall may leave the ground in a disturbed state, loosening initially dense soil.

(7) For concrete cast against soil, a value of k = 1,0 may be assumed.

(8) For a steel sheet pile in clay under undrained conditions immediately after driving, no adhesive or frictional resistance should be assumed. Increases in these values may take place over a period of time.

(9)P The magnitudes of earth pressures and directions of resultant forces shall be calculated according to the selected design approach (see 2.4.7.3), and the limit state being considered.

(10) The value of an earth pressure at an ultimate limit state is generally different from its value at a serviceability limit state. These two values are determined from two fundamentally different calculations. Consequently, when expressed as an action, earth pressure cannot have a single characteristic value.

The note on 9.5.1(1)P above is relevant also to (10).

(11)P In the case of structures retaining rock masses, calculations of the ground pressures shall take account of the effects of discontinuities, with particular attention to their orientation, spacing, aperture, roughness and the mechanical characteristics of any joint filling material.

(12)P Account shall be taken of any swelling potential of the ground when calculating the pressures on the retaining structure.

Guidance on swelling pressures in clay fill may be found in O'Connor and Taylor (1994).

9.5.2 At rest values of earth pressure

(1)P When no movement of the wall relative to the ground takes place, the earth pressure shall be calculated from the at rest state of stress. The determination of the at rest state shall take account of the stress history of the ground.

(2) For normally consolidated soil, at rest conditions should normally be assumed in the ground behind a retaining structure if the movement of the structure is less than $5\times10^{-4}\times h$.

(3) For a horizontal ground surface, the at rest earth pressure coefficient, K_0, should be determined from:

$$K_0 = (1 - \sin\varphi') \times \sqrt{\mathrm{OCR}} \tag{9.1}$$

The formula should not be used for very high values of OCR.

In practice, it is often difficult to know the OCR of overconsolidated soils. It may be better to measure K_0 in situ or to consult literature relevant to the particular soil deposit, noting that at shallow depth below the surface of a stratum, K_0 may vary rapidly with depth.

(4) If the ground slopes upwards from the wall at an angle $\beta \leq \varphi'$ to the horizontal, the horizontal component of the effective earth pressure $\sigma'_{h;0}$ may be related to the effective overburden pressure q' by the ratio $K_{0;\beta}$, where

$$K_{0;\beta} = K_0 \cdot \left(1 + \sin\beta\right) \tag{9.2}$$

The direction of the resulting force should then be assumed to be parallel to the ground surface.

9.5.3 Limiting values of earth pressure

(1)P Limiting values of earth pressures shall be determined taking account of the relative movement of the soil and the wall at failure and the corresponding shape of the failure surface.

(2) Limiting values of earth pressure assuming straight failure surfaces can significantly deviate from the values assuming curved failure surfaces for high angles of shearing resistance and wall-ground interface parameters δ, and so lead to unsafe results.

NOTE Annex C provides some data of relative movements that cause limiting values of earth pressures.

Besides providing data on 'movements that cause limiting values of earth pressures', Annex C provides values for the coefficients of active and passive pressure that should be used in calculations. Paragraph (2) means that the Coulomb formulae for active and passive earth pressure should, in general, not be used. This is particularly so if the angle of wall friction exceeds about one third of the angle of shearing resistance of the soil, which is usually the case.

(3) In cases where struts, anchorages or similar elements impose restraints on movement of the retaining structure, it should be considered that the limiting active and passive values of earth pressure, and their distributions, may not be the most adverse ones.

9.5.4 Intermediate values of earth pressure

(1)P Intermediate values of earth pressure occur if the wall movements are insufficient to mobilise the limiting values. The determination of the intermediate values of earth pressure shall take account of the amount of wall movement and its direction relative to the ground.

NOTE Annex C, figure C.3, gives a diagram, which may be used for the determination of the mobilised passive earth pressure

(2) The intermediate values of earth pressures may be calculated using, for example, various empirical rules, spring constant methods or finite element methods.

9.5.5 Compaction effects

(1)P The determination of earth pressures acting behind the wall shall take account of the additional pressures generated by any placing of backfill and the procedures adopted for its compaction.

NOTE Measurements indicate that the additional pressures depend on the applied compactive energy, the thickness of the compacted layers and the travel pattern of the compaction plant. Horizontal pressure normal to the wall in a layer may reduce when the next layer is placed and compacted. When backfilling is complete, the additional pressure normally acts only on the upper part of the wall.

Design could use the theories of Ingold (1979, 1980), which appear in many textbooks and in PD 6694-1.

(2)P Appropriate compaction procedures shall be specified with the aim of avoiding excessive additional earth pressures, which may lead to unacceptable movements.

9.6 Water pressures

(1)P Determination of characteristic and design water pressures shall take account of water levels both above and in the ground.

(2)P When checking the ultimate and serviceability limit states, water pressures shall be accounted for in the combinations of actions in accordance with 2.4.5.3 and 2.4.6.1, considering the possible risks indicated in 9.4.1(5).

The reference to 2.4.6.1 relates to paragraphs (6) to (8), on which notes are provided in this book.

(3) For structures retaining earth of medium or low permeability (silts and clays), water pressures should normally be assumed to act behind the wall. Unless a reliable drainage system is installed (9.4.2(1)P), or infiltration is prevented, the values of water pressures should normally correspond to a water table at the surface of the retained material.

(4)P Where sudden changes in a free water level may occur, both the non-steady condition occurring immediately after the change and the steady condition shall be examined.

(5)P Where no special drainage or flow prevention measures are taken, the possible effects of water-filled tension or shrinkage cracks shall be considered.

9.7 Ultimate limit state design

9.7.1 General

(1)P The design of retaining structures shall be checked at the ultimate limit state for the design situations appropriate to that state, as specified in 9.3.3, using the design actions or action effects and design resistances.

(2)P All relevant limit modes shall be considered. These will include, as a minimum, limit modes of the types illustrated in figures 9.1 to 9.6 for the most commonly used retaining structures.

(3)P Calculations for ultimate limit states shall establish that equilibrium can be achieved using the design actions or effects of actions and the design strengths or resistances, as specified in clause 2.4. Compatibility of deformations shall be considered in assessing design strengths or resistances.

(4)P Upper or lower design values, whichever are more adverse, shall be used for the strength or resistance of the ground.

(5) Calculation methods may be used, which redistribute earth pressure in accordance with the relative displacements and stiffnesses of ground and structural elements.

Paragraph (5) makes it clear that various forms of numerical analysis are acceptable for ULS checks.

(6)P For fine grained soils, both short- and long-term behaviour shall be considered.

(7)P For walls subject to differential water pressures, safety against failure due to hydraulic heave and piping shall be checked.

These issues are considered in more detail in Section 10 of BS EN 1997-1, which is not included in this book.

9.7.2 Overall stability

(1)P The principles in Section 11 shall be used as appropriate to demonstrate that an overall stability failure will not occur and that the corresponding deformations are sufficiently small.

(2) As a minimum, limit modes of the types illustrated in figure 9.1 should be considered, taking progressive failure and liquefaction into account as relevant.

9.7.3 Foundation failure of gravity walls

(1)P The principles of Section 6 shall be used as appropriate to demonstrate that a foundation failure is sufficiently remote and that deformations will be acceptable. Both bearing resistance and sliding shall be considered.

(2) As a minimum, limit modes of the types illustrated in Figure 9.2 should be considered.

Figure 9.1 — Examples of limit modes for overall stability of retaining structures.

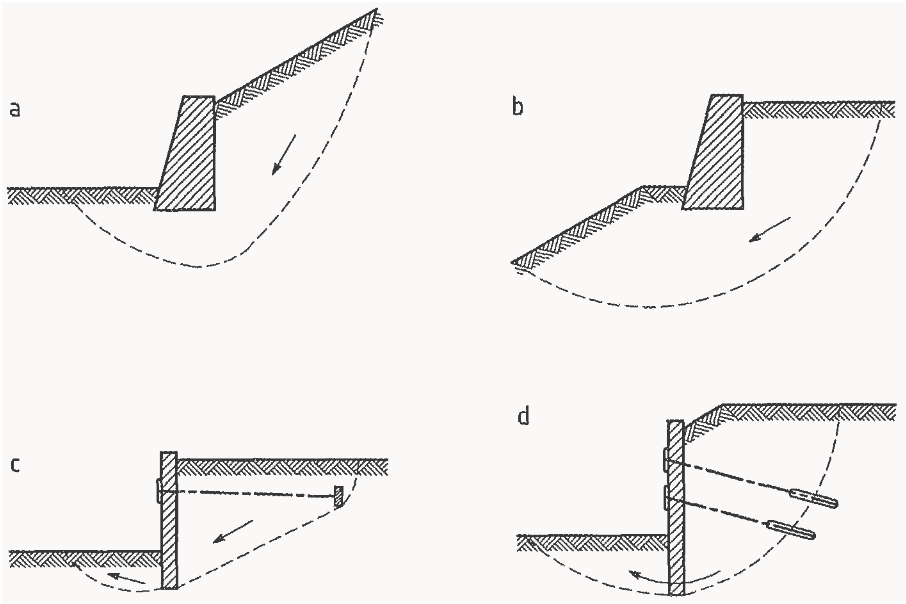
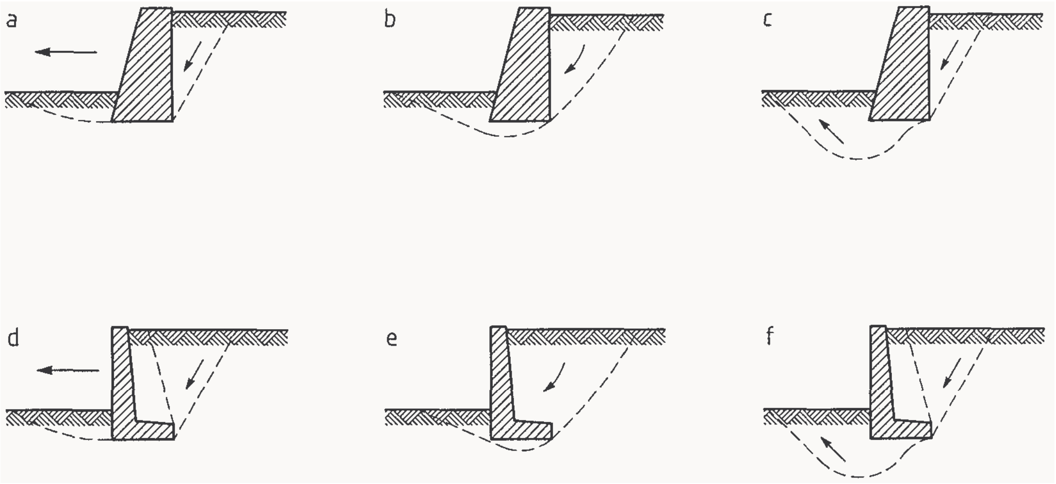


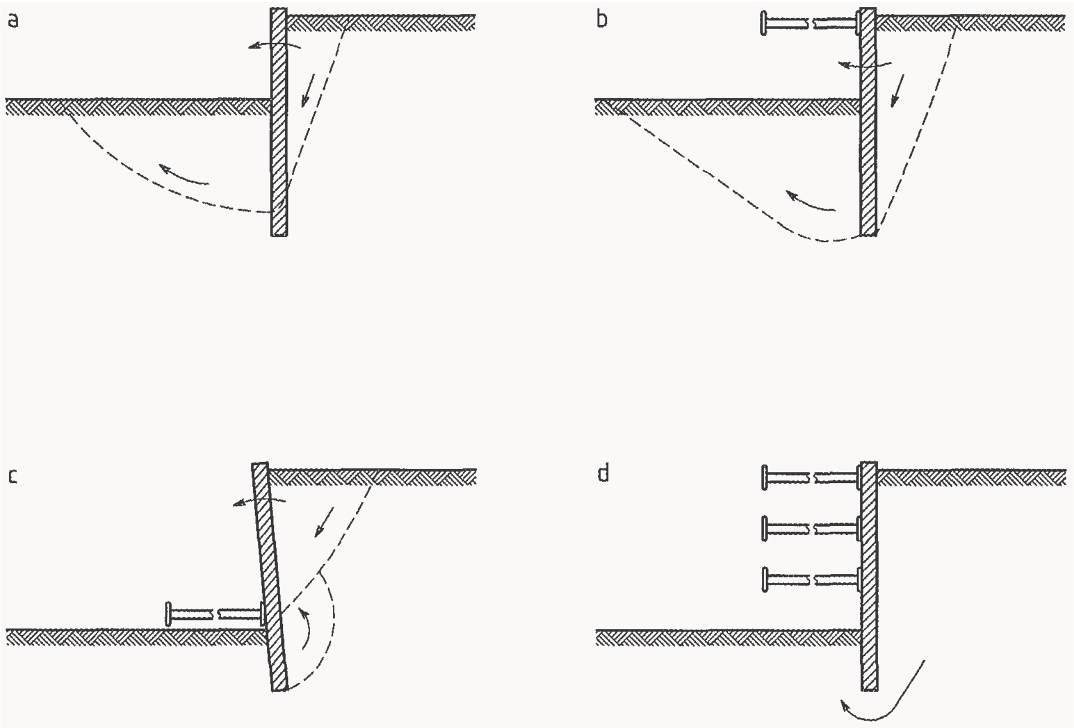
Figure 9.2 — Examples of limit modes for foundation failures of gravity walls



9.7.4 Rotational failure of embedded walls

(1)P It shall be demonstrated by equilibrium calculations that embedded walls have sufficient penetration into the ground to prevent rotational failure.

Figure 9.3 — Examples of limit modes for rotational failures of embedded walls



(2) As a minimum, limit modes of the types illustrated in Figure 9.3 should be considered.

(3)P The design magnitude and direction of shear stress between the soil and the wall shall be consistent with the relative vertical displacement, which would occur in the design situation.

9.7.5 Vertical failure of embedded walls

(1)P It shall be demonstrated that vertical equilibrium can be achieved using the design soil strengths or resistances and design vertical forces on the wall.

(2) As a minimum, the limit mode of the type illustrated in Figure 9.4 should be considered.

(3)P Where downward movement of the wall is considered, upper design values shall be used in the calculation of prestressing forces, such as those from ground anchorages, which have a vertical downward component.

(4)P The design magnitude and direction of shear stress between the soil and the wall shall be consistent with the check for vertical and rotational equilibrium.

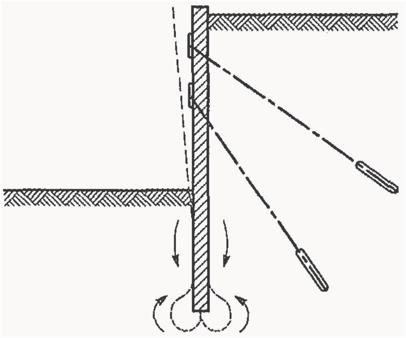
Paragraph (4) relates to a common error whereby wall friction from the retained soil is assumed to support the wall by acting upwards on the wall in checking vertical equilibrium, but is implicitly assumed to act downwards on the wall in the choice of earth pressure coefficient for the earth pressure calculation. This is important because friction acting up on the wall, and down on the soil, considerably increases the coefficient of active pressure.

In checking that there will not be an ultimate limit state, the whole system should be shown to be in horizontal and vertical equilibrium in a consistent manner. The actual direction of these friction forces may be uncertain to the designer but the design will usually be safe so long as it is shown to be fully in equilibrium for a consistent set of forces.

It is sometimes convenient, to avoid uncertainty or complicated calculation, to take an inconsistent but pessimistic combination of forces. For example, the friction from the retained soil might be taken to act down on the wall in checking vertical equilibrium but the potential advantage of this might be ignored in the earth pressure calculation. Paragraph (5) clarifies that in cases where retaining walls are used to carry vertical loads, the vertical equilibrium should be checked against the requirements for piles. Since this may imply that more severe partial factors are used in this check than in the retaining wall calculation, there is a conservative inconsistency between the two checks.

(5)P If the wall acts as the foundation for a structure, vertical equilibrium shall be checked using the principles of Section 7.

Figure 9.4 — Example of a limit mode for vertical failure of embedded walls

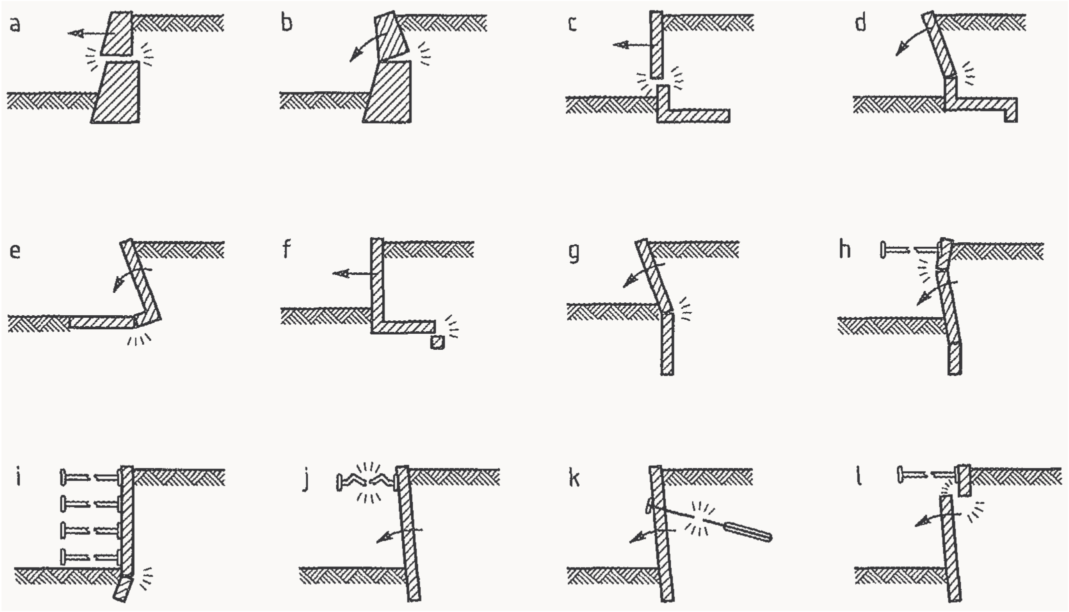


9.7.6 Structural design of retaining structures

(1)P Retaining structures, including their supporting structural elements such as anchorages and props, shall be verified against structural failure in accordance with 2.4 and EN 1992, EN 1993, EN 1995 and EN 1996.

(2) As a minimum, limit modes of the types illustrated in Figure 9.5 should be considered.

Figure 9.5 — Examples of limit modes for structural failure of retaining structures



(3)P For each ultimate limit state, it shall be demonstrated that the required strengths can be mobilised, with compatible deformations in the ground and the structure.

(4) In structural elements, reduction in strength with deformation due to effects such as cracking of unreinforced sections, large rotations at plastic hinges or local buckling of steel sections should be considered in accordance with EN 1992 to EN 1996 and EN 1999.

9.7.7 Failure by pull-out of anchorages

Design of anchorages is outside the scope of this book.

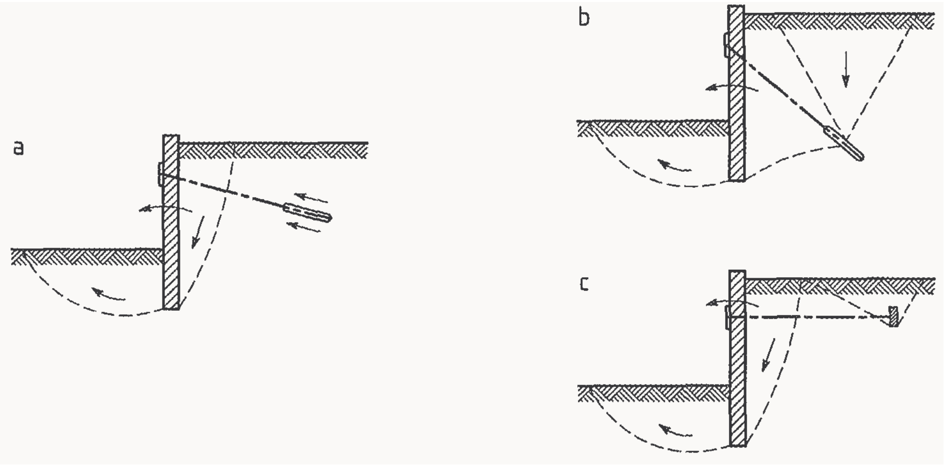
(1)P It shall be demonstrated that equilibrium can be achieved without pull-out failure of ground anchorages.

(2)P Anchors shall be designed in accordance with Section 8.

(3) As a minimum, limit modes of the types illustrated in Figure 9.6 (a, b) should be considered.

(4) For deadman anchors, the failure mode illustrated in Figure 9.6 (c) should also be considered.

Figure 9.6 — Examples of limit modes for failure by pull-out of anchors.



9.8 Serviceability limit state design

9.8.1 General

(1)P The design of retaining structures shall be checked at the serviceability limit state using the appropriate design situations as specified in 9.3.3.

(2) The assessment of design values of earth pressures should take account of the initial stress, stiffness and strength of the ground and the stiffness of the structural elements.

(3) The design values of earth pressures should be derived taking account of the allowable deformation of the structure at its serviceability limit state. These pressures need not necessarily be limiting values.

9.8.2 Displacements

(1)P Limiting values for the allowable displacements of walls and the ground adjacent to them shall be established in accordance with 2.4.8, taking into account the tolerance to displacements of supported structures and services.

(2)P A cautious estimate of the distortion and displacement of retaining walls, and the effects on supported structures and services, shall always be made on the basis of comparable experience. This estimate shall include the effects of construction of the wall. The design may be justified by checking that the estimated displacements do not exceed the limiting values.

A 'cautious estimate' of the displacement of walls is always required. Qualifying this, it should be noted that the limit state does not require an actual assessment of the displacement but merely states that it is unlikely to exceed the 'limiting value' set in accordance with paragraph (1). In some cases, it may be relatively easy to conclude that it will not exceed the limit, even though its actual value is difficult to assess with confidence.

Reliable calculation of the displacement of a retaining wall is generally difficult to achieve. To avoid unnecessary, and possibly spurious, calculation, EC7 allows the design to be justified by assessment of displacements on the basis of 'comparable experience' (as defined in 1.5.2.2). Usually this will either mean local experience of similar walls in similar situations or reference to publications such as CIRIA Report C580, which gives references to other source documents.

For sensitive situations, careful monitoring of retaining wall displacements is often valuable, though it is not specifically mentioned by the code at this point. This could be linked to use of the observational method, as described in 2.7.

The following paragraphs have been retained for information, though further development of them is beyond the scope of this book. They give warning of situations in which specialist analysis is required.

(3)P If the initial cautious estimate of displacement exceeds the limiting values, the design shall be justified by a more detailed investigation including displacement calculations.

(4)P It shall be considered to what extent variable actions, such as vibrations caused by traffic loads behind the retaining wall, contribute to the wall displacement.

The effects of traffic loads are considered in more detail in PD 6694-1.

(5)P A more detailed investigation, including displacement calculations, shall be undertaken in the following situations:

— where nearby structures and services are unusually sensitive to displacement;
— where comparable experience is not well established.

(6) Displacement calculations should also be considered in the following cases:

— where the wall retains more than 6 m of cohesive soil of low plasticity,
— where the wall retains more than 3 m of soils of high plasticity;
— where the wall is supported by soft clay within its height or beneath its base.

(7)P Displacement calculations shall take account of the stiffness of the ground and structural elements and the sequence of construction.

(8) The behaviour of materials assumed in displacement calculations should be calibrated by comparable experience with the same calculation model. If linear behaviour is assumed, the stiffnesses adopted for the ground and structural materials should be appropriate for the degree of deformation computed. Alternatively, complete stress-strain models of the materials may be adopted.

(9)P The effect of vibrations on displacements shall be considered with regard to 6.6.4.

Section 10 Hydraulic failure

Designs that could be subject to hydraulic failure are outside the scope of this book. Nevertheless, this issue is sometimes very important in those situations to which it is relevant and reference should then be made to the full text, which is not reproduced here.

Development of a formal procedure for safety provision in situations dominated by water pressure has proved to be elusive and contentious. It is important that designers consider practically and imaginatively the possible water pressures that could develop and ensure that they are accommodated by the design. The UKNA to BS EN 1997-1 notes the following in A.2.1 and repeats similar text when providing values for partial factors related to water pressures:

> The partial factors specified in the National Annex to BS EN 1990:2002 might not be appropriate for self-weight of water, ground-water pressure and other actions dependent on the level of water, see **2.4.7.3.2**(2). The design value of such actions may be directly assessed in accordance with **2.4.6.1**(2)P and **2.4.6.1**(6)P of BS EN 1997-1:2004. Alternatively, a safety margin may be applied to the characteristic water level, see **2.4.6.1**(8) of BS EN 1997-1:2004.

Further discussion of design for water pressures can be found in Simpson et al (2011).

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Section 11 Overall stability

Before developing the design of the elements considered in Sections 6 to 10 and 12, it is necessary to establish that the construction site (and potentially land beyond the site boundaries) will essentially be stable as the loading on it changes and the construction works progress. Hence each of those sections refer to and rely on this Section 11. The section is also relevant to situations in which new construction has a minimal effect but the stability of an existing slope is important.

11.1 General

(1)P The provisions in this Section shall apply to the overall stability of and movements in the ground, whether natural or fill, around foundations, retaining structures, natural slopes, embankments or excavations.

(2) Account should be taken of overall stability clauses, related to specific structures, in Sections 6 to 10 and 12.

11.2 Limit states

(1)P All possible limit states for the particular ground shall be considered in order to fulfil the fundamental requirements of stability, limited deformations, durability and limitations in movements of nearby structures or services.

(2) Some possible limit states are listed below:

— loss of overall stability of the ground and associated structures;
— excessive movements in the ground due to shear deformations, settlement, vibration or heave;
— damage or loss of serviceability in neighbouring structures, roads or services due to movements in the ground.

11.3 Actions and design situations

(1) The list in 2.4.2(4) should be taken into account when selecting the actions for calculation of limit states.

Paragraph 11.3(2)P constitutes a checklist of actions and design situations relevant to overall stability, which is provided in a ready-to-use form in Appendix 2.

Paragraphs (3) to (6) emphasise the critical importance of selecting design water pressures carefully and cautiously.

**

11.4 Design and construction considerations

(1)P The overall stability of a site and movements of natural or made ground shall be checked taking into account comparable experience, according to 1.5.2.2.

(2)P The overall stability and movement of ground supporting existing buildings, new structures, slopes or excavations shall be considered.

(3) In cases where the stability of the ground cannot be clearly verified prior to design, additional investigations, monitoring and analysis should be specified according to the provisions of 11.7.

(4) Typical structures for which an analysis of overall stability should be performed are:

— ground retaining structures;
— excavations, slopes or embankments;
— foundations on sloping ground, natural slopes or embankments;
— foundations near an excavation, cut or buried structures, or shore.

NOTE Stability problems or creep movements occur primarily in cohesive soils with a sloping ground surface. However, instability can also occur in non-cohesive soils and fissured rocks in slopes where the inclination, which may be determined by erosion, is close to the angle of shearing resistance. Increased movements are often observed at elevated pore-water pressures or close to the ground surface during freezing and thawing cycles.

(5)P If the stability of a site cannot readily be verified or the movements are found to be not acceptable for the site's intended use, the site shall be judged to be unsuitable without stabilising measures.

(6)P The design shall ensure that all construction activities in and on the site can be planned and executed such that the occurrence of an ultimate or serviceability limit state is sufficiently improbable.

(7)P Slope surfaces exposed to potential erosion shall be protected if required, to ensure that the safety level is retained.

(8) Slopes should be sealed, planted or protected artificially. For slopes with berms, a drainage system within the berm should be considered.

(9)P Construction processes shall be taken into account as far as they might affect the overall stability or the magnitude of movement.

(10) Potentially unstable slopes may be stabilised by:

— a concrete cover with or without anchorage;
— an abutment of gabions, either of steel net or geotextile cages;
— ground nailing;
— vegetation;
— a drainage system;
— a combination of the above.

(11) The design should follow the general principles of Sections 8 and 9.

11.5 Ultimate limit state design

11.5.1 Stability analysis for slopes

(1)P The overall stability of slopes including existing, affected or planned structures shall be verified in ultimate limit states (GEO and STR) with design values of actions, resistances and strengths, where the partial factors defined in A.3.1(1)P, A.3.2(1)P and A.3.3.6(1)P shall be used.

The values of partial factors to be used for construction in the UK are provided in the UKNA, paragraphs A.3.1, A.3.2 and A.3.3.6.

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(2)P In analysing the overall stability of the ground, of soil or rock, all relevant modes of failure shall be taken into account.

Paragraphs (3) to (9) give general advice about the requirements of slope stability analyses.

Paragraph (8) relates to existing slopes. These can sometimes be justified on the basis of observation of their apparent stability, provided that any new works will make them no less stable and will preferably improve their level of safety. This can sometimes be achieved by providing drainage to reduce water pressures, for example.

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(8) Existing failed slopes, which can potentially be reactivated should be analysed, considering circular, as well as non-circular failure surfaces. Partial factors normally used for overall stability analyses then need not be appropriate.

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(10) A slope analysis should verify the overall moment and vertical stability of the sliding mass. If a method of slices is used and horizontal equilibrium is not checked, the inter-slices forces should be assumed to be horizontal.

Paragraph (10) sets a critical requirement for the types of analysis that can be used in the method of slices. Its implication is to preclude use of the ‘Swedish Circle Method’ and ‘Janbu’s simplified method’. (The Swedish Circle Method is sometimes known as the ‘Fellenius method’ or the ‘Ordinary Method of Slices’.) Janbu’s simplified method has no check on vertical equilibrium. Some of the fundamentals that underlie this requirement are discussed by Whitman and Bailey (1967) and by Bromhead (1992).

**

(12) Since a distinction between favourable and unfavourable gravity loads is not possible in assessing the most adverse slip surface, any uncertainty about weight density of the ground should be considered by applying upper and lower characteristic values of it.

Paragraph (12) makes it clear that no attempt should be made to partition the sliding mass into ‘favourable’ and ‘unfavourable’ ground. Even when the Design Approach or combination in use requires different factors on favourable or unfavourable permanent actions, the weight of the ground is to be considered as a ‘single source’ in the terms of 2.4.2(9).

(13)P The design shall show that the deformation of the ground under design actions due to creep or regional settlements will not cause unacceptable damage to structures or infrastructure sited on, in or near the particular ground.

Paragraph (13)P is included in this sub-clause on ultimate limit states although it does not refer to states of complete mechanism in the ground. Presumably, this is because deformation, creep or regional settlement might be so severe as to cause severe damage and danger to supported structures. It is unlikely that slopes that conform to the factors of safety required in the UKNA would be problematic in this regard. (The UK factors are currently the same as the CEN default values for this situation.) However, the paragraph is particularly relevant to existing slopes considered as under paragraph (8).

11.5.2 Slopes and cuts in rock masses

Design of slopes and cuts in rock masses is not included within the scope of this book.

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11.5.3 Stability of excavations

(1)P The overall stability of the ground close to an excavation, including excavation spoil and existing structures, roads and services shall be checked (see Section 9).

(2)P The stability of the bottom of an excavation shall be checked in relation to the design pore-water pressure in the ground. For the analysis of hydraulic failure (see Section 10).

(3)P Heave of the bottom of deep excavations due to unloading shall be considered.

11.6 Serviceability limit state design

(1)P The design shall show that the deformation of the ground will not cause a serviceability limit state in structures and infrastructure on or near the particular ground.

(2) Subsidence of the ground due to the following causes should be considered:

— change in ground-water conditions and corresponding pore-water pressures;
— long-term creep under drained conditions;
— volume loss of deep soluble strata;
— mining or similar works such as gas extraction.

(3) Since the analytical and numerical methods available at present do not usually provide reliable predictions of the deformation of a natural slope, the occurrence of serviceability limit states should be avoided by one of the following:

— limiting the mobilised shear strength;
— observing the movements and specifying actions to reduce or stop them, if necessary.

Paragraph (3), related to SLS, is similar to 11.5.1(13), which is related to ULS. It is unlikely that slopes that conform to the full factors of safety required in the UKNA would be problematic in this regard.

11.7 Monitoring

Monitoring is not included within the scope of this book.

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Section 12 Embankments

Design of embankments is not included within the scope of this book. More extensive guidance can be found in BS 6031:2009 (*Code of practice for earthworks*), which uses the concepts of this section.

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Annex A (normative)
Partial and correlation factors for ultimate limit states and recommended values

Annex A is completely replaced by Annex A of the UKNA.

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UK National Annex to Eurocode 7: Geotechnical design – Part 1: General rules

Annex A (informative)
Design Approach and values of partial, correlation and model factors for ultimate limit states to be used in conjunction with BS EN 1997-1:2004

Because Annex A of the UKNA provides all the values of partial factors required for geotechnical design, it is included here almost in full.

A.1 Nationally Determined Parameters

A.1.1

Annex A provides values for partial factors γ and correlation factors ξ. It includes values of model factors for pile design, here termed γ_{Sd}, with some additional comments on use of model factors in other situations.

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A.1.2 As stated in **NA.2**, paragraph 1, only Design Approach 1 is used in the UK for the STR and GEO limit states. This Annex therefore only provides partial factors appropriate for Design Approach 1.

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The partial factors specified for permanent actions in this Annex have been established to be consistent with the principle that a single partial factor can be applied to permanent actions arising from a single source for the STR and GEO limit states (see Note to **2.4.2**(9)P of BS EN 1997-1:2004).

The single source principle is particularly relevant to consideration of water pressures. This affects A.2.1, A.3.1 and A.5, and the EQU limit state discussed under 2.4.7.2 in this book.

A.2 Partial factors for the equilibrium limit state (EQU) verification

For comment on the EQU limit state calculations, see the notes in this book under 2.4.7.2.

A.2.1 Partial factors on actions (γ_F)

For the verification of the equilibrium limit state (EQU), the values of the partial factors on actions can be found in the National Annex to BS EN 1990:2002, using Table NA.A1.2(A) (Set A) for buildings and Table NA.A2.4(A) (Set A) for bridges. The terms $\gamma_{G;sup}$ and $\gamma_{G;inf}$ in BS EN 1990:2002 correspond with $\gamma_{G;dst}$ and $\gamma_{G;stb}$ in BS EN 1997-1:2004. Table A.NA.1 below shows the appropriate tables in BS EN 1990:2002.

In cases where overturning instability of a structure could occur without the resistance of the ground being exceeded the partial factors specified in the National Annex to BS EN 1990:2002 can give an overall factor of safety on overturning lower than that from which confidence has been gained through past UK practice. In such cases it is recommended that consideration be given to the use of higher partial factors.

In practice, it is very unlikely that a structure will have an overturning instability (under the EQU requirements) without the resistance of the ground being exceeded. This would usually require very high concentrated stresses, which would fail the ground (in the STR/GEO requirements) or, alternatively, would cause failure in the structure.

The partial factors specified in the National Annex to BS EN 1990:2002 might not be appropriate for self-weight of water, ground-water pressure and other actions dependent on the level of water, see **2.4.7.3.2**(2). The design value of such actions may be directly assessed in accordance with **2.4.6.1**(2)P and **2.4.6.1**(6)P of BS EN 1997-1:2004. Alternatively, a safety margin may be applied to the characteristic water level, see **2.4.6.1**(8) of BS EN 1997-1:2004.

The issue of finding design water pressures is discussed in EC7 under 2.4.6.1(6).

The design value of earth pressures should be based on the design value of the actions giving rise to the earth pressure. For bridge design, in some cases, additional model factors might be required when evaluating horizontal earth pressures (see **A.6.3** of this National Annex).

Derivation of design values for earth pressures will include partial factors; in some cases, these may have the value 1.0. PD 6694-1 suggests that, for structures supporting traffic loads, it will be necessary to apply an additional model factor to earth pressures if previously obtained levels of safety and robustness are to be maintained.

Actions listed in BS EN 1997-1:2004, **2.4.2** for which no values are set in BS EN 1991 may be specified for a particular project. The values of these actions and their partial factors and combination factors should be agreed with the client and relevant authorities.

All the actions listed in 2.4.2(4) are shown in a checklist in Appendix 2.

Table A.NA.1 — Partial factors on Actions (γ_F) for the equilibrium (EQU) limit state

Structure	Value
Buildings	See Table NA.A1.2(A) in the National Annex to BS EN 1990:2002
Bridges	See Table NA.A2.4(A) in the National Annex to BS EN 1990:2002

Figure X7 shows the text of the UKNA to BS EN 1990 from NA.2.2 for buildings. For bridges, the equivalent information, which is more complex and extensive, is given in NA.2.3.

A.2.2 Partial factors for soil parameters (γ_M)

For the verification of the equilibrium limit state (EQU) the values of the partial factors on soil parameters should be taken from Table A.NA.2.

Table A.NA.2 — Partial factors for soil parameters (γ_M) for the EQU limit state

Soil parameter	Symbol	Value
Angle of shearing resistance[A)]	$\gamma_{\varphi'}$	1.1
Effective cohesion	$\gamma_{c'}$	1.1
Undrained shear strength	γ_{cu}	1.2
Unconfined strength	γ_{qu}	1.2

A) Applied to tan φ' and tan φ'_{cv}, although it might be more appropriate to determine the design value of φ'_{cv} directly

NOTE The value of the partial factor should be taken as the reciprocal of the specified value if such a reciprocal value produces a more onerous effect than the specified value (but see also the Note to ***2.4.2****(9)P of BS EN 1997-1:2004).*

Although EQU is primarily concerned with situations where actions are nearly balanced, it will often be the case that some material strength is used to mitigate the slight imbalance modelled by the partial factors. It is therefore necessary to have material factors for EQU. For further discussion on EQU, see notes under 2.4.7.2 in this book. For comment on the use of φ'_{cv} and the derivation of appropriate design values, see the notes under Table A.NA.4 in this book.

A.3 Partial factors for structural (STR) and geotechnical (GEO) limit states verification

The partial factors required for the design of buildings to ULS STR/GEO were summarised in Figure X1. For pile design, further factors γ_{Rd} and ξ are needed, as noted below.

A.3.1 Partial factors on actions (γ_F) or the effects of actions (γ_E)

Table A.NA.3 — Partial factors on actions (γ_F) or the effects of actions (γ_E) for the structural (STR) and geotechnical (GEO) limit states

Structure type	Value	
	Set A1	**Set A2**
Buildings	See Table NA.A1.2(B) in the National Annex to BS EN 1990:2002	See Table NA.A1.2(C) in the National Annex to BS EN 1990:2002
Bridges	See Table NA.A2.4(B) in the National Annex to BS EN 1990:2002	See Table NA.A2.4(C) in the National Annex to BS EN 1990:2002

Figures X8 and X9 show the text of the UKNA to BS EN 1990 from NA.2.2 for *buildings*. The Set A1 values in BS EN 1997-1 are called Set B in BS EN 1990, and set A2 are called Set C. For *bridges*, the equivalent information, which is more complex and extensive, is given in NA.2.3.

The partial factors specified in the National Annex to BS EN 1990:2002 might not be appropriate for self-weight of water, ground-water pressure and other actions dependent on the level of water, see **2.4.7.3.2**(2). The design value of such actions may be directly assessed in accordance with **2.4.6.1**(2)P and **2.4.6.1**(6)P of BS EN 1997-1:2004. Alternatively, a safety margin may be applied to the characteristic water level, see **2.4.6.1**(8) of BS EN 1997-1:2004.

The issue of finding design water pressures is discussed in this book under 2.4.6.1(6).

The design value of earth pressures should be based on the design value of the actions giving rise to the earth pressure. For bridge design, in some cases, additional model factors might be required when evaluating horizontal earth pressures, see **A.6.3** of this National Annex.

Derivation of design values for earth pressures will include partial factors; in some cases, these may have the value 1.0. PD 6694-1 suggests that, for structures supporting traffic loads, it will be necessary to apply an additional model factor to earth pressures if previously obtained levels of safety and robustness are to be maintained.

Actions listed in BS EN 1997-1:2004 **2.4.2** for which no values are set in BS EN 1991 may be specified for a particular project. The values of these actions and their partial factors and combination factors might need to be agreed with the client and relevant authorities.

A.3.2 Partial factors for soil parameters (γ_M)

For the verification of the structural (STR) and geotechnical (GEO) limit states, the values of the partial factors on soil parameters should be taken from Table A.NA.4.

Table A.NA.4 — Partial factors for soil parameters (γ_M) for the STR and GEO limit state

Soil parameter	Symbol	Set	
		M1	M2
Angle of shearing resistance[A)]	$\gamma_{\varphi'}$	1.0	1.25
Effective cohesion	$\gamma_{c'}$	1.0	1.25
Undrained shear strength	γ_{cu}	1.0	1.4
Unconfined strength	γ_{qu}	1.0	1.4

A) Applied to tan φ' and tan φ'_{cv}, although it might be more appropriate to determine the design value of φ'_{cv} directly.

*NOTE The value of the partial factor should be taken as the reciprocal of the specified value if such a reciprocal value produces a more onerous effect than the specified value (but see also the Note to **2.4.2**(9)P in BS EN 1997-1:2004).*

The same value (1.25) is given for the factors on both angle of shearing resistance and effective cohesion. This has been controversial because it is generally considered that effective cohesion is less reliable than the angle of shearing resistance. Paragraph 2.4.5.2(3) notes this fact, and requires that it be considered in selecting characteristic values.

As discussed in the comments on 2.4.5.2, the symbol φ'_{cv} represents the 'constant volume' or 'critical state' angle of shearing resistance of a soil (i.e. its angle of shearing resistance in a loose, disturbed state). It is often possible to be more confident about the value of φ'_{cv} than of any higher values that might be operative in more compact soil. Furthermore, φ' for compact soil tends to fall to φ'_{cv} as strains become large, which would usually happen if an ultimate limit state developed. It is therefore arguable that the *design* value of φ'_{cv} for ULS calculations could be assessed directly, without the need for a further reduction factor. If this is done, however, it is important to bear in mind that the partial factors applied to loads and materials in reality have to provide a sufficient margin to cover other minor eventualities such as imperfections in geometry or other features of construction. It is therefore important that a sufficient margin remains for these.

The use of φ'_{cv} is specifically required by EC7 to limit interface friction between structures and soil (6.5.3(10), 9.5.1(6)). Guidance on selection of values for φ'_{cv} can be found in PD 6694-1.

A.3.3 Partial resistance factors (γ_R)

A.3.3.1 Partial resistance factors for spread foundations

For the verifications of the structural (STR) and geotechnical (GEO) limit states the values of the partial factors $\gamma_{R;v}$ on bearing resistance and $\gamma_{R;h}$ on sliding resistance should be as given in Table A.NA.5.

Table A.NA.5 — Partial resistance factors (γ_R) for spread footings for the STR and GEO limit states

Resistance	Symbol	Set R1
Bearing	$\gamma_{R;v}$	1.0
Sliding	$\gamma_{R;h}$	1.0

In Design Approach 1, adopted by the UK, partial resistance factors are, in effect, not used for bearing and sliding. In a formal sense, BS EN 1997-1, 2.4.7.3.4.2(1) shows that the resistance factors to be used are set R1, and the annex shows that their values are 1.0.

A.3.3.2 Partial resistance factors for pile foundations

A.3.3.2 provides the values of partial factors on base and shaft resistance, γ_b and γ_s, for use with Section 7 of EC7.

For most aspects of ULS analysis, Design Approach 1, adopted by the UKNA, uses partial factors on material strength rather than on 'resistance'. For piles and anchors, however, resistance factors are used, as noted in 2.4.7.3.4.2(2)P. These factors can also be seen as material factors, chosen for the specific zones of soil affecting the ultimate resistance of piles, adjacent to the pile shaft and local to the pile base. One important reason for the use of resistance factors in this case is that load testing is an important feature of pile design; the tests measure resistance directly, rather than soil strength.

Pile design in the UK is usually based on calculations that use measured or derived soil properties (Equation 7.9). The calculations are often supplemented by loading tests. The UKNA provides a set of factors for use in calculations using material strengths that vary according to the extent of available loading tests. The model factor γ_{Rd} can be reduced if at least one pile is test loaded to the calculated, unfactored ultimate resistance (a typical trial pile test). The partial factors on shaft and base, γ_s and γ_b, can be reduced if 1% of the piles are tested to 1.5 times the representative load (a typical working pile test).

Figure X11 illustrates the process required, which is described in the text of the UKNA below. The calculation model used to derive pile resistance from ground strength is an uncertain one. In this case, the uncertainty lies in the derivation of the *characteristic* resistance, and for this reason a model factor γ_{Rd} has been introduced at this stage in the process. A more detailed discussion can be found in Bond and Simpson (2009–10).

The possibility of designing based principally on load testing or other forms of testing is also allowed by the UKNA, with factors provided in A.3.3.3 below. Even if this route is followed, however, the test results should always be shown by calculation to lie within reasonably expected bounds, following 7.4.1.

The values of factors provided here are considered to be generally applicable for pile foundations. However, variation of these factors is permitted in particular circumstances when justified by thorough consideration and documented experience, and after being agreed, where appropriate, with the client and other relevant authorities.

For verifications of the structural (STR) and geotechnical (GEO) limit states of pile foundations, the values of the partial factors on resistance (γ_R) should be those given in Table A.NA.6, Table A.NA.7 and Table A.NA.8. These values are used to convert characteristic resistances to design values for ultimate limit state calculations. They apply irrespective of the process by which the characteristic resistances are derived.

Characteristic resistances may be derived from static load tests using EN1997-1 **7.6.2.2** (**7.6.3.2** for tensile loading), or from ground test results using EN1997-1 Equations 7.8 or 7.9 (7.17 or 7.18 for tensile loading). When the approach of Equations 7.9 or 7.18 is used to derive the characteristic resistances, a model factor should be applied to the shaft and base resistance calculated using characteristic values of soil properties by a method complying with EN1997-1, **2.4.1**(6). The value of the model factor should be 1.4, except that it may be reduced to 1.2 if the resistance is verified by a maintained load test taken to the calculated, unfactored ultimate resistance.

Table A.NA.6 — Partial resistance factors (γ_R) for driven piles for the STR and GEO limit states

Resistance	**Symbol**	**Set**		
		R1	**R4 without explicit verification of SLS**[A)]	**R4 with explicit verification of SLS**[A)]
Base	γ_b	1.0	1.7	1.5
Shaft (compression)	γ_s	1.0	1.5	1.3
Total/combined (compression)	γ_t	1.0	1.7	1.5
Shaft in tension	$\gamma_{s;t}$	1.0	2.0	1.7

A) The lower γ-values in R4 may be adopted (a) if serviceability is verified by load tests (preliminary and/or working) carried out on more than 1% of the constructed piles to loads not less than 1.5 times the representative load for which they are designed, or (b) if settlement is explicitly predicted by a means no less reliable than in (a), or (c) if settlement at the serviceability limit state is of no concern.

Table A.NA.7 — Partial resistance factors (γ_R) for bored piles for the STR and GEO limit states

Resistance	Symbol	Set		
		R1	**R4 without explicit verification of SLS**[A)]	**R4 with explicit verification of SLS**[A)]
Base	γ_b	1.0	2.0	1.7
Shaft (compression)	γ_s	1.0	1.6	1.4
Total/combined (compression)	γ_t	1.0	2.0	1.7
Shaft in tension	$\gamma_{s;t}$	1.0	2.0	1.7

A) The lower γ-values in R4 may be adopted (a) if serviceability is verified by load tests (preliminary and/or working) carried out on more than 1% of the constructed piles to loads not less than 1.5 times the representative load for which they are designed, or (b) if settlement is explicitly predicted by a means no less reliable than in (a), or (c) if settlement at the serviceability limit state is of no concern.

Table A.NA.8 — Partial resistance factors (γ_R) for continuous flight auger CFA piles for the STR and GEO limit states

Resistance	Symbol	Set		
		R1	**R4 without explicit verification of SLS**[A)]	**R4 with explicit verification of SLS**[A)]
Base	γ_b	1.0	2.0	1.7
Shaft (compression)	γ_s	1.0	1.6	1.4
Total/combined (compression)	γ_t	1.0	2.0	1.7
Shaft in tension	$\gamma_{s;t}$	1.0	2.0	1.7

A) The lower γ-values in R4 may be adopted (a) if serviceability is verified by load tests (preliminary and/or working) carried out on more than 1% of the constructed piles to loads not less than 1.5 times the representative load for which they are designed, or (b) if settlement is explicitly predicted by a means no less reliable than in (a), or (c) if settlement at the serviceability limit state is of no concern.

A.3.3.3 Correlation factors for pile foundations

For the verifications of Structural (STR) and Geotechnical (GEO) limit states, the following correlation factors ξ should be applied to derive the characteristic resistance of axially loaded piles:

ξ_1 on the mean values of the measured resistances in static load tests;

ξ_2 on the minimum value of the measured resistances in static load tests;

ξ_3 on the mean values of the calculated resistances from ground test results;

ξ_4 on the minimum value of the calculated resistances from ground test results;

ξ_5 on the mean values of the measured resistances in dynamic load tests;

ξ_6 on the minimum value of the measured resistances in dynamic load tests.

Table A.NA.9, Table A.NA.10 and Table A.NA.11 give the correlation factor values.

Table A.NA.9 — Correlation factors (ξ) to derive characteristic values of the resistance of axially loaded piles from static pile load tests (n – number of tested piles)

ξ for n =	**1**	**2**	**3**	**4**	**$\geqslant$ 5**
ξ_1	1.55	1.47	1.42	1.38	1.35
ξ_2	1.55	1.35	1.23	1.15	1.08

NOTE For structures having sufficient stiffness and strength to transfer loads from "weak" to "strong" piles, values of ξ_1 and ξ_2 may be divided by 1.1, provided that ξ_1 is never less than 1.0, see EN 1997-1 ***7.6.2.2****(9).*

Table A.NA.10 — Correlation factors (ξ) to derive characteristic values of the resistance of axially loaded piles from ground test results (n – the number of profiles of tests)

ξ for n =	**1**	**2**	**3**	**4**	**5**	**7**	**10**
ξ_3	1.55	1.47	1.42	1.38	1.36	1.33	1.30
ξ_4	1.55	1.39	1.33	1.29	1.26	1.20	1.15

NOTE For structures having sufficient stiffness and strength to transfer loads from "weak" to "strong" piles, values of ξ_3 and ξ_4 may be divided by 1.1, provided that ξ_3 is never less than 1.0, see EN 1997-1 ***7.6.2.3****(7).*

Table A.NA.11 — Correlation factors (ξ) to derive characteristic values of the resistance of axially loaded piles from dynamic impact tests (where n is the number of tested piles)

ξ **for** n =	⩾ **2**	⩾ **5**	⩾ **10**	⩾ **15**	⩾ **20**
ξ_5	1.94	1.85	1.83	1.82	1.81
ξ_6	1.90	1.76	1.70	1.67	1.66

NOTE 1 The ξ-values may be multiplied with a model factor of 0.85 when using dynamic impact tests with signal matching.

NOTE 2 The ξ-values should be multiplied with a model factor of 1.10 when using a pile driving formula with measurement of the quasi-elastic pile head displacement during the impact.

NOTE 3 The ξ-values should be multiplied with a model factor of 1.20 when using a pile driving formula without measurement of the quasi-elastic pile head displacement during the impact.

NOTE 4 If different piles exist in the foundation, groups of similar piles should be considered separately when selecting the number n of test piles.

A.3.3.4 Partial resistance factors (γ_R) for pre-stressed anchorages

As noted under Section 8 of EC7, design of ground anchors to EC7 is not recommended at the present stage of development.

For pre-stressed anchorages and verifications of the structural (STR) and geotechnical (GEO) limit states, the partial factors to be applied on resistance (γ_R) should be as given in Table A.NA.12.

Table A.NA.12 — Partial resistance factors for pre-stressed anchorages at the STR and GEO limit states

Resistance	**Symbol**	**Set**	
		R1	**R4**
Temporary	$\gamma_{a;t}$	1.1	1.1
Permanent	$\gamma_{a;p}$	1.1	1.1

*NOTE Larger values of $_R$ should be used for non-prestressed anchorages, to make their designs consistent with those of tension piles (**A.3.3.2** and **A.3.3.3**) or retaining structures (**A.3.3.5**), as appropriate.*

A.3.3.5 Partial resistance factors (γ_R) for retaining structures

For retaining structures and verifications of the structural (STR) and geotechnical (GEO) limit states, the partial factors to be applied on resistance (γ_R) should be as given in Table A.NA.13.

Because Design Approach 1 is used, all these resistance factors have the value 1.0.

Table A.NA.13 — Partial resistance factors for retaining structures at the STR and GEO limit states

Resistance	Symbol	Set R1
Bearing capacity	$\gamma_{R;v}$	1.0
Sliding resistance	$\gamma_{R;h}$	1.0
Earth resistance	$\gamma_{R;e}$	1.0

A.3.3.6 Partial resistance factors (γ_R) for slopes and overall stability

For slopes and overall stability verifications of the structural (STR) and geotechnical (GEO) limit states, the partial factors to be applied on ground resistance ($\gamma_{R;e}$) should be as given in Table A.NA.14.

Table A.NA.14 — Partial resistance factors for slopes and overall stability at the STR and GEO limit states

Resistance	Symbol	Set R1
Earth resistance	$\gamma_{R;e}$	1.0

A.4 Partial Factors for the uplift limit state (UPL) verification

The UPL limit state is not included in this book.

A.4.1 Partial factors on actions (γ_F)

For the verification of the uplift limit state (UPL) the values for the partial factors on actions (γ_F) should be as given in Table A.NA.15.

Table A.NA.15 — Partial factors on actions (γ_F) at the UPL limit states

Action	Symbol	Value
Permanent		
Unfavourable[A)]	$\gamma_{G;dst}$	1.1
Favourable[B)]	$\gamma_{G;stb}$	0.9
Variable		
Unfavourable[A)]	$\gamma_{Q;dst}$	1.5
Favourable[B)]	$\gamma_{Q;stb}$	0

A) Destabilizing

B) Stabilizing

*NOTE The partial factor specified for permanent unfavourable actions does not account for uncertainty in the level of ground water or free water. In cases where the verification of the UPL limit state is sensitive to the level of ground water or free water, the design value of uplift due to water pressure may be directly assessed in accordance with **2.4.6.1**(2)P and **2.4.6.1**(6)P of BS EN 1997-1:2004. Alternatively, a safety margin may be applied to the characteristic water level, see **2.4.6.1**(8) of BS EN 1997-1:2004.*

A.4.2 Partial factors on soil parameters (γ_M) and resistances (γ_R)

For the verification of the uplift limit state (UPL), the partial factors on soil parameters should be as given in Table A.NA.16.

Table A.NA.16 — Partial factors for soil parameters (γ_M) and resistances (γ_R) at the uplift (UPL) limit state

Soil parameter	Symbol	Value
Angle of shearing resistance[A)]	$\gamma_{\phi'}$	1.25
Effective cohesion	$\gamma_{c'}$	1.25
Undrained shear strength	γ_{cu}	1.4

Resistance	Symbol	Value
Tensile pile resistance	$\gamma_{s;t}$	See Note 2
Anchorage	γ_a	1.4 [B)]

A) Applied to tan φ' and tan φ'_{cv}, although it might be more appropriate to determine the design value of φ'_{cv} directly.

B) Larger values of γ_R should be used for non-prestressed anchorages, to make their designs consistent with those of tension piles (**A.3.3.2** and **A.3.3.3**) or retaining structures (**A.3.3.5**), as appropriate.

*NOTE 1 The value of the partial factor for soil parameters should be taken as the reciprocal of the specified value if such a reciprocal value produces a more onerous effect than the specified value (but see also the Note to **2.4.2**(9)P in BS EN 1997-1:2004).*

*NOTE 2 Pile design should comply with clauses **A.3.3.2** and **A.3.3.3**.*

A.5 Partial Factors for actions for the Hydraulic Heave limit state (HYD) verification

The HYD limit state is not included in this book. It should be noted, however, that there appears to be a printing error in Table A.NA.17: the factor 1.335 for permanent unfavourable actions should read 1.35.

For the verification of the Hydraulic Heave limit state (HYD) the partial factors on actions (γ_F) are as given in Table A.NA.17.

Table A.NA.17 — Partial factors on actions (γ_F) at the Hydraulic Heave (HYD) limit state

Action	Symbol	Value
Permanent		
Unfavourable[A)]	$\gamma_{G;dst}$	1.335
Favourable[B)]	$\gamma_{G;stb}$	0.9
Variable		
Unfavourable[A)]	$\gamma_{Q;dst}$	1.5
Favourable[B)]	$\gamma_{Q;stb}$	0

[A)] Destabilizing
[B)] Stabilizing
*NOTE In applying the specified partial factors in Equation (2.9a) of BS EN 1997-1:2004, the hydrostatic component of the destabilizing total pore water pressure ($u_{dst;d}$) and the stabilizing total vertical stress ($\sigma_{stb;d}$) can be considered to arise from a single source, see Note to **2.4.2**(9)P in BS EN 1997-1:2004.*

A.6 Model Factors

A.6.1 BS EN 1997-1:2004, **2.4.7.1**(6) states that model factors may be applied to the design value of a resistance or the effect of an action to ensure that the results of the design calculation model are either accurate or err on the safe side.

A.6.2 For buildings designed using conventional calculation methods, it can be assumed that the necessary model factors are incorporated in the partial factors given in this Appendix except as specified in **A.6.5** to **A.6.6**.

A.6.3 For bridges and other structures subject to highway loading, an additional model factor may be introduced for the evaluation of the earth pressure coefficient *K*, see PD 6694-1.

A.6.4 Additionally, where the method of analysis of a building or a bridge is innovative, or where the results of a calculation are of uncertain reliability, model factors may be applied. In such cases the values should be agreed with the client and relevant authorities. In selecting the values of a model factor, the principles described in BS EN 1997-1:2004, **2.4.1**(8) and **2.4.1**(9) should be applied.

A.6.5 Model factors required in pile design are provided in **A.3.3.2** and **A.3.3.3**.

A.6.6 BS EN 1997-1:2004, **8.6**(4) requires a model factor to be applied to the SLS value of an anchorage force to ensure that the resistance of the anchorage is sufficiently safe. The meaning of this paragraph is being questioned with the Eurocode 7 Maintenance Group. Until clarification is received, no value for the model factor is recommended by this National Annex.

Appendix 1 Figures

Figure X1 – Summary of the factors required for design of buildings by the UKNA for ULS calculations for STR and GEO

			Combination 1			Combination 2			Combination 2 – piles and anchors			
			A1	M1	R1	A2	M2	R1	A2	M1 or …	M2	R4
Actions	Permanent	unfav	1,35									
		fav										
	Variable	unfav	1,5			1,3			1,3			
Soil	tan ϕ'						1,25				1,25	
	Effective cohesion						1,25				1,25	
	Undrained strength						1,4				1,4	
	Unconfined strength						1,4				1,4	
	Weight density											
Spread footings	Bearing											
	Sliding											
Driven piles	Base											1,7/1.5
	Shaft (compression)											1.5/1.3
	Total/combined											1.7/1.5
	Shaft in tension											2.0/1.7
Bored piles	Base											2.0/1.7
	Shaft (compression)											1.6/1.4
	Total/combined											2.0/1.7
	Shaft in tension											2.0/1.7
CFA piles	Base											As
	Shaft (compression)											for
	Total/combined											bored
	Shaft in tension											piles
Anchors	Temporary											1,1
	Permanent											1,1
Retaining walls	Bearing capacity											
	Sliding resistance											
	Earth resistance											
Slopes	Earth resistance											

indicates partial factor = 1.0

Figure X2 – Some ULS failure modes for spread foundations
(a) overall stability; (b) bearing failure; (c) sliding; (d) combined failure in soil and structure

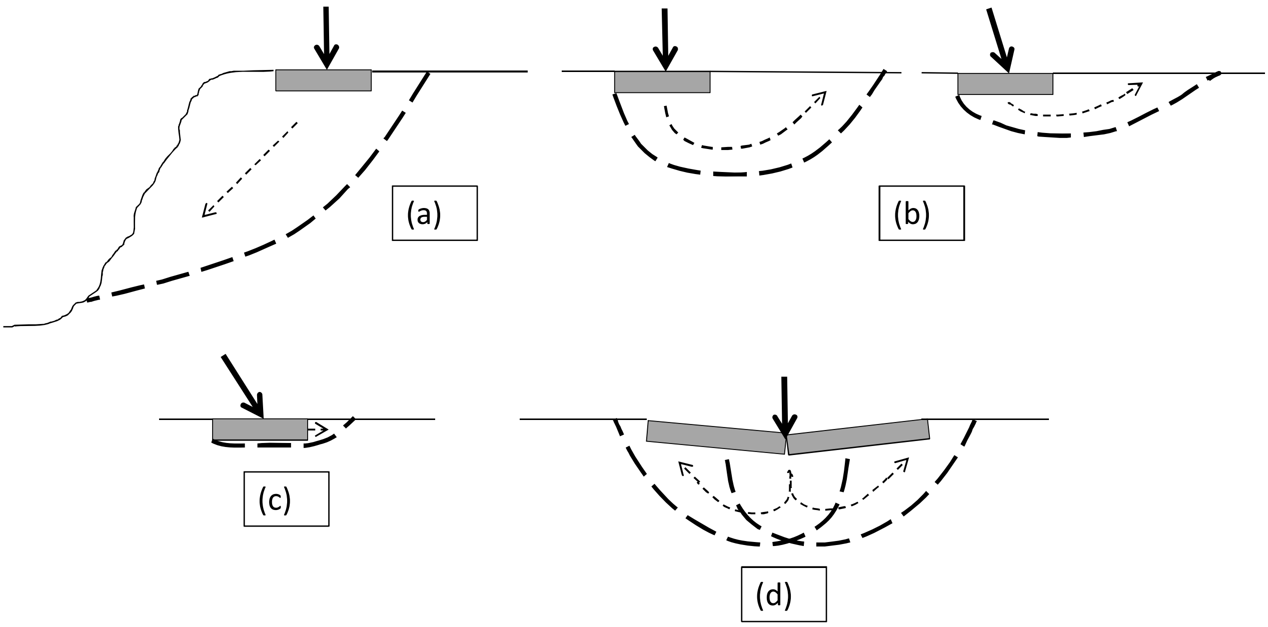

Figure X3 – Linear distributions of bearing pressure used for structural design of spread foundations
(a) central load; (b) mildly eccentric load; (c) strongly eccentric load

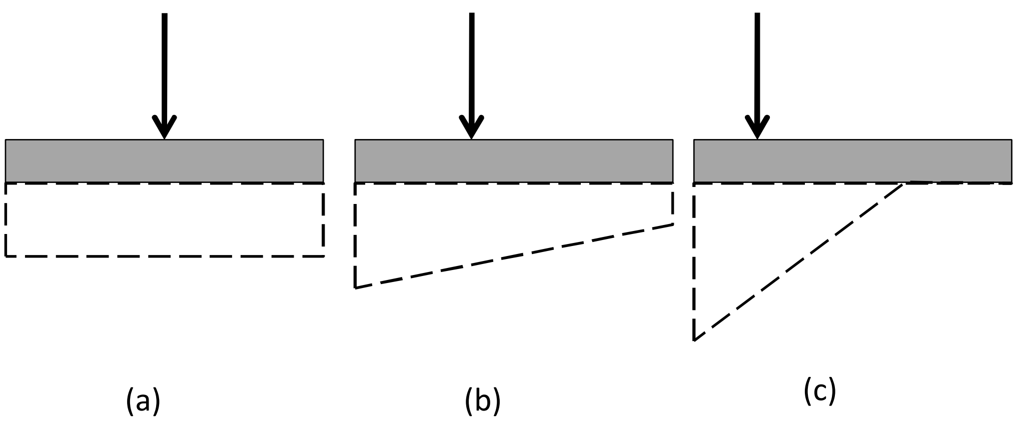

Figure X4 – The 10% "over-excavation" safety margin

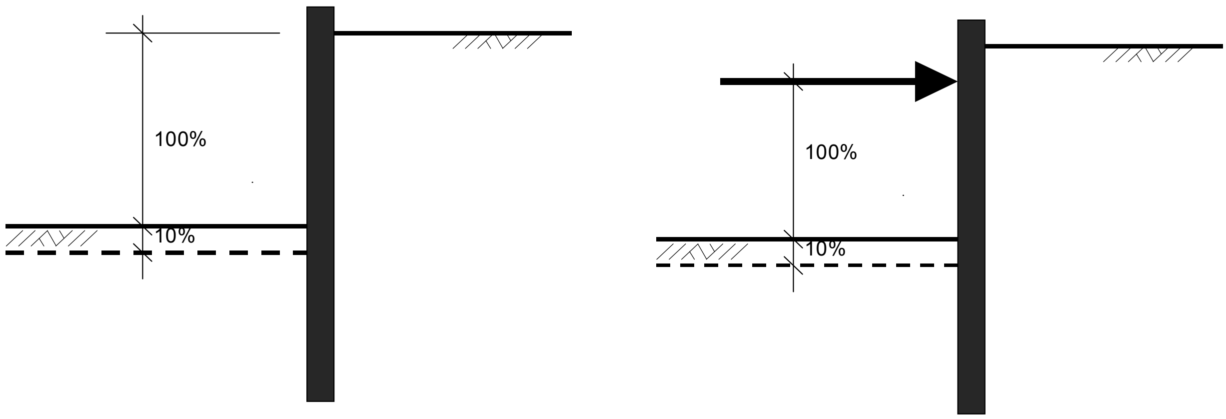

Figure X5 – Bearing capacity factors derived from D.4

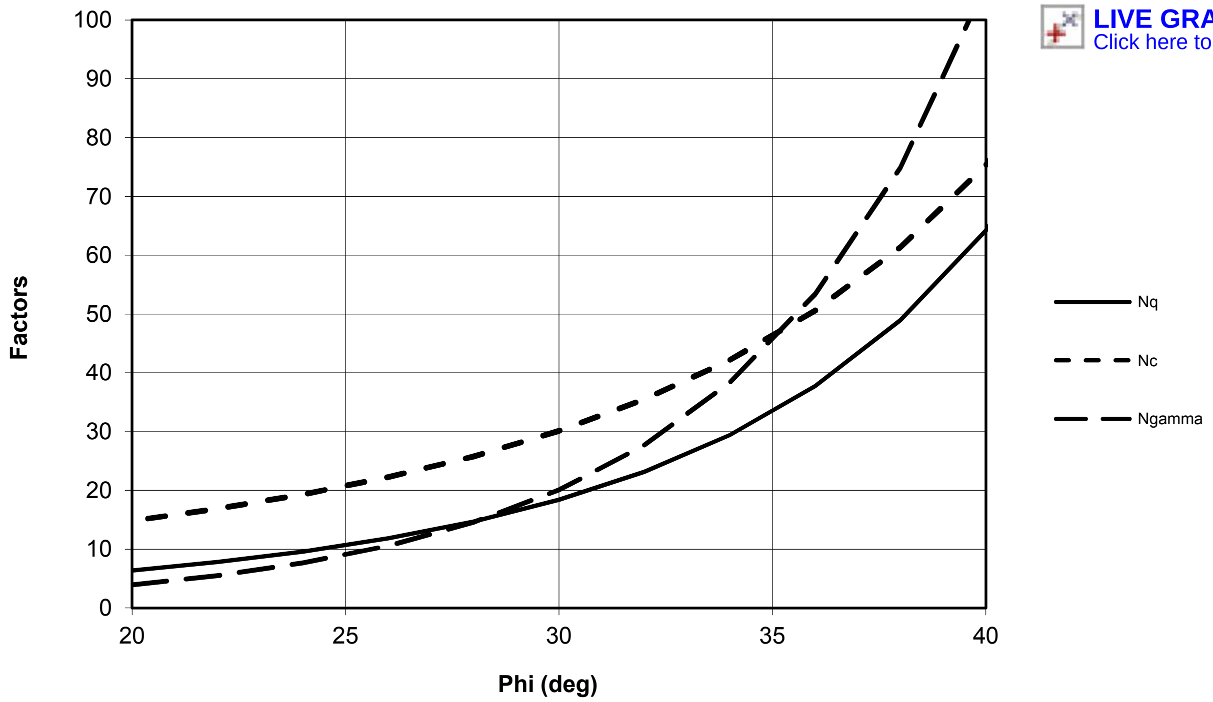

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Figure X6 – Inclination factors for strip footings derived from D.4

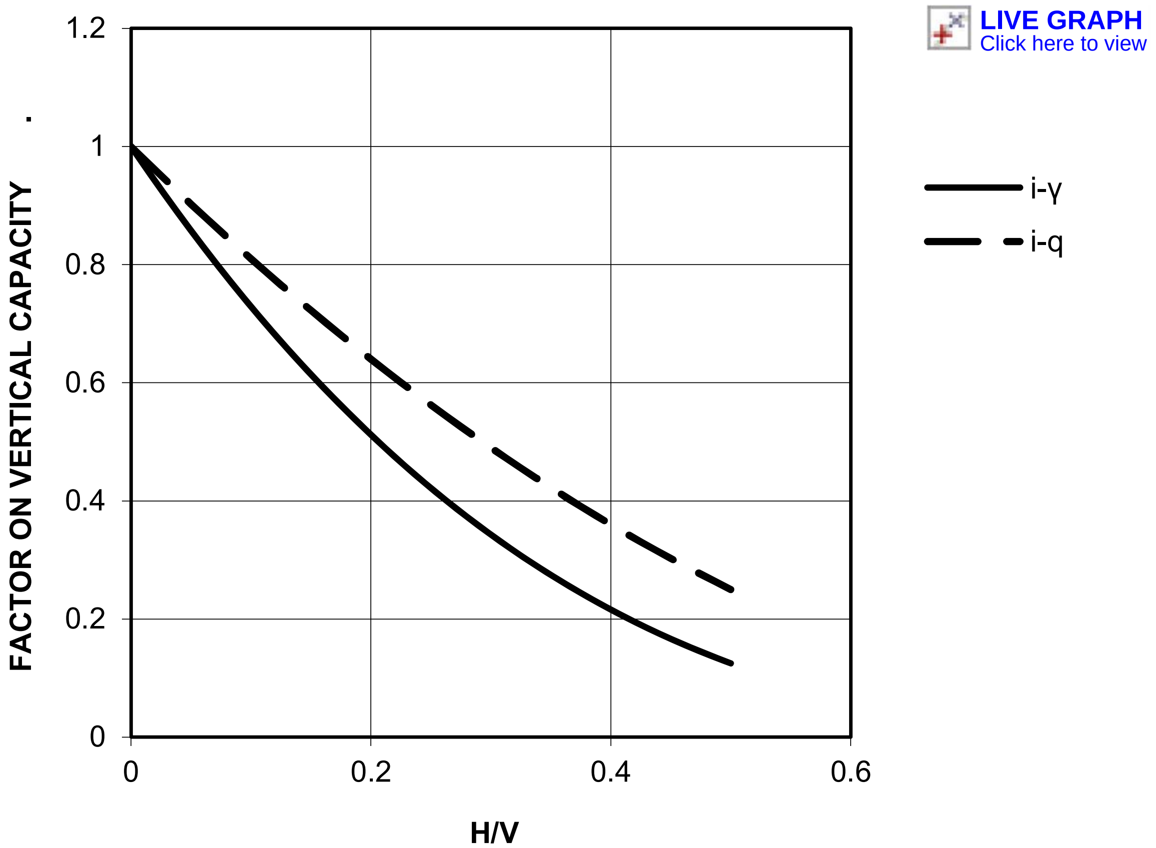

Figure X7 – EN 1990 UKNA 2.2.3.1

NA.2.2.3.1 *Values for the symbols γ of Table A1.2 (A)*

Table NA.A1.2 (A) provides the values for the symbols γ of Table A1.2 (A). The values chosen are:

$\gamma_{Gj,sup} = 1,10$

$\gamma_{Gj,inf} = 0,90$

$\gamma_{Q,1} = 1,50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1,50$ where unfavourable (0 where favourable)

NOTE For Ψ values see Table A1.1 (BS).

Table NA.A1.2 (A) — Design values of actions (EQU) (Set A)

Persistent and transient design situations	Permanent actions		Leading variable action[a]	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	1,10 $G_{kj,sup}$	0,90 $G_{kj,inf}$	1,5 $Q_{k,1}$ (0 when favourable)		$1,5\Psi_{0,i}Q_{k,i}$ (0 when favourable)

[a] Variable actions are those considered in Table NA.A1.1.

In cases where the verification of static equilibrium also involves the resistance of structural members, as an alternative to two separate verifications based on Tables NA.A1.2 (A) and A1.2 (B), a combined verification, based on Table NA.A1.2 (A), should be adopted, with the following set of values.

$\gamma_{Gj,sup} = 1,35$

$\gamma_{Gj,inf} = 1,15$

$\gamma_{Q,1} = 1,50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1,50$ where unfavourable (0 where favourable)

provided that applying $\gamma_{Gj,inf} = 1,00$ both to the favourable part and to the unfavourable part of permanent actions does not give a more unfavourable effect.

Figure X8 – EN 1990 UKNA 2.2.3.2

NA.2.2.3.2 *Values for the symbols γ and ξ of Table A1.2 (B)*

Table NA.A1.2 (B) provides the values for the symbols γ and ξ of Table A1.2 (B). The values chosen are:

$\gamma_{Gj,sup} = 1,35$

$\gamma_{Gj,inf} = 1,00$

$\gamma_{Q,1} = 1,50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1,50$ where unfavourable (0 where favourable)

$\xi = 0,925$

NOTE For Ψ values see Table NA.A1.1.

Table NA.A1.2 (B) — Design values of actions (STR/GEO) (Set B)

Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions[a]		Persistent and transient design situations	Permanent actions		Leading variable action[a]	Accompanying variable actions[a]	
	Unfavourable	Favourable		Main (if any)	Others		Unfavourable	Favourable	Action	Main	Others
(Eq. 6.10)	$1,35G_{kj,sup}$	$1,00G_{kj,inf}$	$1,5Q_{k,1}$		$1,5\psi_{0,1}Q_{k,i}$	(Eq. 6.10a)	$1,35G_{kj,sup}$	$1,00G_{kj,inf}$		$1,5\psi_{0,1}Q_{k,1}$	$1,5\psi_{0,1}Q_{k,i}$
						(Eq. 6.10b)	$0,925*1,35G_{kj,sup}$	$1,00G_{kj,inf}$	$1,5Q_{k,1}$		$1,5\psi_{0,1}Q_{k,i}$

NOTE 1 Either expression 6.10, or expression 6.10a together with and 6.10b may be made, as desired.

NOTE 2 The characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,sup}$ if the total resulting action effect is unfavourable and $\gamma_{G,inf}$ if the total resulting action effect is favourable. For example, all actions originating from the self weight of the structure may be considered as coming from one source; this also applies if different materials are involved.

NOTE 3 For particular verifications, the values for γ_G and γ_Q may be subdivided into γ_g and γ_q and the model uncertainty factor γ_{sd}. A value of γ_{sd} in the range 1,05 to 1,15 can be used in most common cases and can be modified in the National Annex.

NOTE 4 When variable actions are favourable Q_k should be taken as 0.

[a] Variable actions are those considered in Table NA.A1.1.

Figure X9 – EN 1990 UKNA 2.2.3.3

NA.2.2.3.3 *Values for the symbols γ of Table A1.2 (C)*

Table NA.A1.2 (C) provides the values for the symbols γ of Table A1.2 (C). The values chosen are:

$\gamma_{Gj,sup} = 1{,}00$

$\gamma_{Gj,inf} = 1{,}00$

$\gamma_{Q,1} = 1{,}30$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1{,}30$ where unfavourable (0 where favourable)

NOTE For Ψ values see Table NA.A1.1.

Table NA.A1.2 (C) — Design values of actions (STR/GEO) (Set C)

Persistent and transient design situation	Permanent actions		Leading variable action[a]	Accompanying variable actions[a]	
	Unfavourable	Favourable		Main (if any)	Others
(Eq 6.10)	$1{,}0\ G_{kj,sup}$	$1{,}0G_{kj,inf}$	$1{,}3Q_{k,1}$ (0 when favourable)		$1{,}3\Psi_{0,i}Q_{k,i}$ (0 when favourable)

[a] Variable actions are those considered in Table NA.A1.1.

Figure X10 – Example in which EQU is relevant

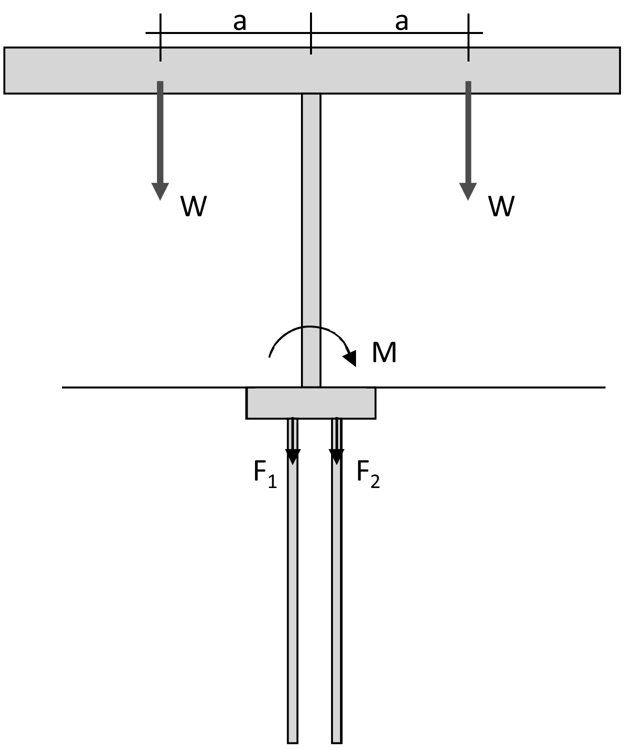

Figure X11 – Derivation of design ultimate shaft and base resistance from characteristic soil strengths

Characteristic soil strengths
($c_{u,k}$, $\tan\phi_k$, etc)

▼

Calculation model – accurate or erring on the side of safety

shaft and base resistance calculated using characteristic values of soil properties by a method complying with EN 1997-1, **2.4.1**(6)

▼

Calculated shaft and base resistance

▼

γ_{Rd} = 1.4 or 1.2

a model factor should be applied to the shaft and base resistance … value … 1.4, except that it may be reduced to 1.2 if the resistance is verified by a maintained load test taken to the calculated, unfactored ultimate resistance.

▼

Characteristic shaft and base resistance

▼

γ_s and γ_b

For partial resistance factors see Tables A.NA.6-8.

▼

Design shaft and base resistance

Figure X12 – Situation in which a model factor might be used in calculation

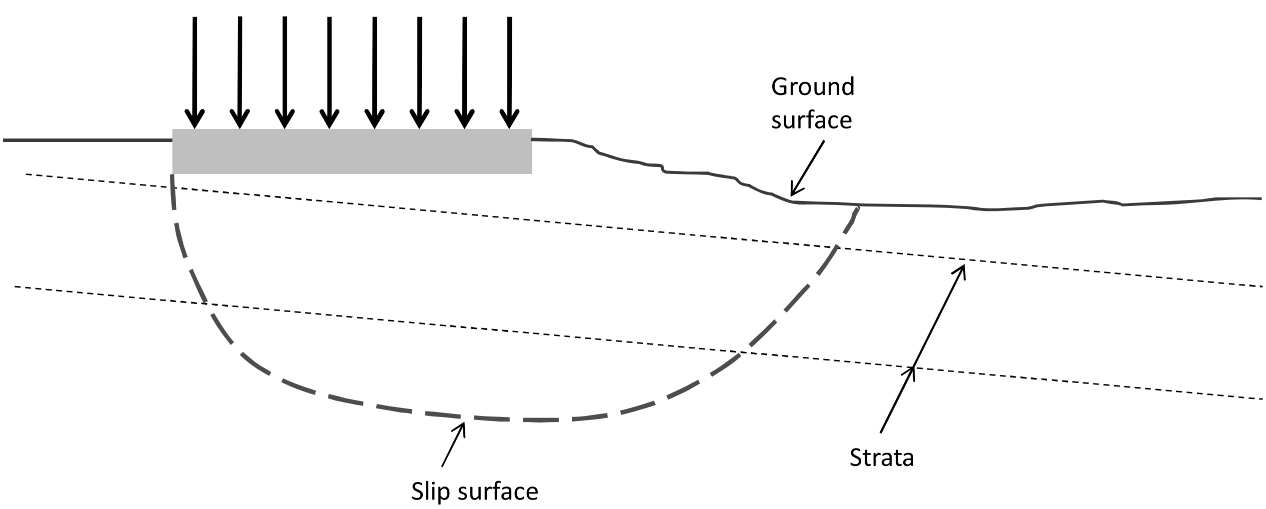

Figure X13 – Failure surfaces in variable ground

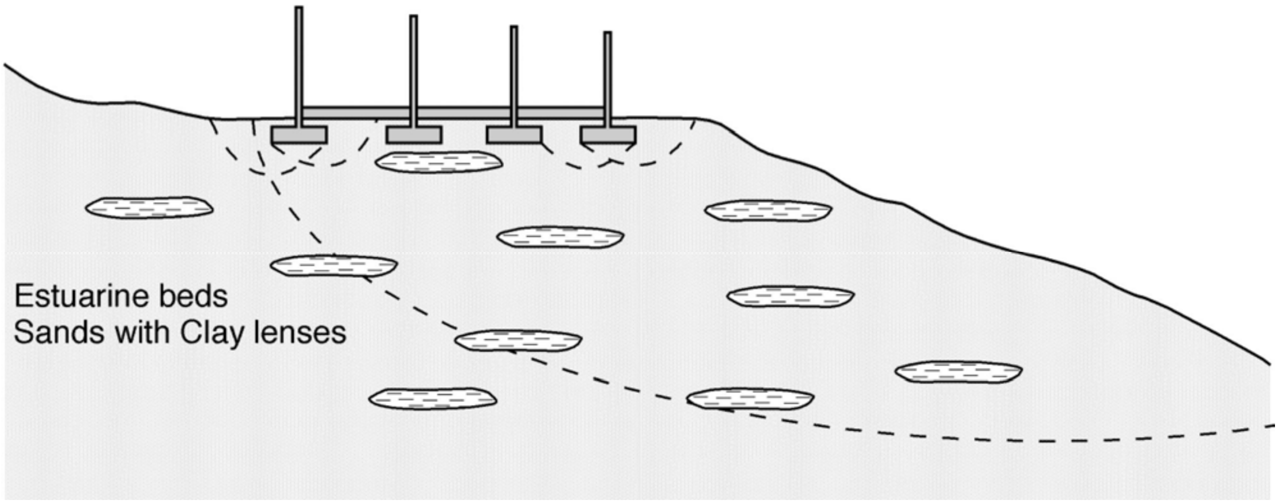

Figure X14 – Non-linear relationships in frictional soil

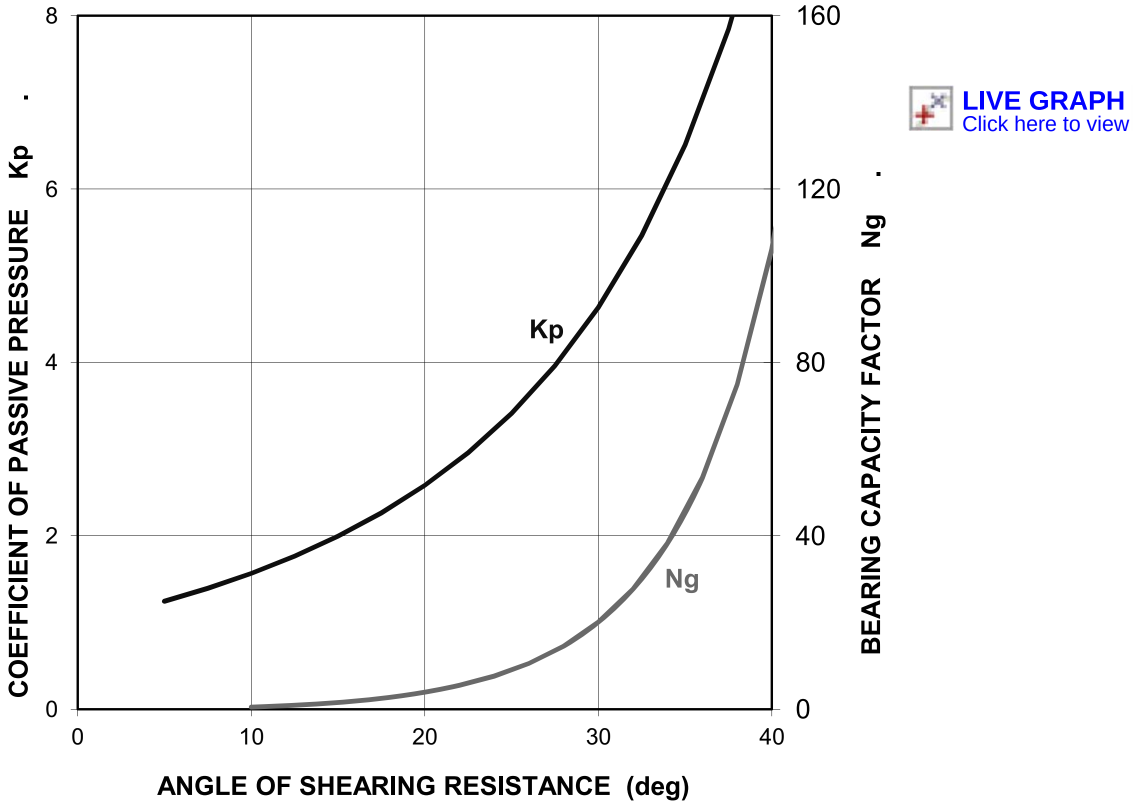

Figure X15 – Coefficient of active pressure computed from EC7 C.2 for δ/φ'=0

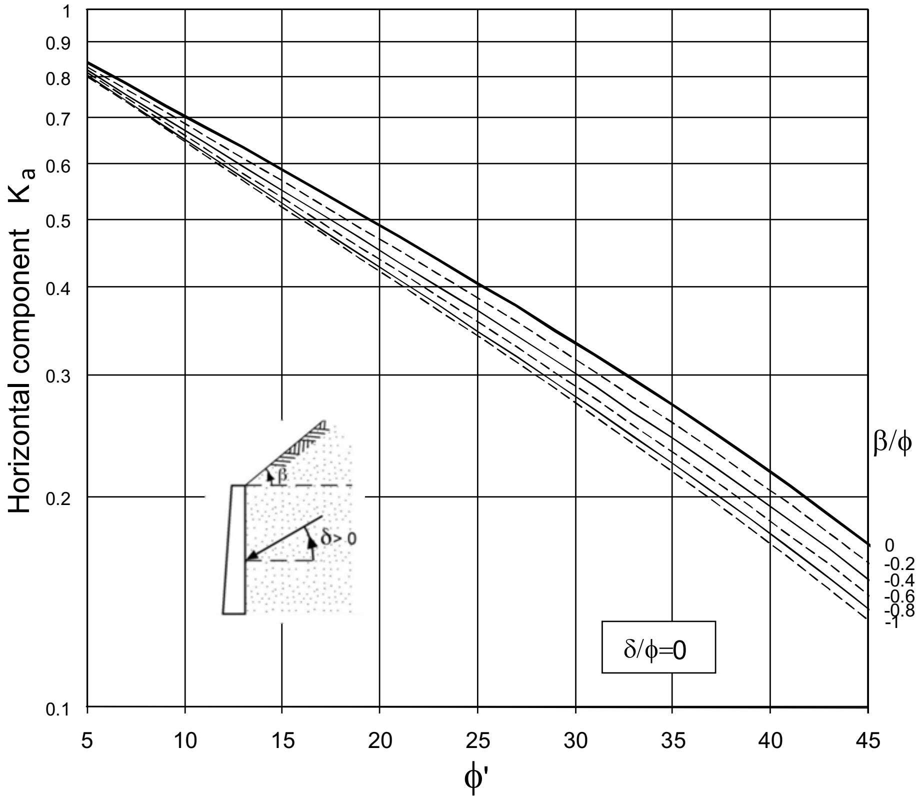

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Figure X16 – Coefficient of active pressure computed from EC7 C.2 for δ/φ'=0.5

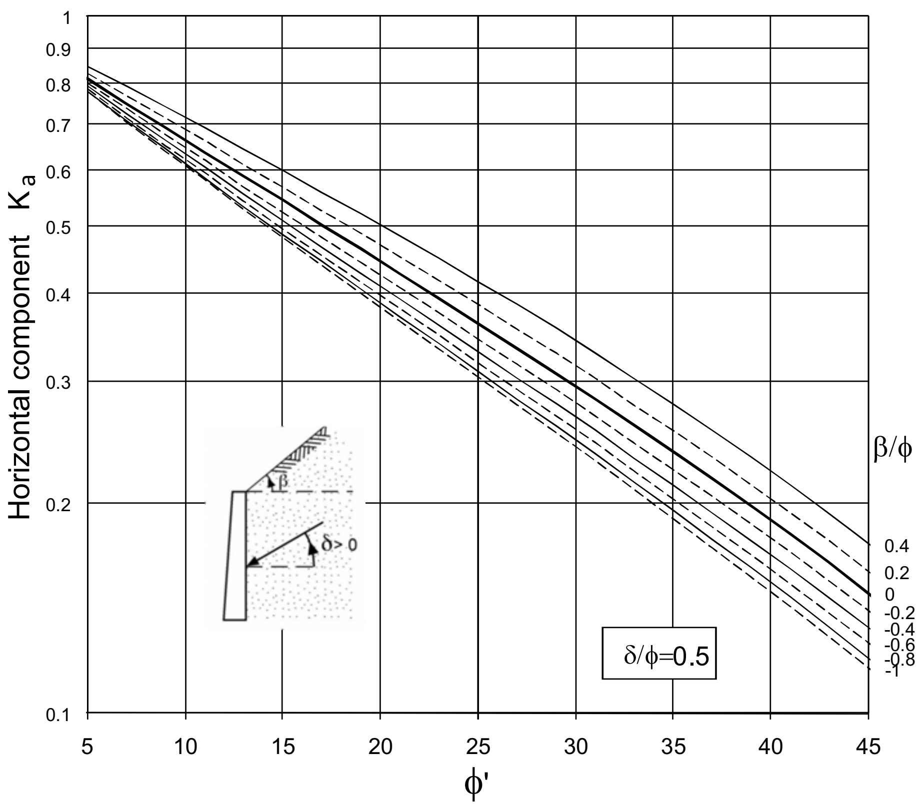

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Figure X17 – Coefficient of active pressure computed from EC7 C.2 for δ/φ'=0.67

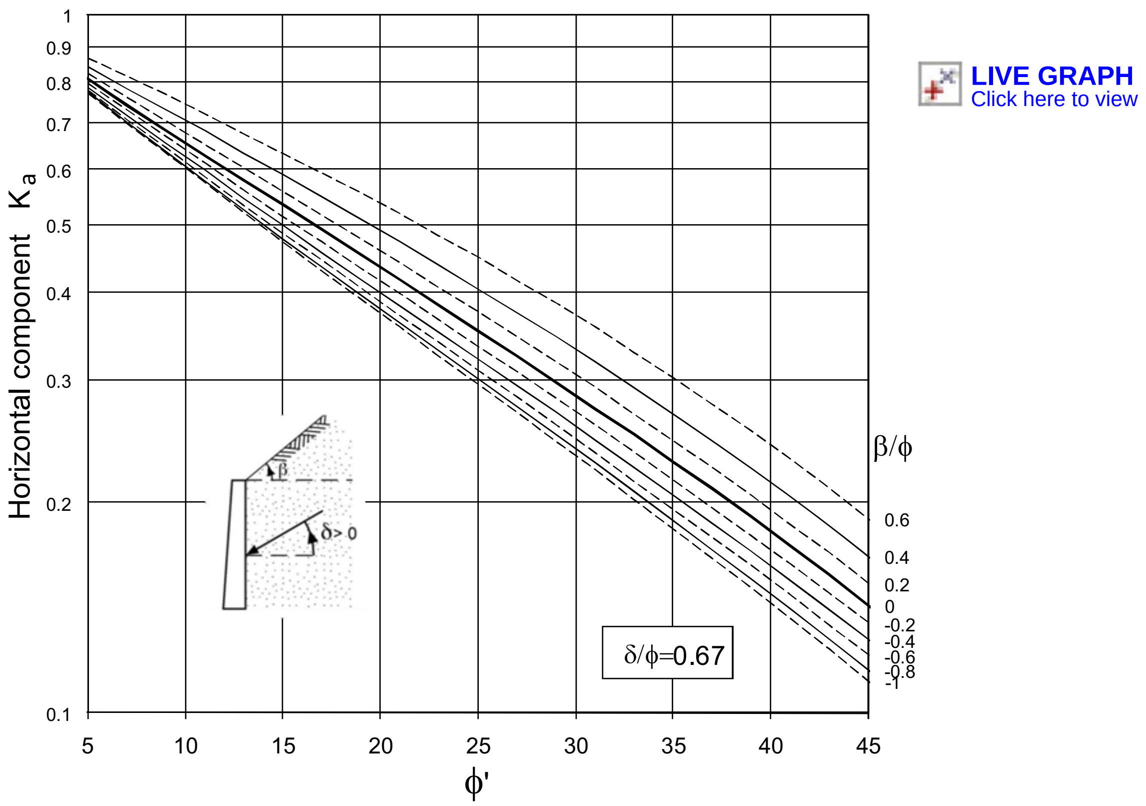


Figure X18 – Coefficient of active pressure computed from EC7 C.2 for δ/φ'=1.0

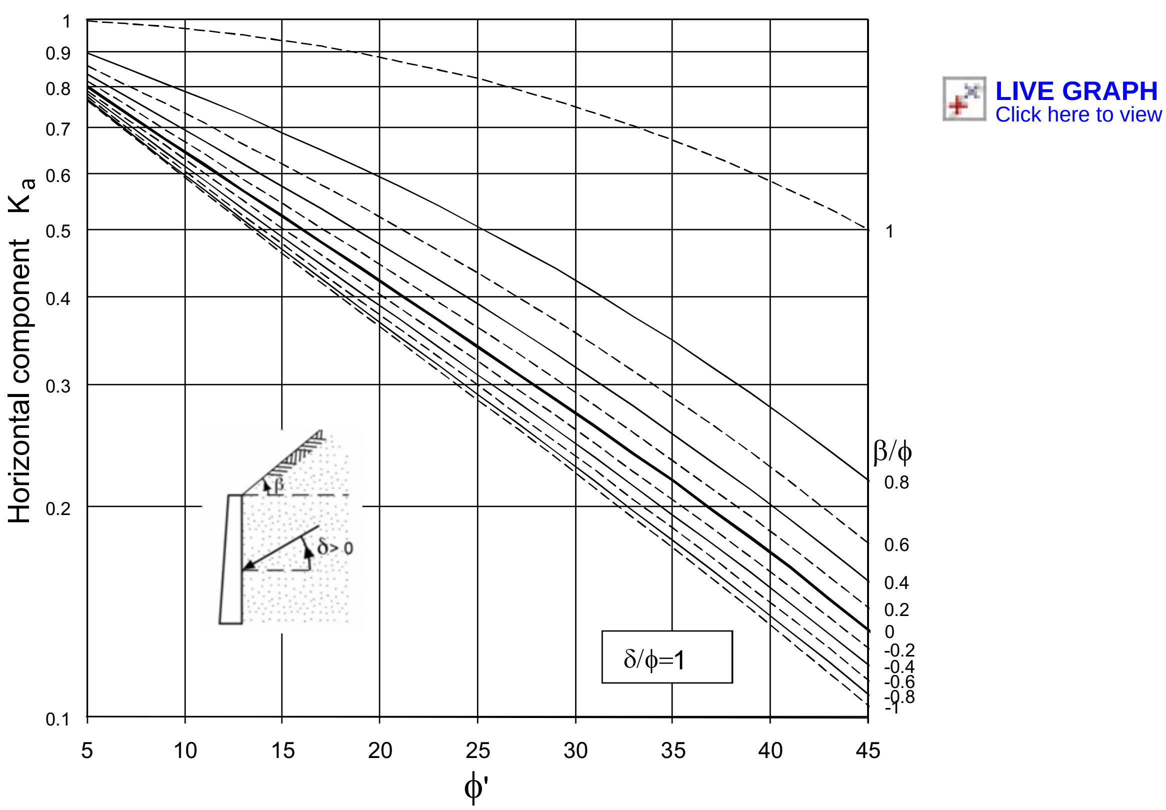
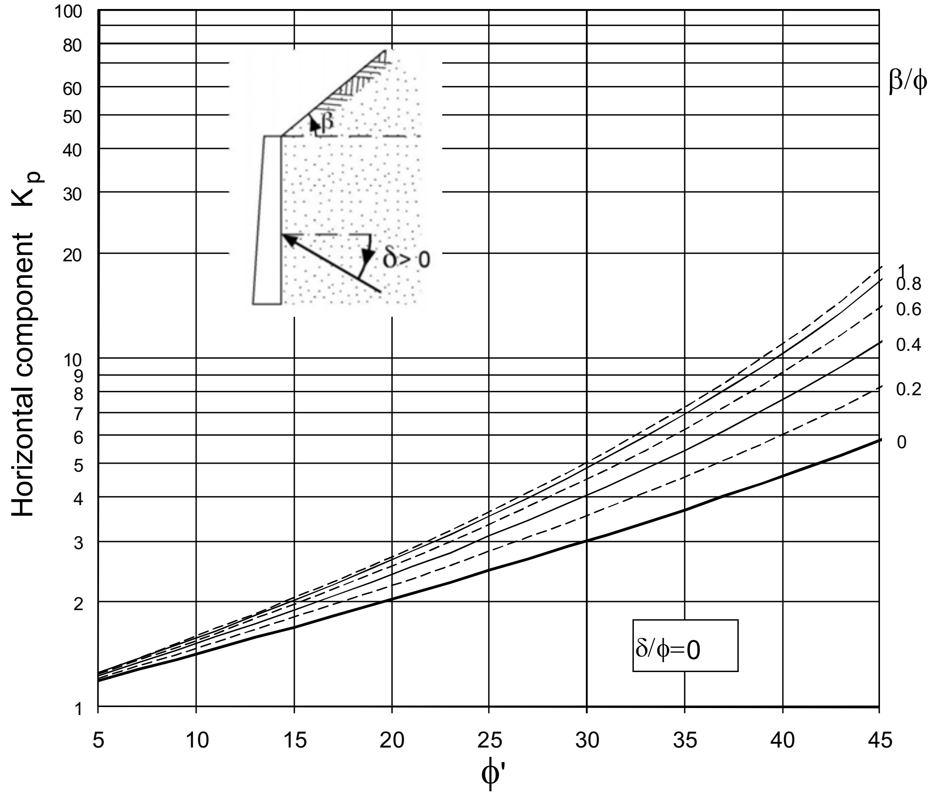
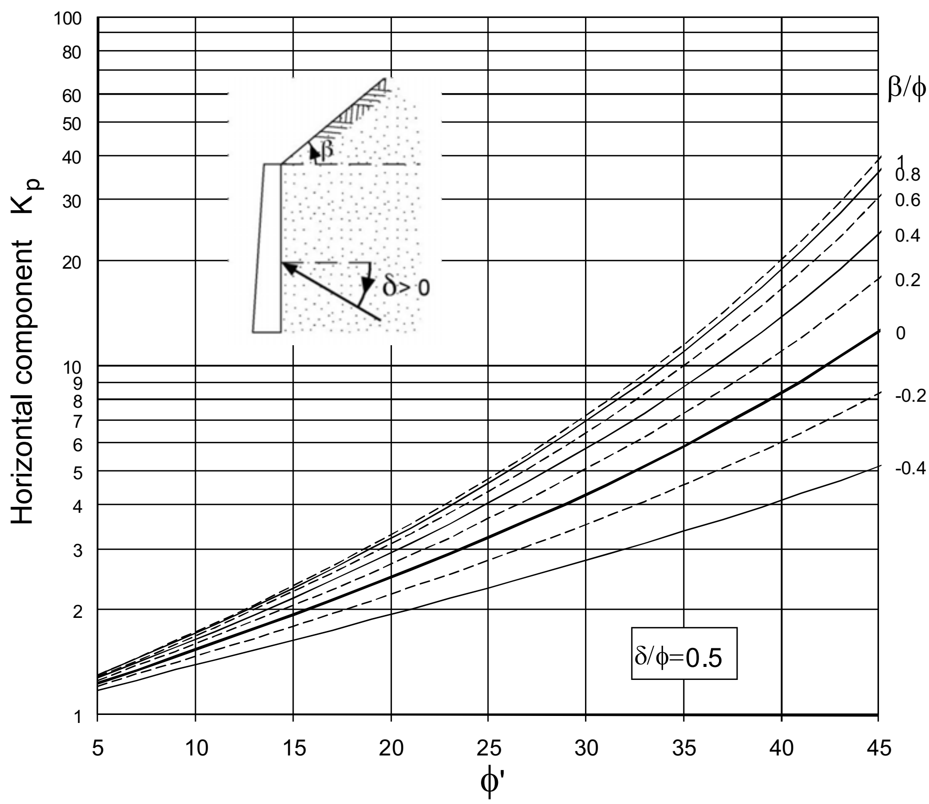


Figure X19 – Coefficient of passive pressure computed from EC7 C.2 for δ/φ'=0



LIVE GRAPH
Click here to view

Figure X20 – Coefficient of passive pressure computed from EC7 C.2 for δ/φ'=0.5



LIVE GRAPH
Click here to view

Figure X21 – Coefficient of passive pressure computed from EC7 C.2 for δ/φ'=0.67

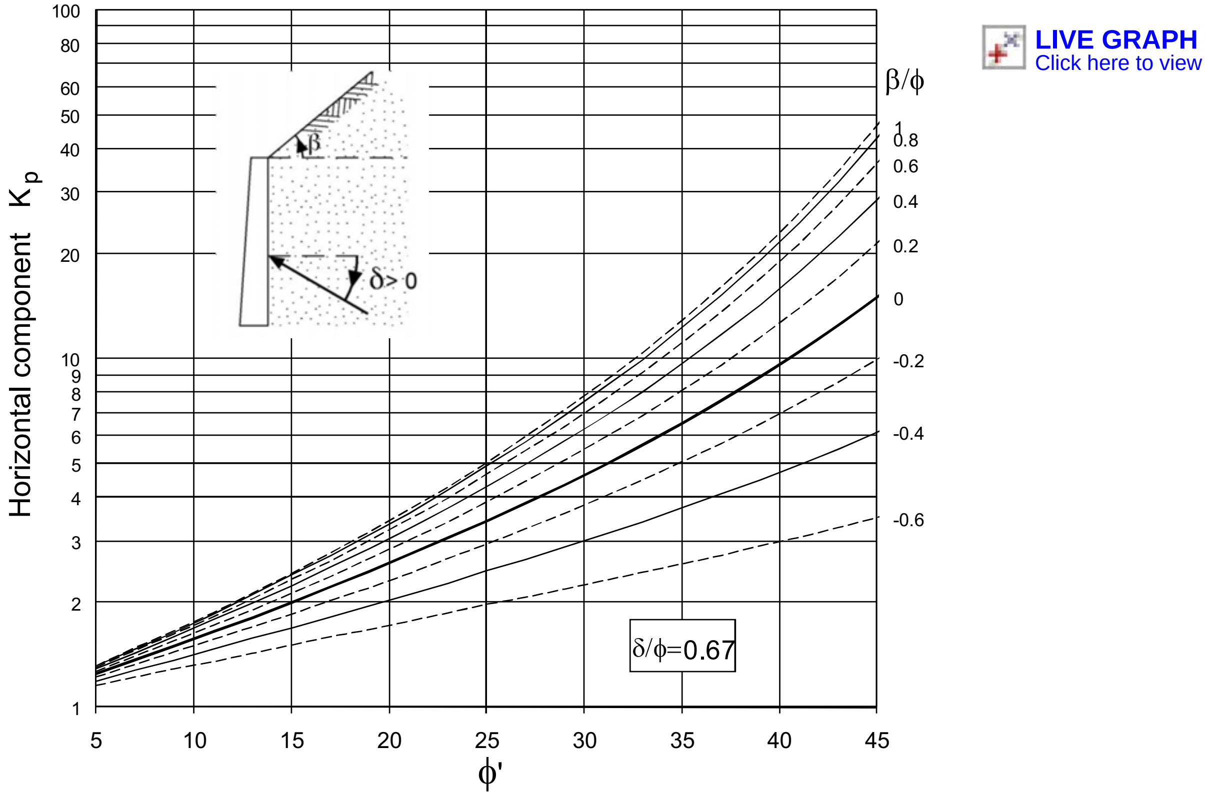
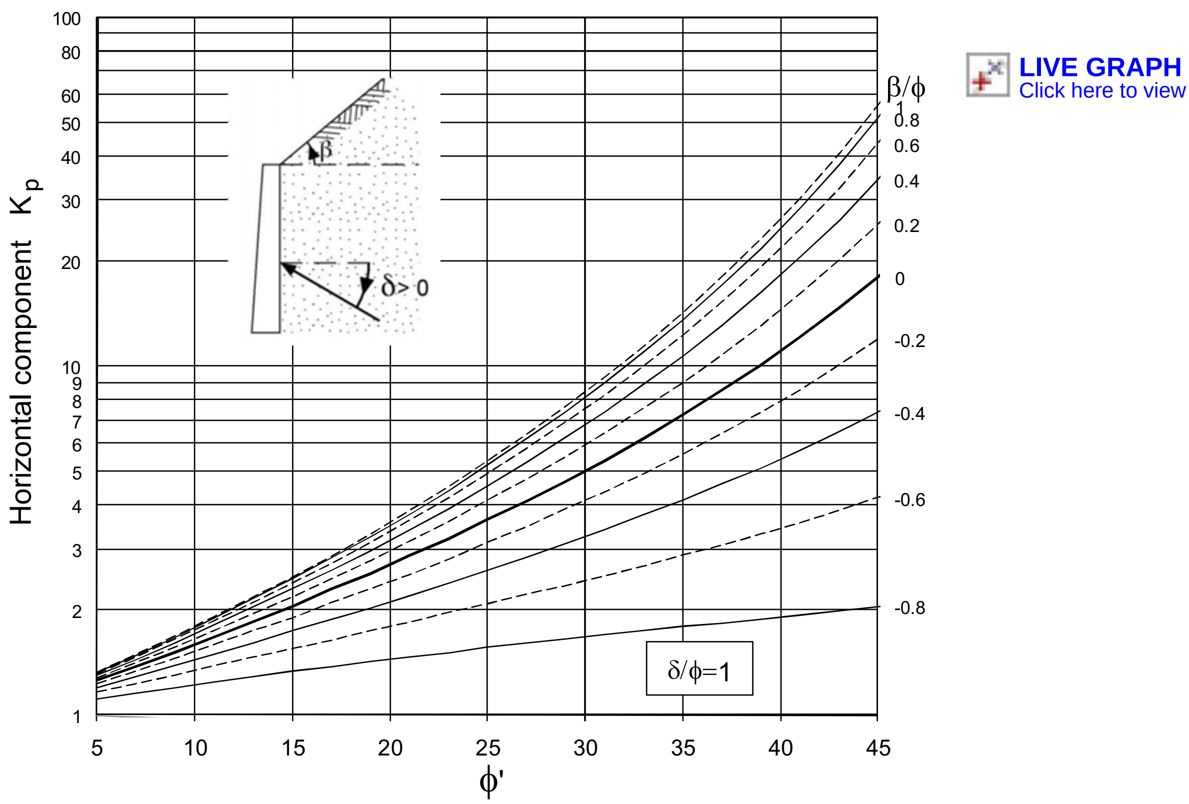


Figure X22 – Coefficient of passive pressure computed from EC7 C.2 for δ/φ'=1.0



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