Manual for the geotechnical design of structures to Eurocode 7

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The Institution of Structural Engineers

Constitution of the Task Group

M J Puller DIC FIStructe FICE *Chairman*
Professor J B Burland* CBE FREng FRS PhD MSc(Eng) DSc(Eng) CEng FIStructE FICE FCGI (Imperial College, London), *Vice-Chairman*
B C Bell MA MSc DIC CEng FIStructE FICE (Bell Johnson Ltd)
Professor S Denton MS PhD CEng MICE (Parsons Brinckerhoff)
M Derewicz CEng MIStructE MICE (Jenkins & Potter)
R McGall (Deep Soil Mixing Ltd)
Dr C Menkiti* BSc MSc PhD DIC (Geotechnical Consulting Group)
P Rutty MSc DIC CEng FIEI MICE (Mott MacDonald Limited)
**representing BGA*

Consultants

Dr P Morrison BA BAI PhD CEng MIEI (Arup Geotechnics)
S Pennington BEng BSurv CEng MICE (Arup Geotechnics)

Secretary to the task group

Dr J D Littler PhD (The Institution of Structural Engineers) (until July 2011)
B Chan BSc(Hons) AMIMechE (The Institution of Structural Engineers) (from July 2011)
L Kirk MEng(Hons) (Technical Assistant, The Institution of Structural Engineers) (from September 2012)

Acknowledgements

P Dauncey MA CEng MICE (Arup Geotechnics)
Dr B Simpson OBE FREng MA PhD FICE (Arup Geotechnics)

Published by the Institution of Structural Engineers
International HQ, 11 Upper Belgrave Street, London SW1X 8BH
Telephone: +44(0)20 7235 4535 Fax: +44(0)20 7235 4294
Email: mail@istructe.org, Website: www.istructe.org
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1 Introduction

1.1 Aims of this *Manual*

This *Manual* forms part of a suite of Eurocode Manuals prepared by the Institution of Structural Engineers. This *Manual* has been specifically prepared with the following aims:
– To provide guidance to structural engineers on geotechnical design for common forms of construction for UK buildings to BS EN 1997-1:2004[1] and BS EN 1997-2:2007[2] and their UK National Annexes. The term ‘buildings’ includes basements and retaining walls. Introductory comments on specialist geotechnical techniques which may be associated with building design and construction are provided.
– To promote interaction and understanding between structural and geotechnical engineers.

This *Manual* is primarily addressed to those carrying out calculations by hand or using standard geotechnical software. It does not address situations where specialist geotechnical knowledge would be required for more complex design and construction procedures. It relates only to the design of buildings to be constructed in the United Kingdom.

1.2 Eurocode system

1.2.1 Background

The origin of the Eurocode system lies with the Construction Products Directive (89/106/CE) produced by the Commission of the European Community (CEC) in 1988. The aim of this directive was to remove trade barriers and specify requirements for construction products within the European Economic Area (EEA).

The European Committee for Standardisation (CEN) which produces European Standards and technical specifications was appointed to prepare a series of integrated European Standards, or Euronorms (ENs) to provide common guidance on the assessment of structures. CEN primarily comprises thirty member nations who work together to produce ENs. Within CEN there are many technical committees (TCs) that write standards. The Eurocodes are a specific set of Euronorms, currently numbered EN 1990 to EN 1999, concerned with the design of buildings and civil engineering structures. The technical committee responsible for the Eurocodes is TC 250. Other relevant technical committees include: TC 288 (Execution Standards) and TC 341 (Ground Properties Standards).

For each member nation within CEN, there is a local standard body. For the UK this body is the British Standards Institution (BSI). It is BSI's responsibility to publish ENs as UK national standards (BS ENs). Eurocodes, unlike other Euronorms, also have National Annexes so while the base document remains the same, BS EN Eurocodes have a title page, foreword and National Annex specific to the UK.

The National Annex (NA) contains information pertinent to the UK. This information includes: UK specific data, nationally determined parameters (NDPs) where the EN allows choice, general guidance specific to the UK and references to non-contradictory complementary information (NCCI).

For the avoidance of doubt, each nation publishing Eurocodes will also publish National Annexes which are an integral part of the national standards.

1.2.2 Suite of Eurocodes

The Eurocodes published in the UK comprise:

BS EN 1990:	Eurocode	Basis of structural design (referred to herein as EC0)
BS EN 1991:	Eurocode 1:	Actions on structures (EC1)
BS EN 1992:	Eurocode 2:	Design of concrete structures (EC2)
BS EN 1993:	Eurocode 3:	Design of steel structures (EC3)
BS EN 1994:	Eurocode 4:	Design of composite steel and concrete structures (EC4)
BS EN 1995:	Eurocode 5:	Design of timber structures (EC5)
BS EN 1996:	Eurocode 6:	Design of masonry structures (EC6)
BS EN 1997:	Eurocode 7:	Geotechnical design (EC7) in parts
	Part 1: General rules (EC7 Part 1)	
	Part 2: Ground investigation and testing (EC7 Part 2)	
BS EN 1998:	Eurocode 8:	Design of structures for earthquake resistance (EC8)
BS EN 1999:	Eurocode 9:	Design of aluminium structures (EC9)

For brevity and simplicity, for the rest of this *Manual* each Eurocode will be referred to by the abbreviation in brackets above.

In addition to EC7, the following standards are relevant to geotechnical structures:

- EC0[3]
- EC1 Part 1-1 – General actions – densities, self-weight, imposed loads for buildings[4]
- EC2 Part 1-1 – General rules and rules for buildings[5]
- EC3 Part 5 – Piling[6] (which includes sheet piling).

1.2.3 Editorial style

All Eurocodes follow a common editorial style; sections are the main divisions and clauses are the next subdivision. Clauses are designated as ‘Principles’ or ‘Application Rules’ while annexes are either normative (usually related to structural material properties and partial factors which may be altered by the national annexes) or informative (presentation of accepted good practice for typical design situations).

Principles are statements, definitions, requirements and analytical models which are mandatory (normative). Application Rules are generally recognised rules that follow the Principles in the Eurocode; they are, however, not normative.

The Eurocodes allow alternatives to Application Rules provided that compliance with the Principles can be demonstrated; however, if alternatives are used the resulting design cannot be claimed to be wholly in accordance with the Eurocodes. As the intention of this *Manual* is to provide guidance on design in accordance with EC7, a distinction between Principles and Application Rules is not made.

1.3 Use of this *Manual*

All designers (assumed to include checkers) must be appropriately qualified, experienced and competent to carry out the work upon which they are embarking. In presenting this *Manual* it is assumed that this test will be foremost in the designer’s mind and that if there is any doubt a suitable second opinion will be obtained. Attention is also drawn to the Institution of Structural Engineers’ *Code of Conduct and Guidance Notes*.[7]

Accepting the above, the following checklist is appropriate when using this *Manual*:

– Geology, ground and groundwater are inherently variable and can be unpredictable even to the most experienced engineers/geologists. This is recognised in EC7 Part 1[1] where three Geotechnical Categories are presented. This *Manual* addresses Geotechnical Categories 1 and 2 only (see Section 2.9).
– The Institution of Structural Engineers is developing this suite of Manuals for common types of buildings in the UK; large and/or unusual structures are not considered herein. In preparing this *Manual* it was envisaged that common types of buildings would be limited to: 10 storeys with regular column layouts and single level temporary propping for excavations.
– Geotechnical conditions and soil-structure interactions are adequately understood.
– Specialist geotechnical techniques are introduced in Chapter 9. However, detailed design is not addressed as this will normally be carried out by geotechnical specialists.

EC7 Part 1[1] presents the following additional assumptions for consideration by designers and clients:
- Data required for design are collected, recorded and interpreted by appropriately qualified personnel.
- Adequate continuity and communication exist between the personnel involved in data collection, design and construction.
- Adequate on site supervision and quality control are provided during all stages of production (e.g. pre-cast piles) and construction (e.g. pile installation).
- Execution is carried out to relevant standards and specifications by personnel with appropriate skill and experience.
- Construction materials and products are used as specified.
- Structures will be adequately maintained for the design service life.
- Structures will be used for the purpose for which they were designed.

Compliance with these assumptions should be documented in the Geotechnical Design Report (GDR) (see Section 3.8).

A number of the limitations above can clearly be interpreted subjectively. To aid interpretation, Figure 1.1 shows those topics that are considered in detail and those that are only mentioned in passing or not at all. A dashed line between the two lists is used to illustrate the fact that this *Manual* mentions many of the 'excluded' topics but does not cover all aspects of the 'included' topics.

1.4 Structure of this *Manual*

EC7 Part 1 and EC7 Part 2 provide guidance on geotechnical design. Their structure is such that ground investigation and assessment of ground conditions are largely addressed in Part 2 while use of the ground parameters and subsequent design is addressed in Part 1.

EC7 Part 1[1] is the main topic of this *Manual*. EC7 Part 2[2] is presented in Chapter 3 wherein an introduction to geotechnical investigation and reporting is provided.

Figure 1.2 shows how the chapter structure adopted in this *Manual* relates to the EC7 section structure.

1.5 Other documents

In addition to the suite of Eurocodes presented above, there are numerous other documents which are relevant to design in the UK involving EC7. These

FOCUS OF THIS *MANUAL*
(*'included' topics*)

- Geotechnical Categories 1 and 2
- Ground Investigation Report
- Geotechnical Design Report
- Overall stability
- Design of conventional foundations (pads, strips and piles)
- Cantilever or single propped excavations where ground movements are not especially critical
- Concept design of specialist geotechnical constructions
- Strength, stability, serviceability and durability
- Characteristic values.

TOPICS BEYOND THE SCOPE OF THIS *MANUAL*
(*'excluded' topics*)

- Geotechnical Category 3
- Large/unusual structures
- Design situations requiring iterative solutions such as piled rafts
- Design of deep multi-propped basements and basements where anticipated ground movements may cause damage to neighbouring structures
- Scheme/detailed design of specialist geotechnical constructions
- Bridges/structures subject to traffic loading
- Design in seismic areas
- Sustainability
- Derived values (geotechnical parameters)
- Fatigue/cyclic loading/dynamic loading
- Specification of pile load tests
- Pile dynamic testing
- Internal erosion of soils
- Semi-empirical/indirect design.

The following are excluded but have short introductory sections:

- Specification of ground investigations
- Dewatering
- Ground improvement
- Reinforced soil structures
- Ground anchors
- Fill/embankments.

Fig 1.1 Illustrative aid to the interpretation of the scope of this *Manual*

documents are produced by organisations including: European Committee for Standardization (CEN), International Organisation for Standardisation (ISO), British Standards Institution (BSI), Institution of Civil Engineers (ICE), Highways Agency (HA), Construction Industry Research and Information Association (CIRIA) and others. A number of the more relevant publications are listed below (current as of October 2012).

Standards prepared by CEN TC 288 (the committee responsible for standardisation in the field of special geotechnical works) include:
BS EN 1536 Bored piles[8]
BS EN 1537 Ground anchors[9]

Chapters in this *Manual*	EC7 Part 1: sections	EC7 Part 2: sections
1 Introduction	1 General	
2 Basis of design	2 Basis of design	
3 Ground investigation and reporting	3 Geotechnical data	1 General 2 Planning of ground investigations 3 Soil and rock sampling and groundwater measurement 4 Field tests in soil and rock 5 Laboratory tests in soil and rock 6 Ground Investigation Report
4 Introduction to design	(2) (Basis of design)	
5 Overall stability and slopes	11 Overall stability	
6 Spread foundations	6 Spread foundations	
7 Pile foundations	7 Pile foundations	
8 Retaining structures and basement stability	9 Retaining structures 10 Hydraulic failure	
9 Special geotechnical works	5 Fill, dewatering, ground improvement and reinforcement	
10 Construction monitoring and maintenance	4 Supervision of construction, monitoring and maintenance	
	8 Anchorages 12 Embankments	

Fig 1.2 Structure of this *Manual*

BS EN 1538	Diaphragm walls[10]
BS EN 12063	Sheet pile walls[11]
BS EN 12699	Displacement piles[12]
BS EN 12715	Grouting[13]
BS EN 12716	Jet grouting[14]
BS EN 14199	Micropiles[15]
BS EN 14475	Reinforced fill[16]
BS EN 14490	Soil nailing[17]
BS EN 14679	Deep mixing[18]

BS EN 14731	Ground treatment by deep vibration[19]
BS EN 15237	Vertical drainage[20]

Standards prepared by CEN TC 341 and ISO TC 182-SC1 (the committees responsible for standardisation in geotechnical investigation and testing) include:

CEN ISO/TS 17892	Laboratory testing[21]
BS EN ISO 14688	Identification and classification of soil[22]
BS EN ISO 14689	Identification and classification of rock[23]
BS EN ISO 22475	Sampling and groundwater measurements[24]
BS EN ISO 22476	Field testing[25] (in various levels of completeness)
	Part 1: Electrical cone and piezocone penetration tests
	Part 2: Dynamic probing
	Part 3: Standard penetration test
	Part 4: Ménard pressuremeter test
	Part 5: Flexible dilatometer test
	Part 6: Self-boring pressuremeter test
	Part 7: Borehole jacking test
	Part 8: Full displacement pressuremeter test
	Part 9: Field vane test
	Part 10: Weight sounding test
	Part 11: Flat dilatometer test
	Part 12: Mechanical cone penetration test (CPTM)
	Part 13: Plate loading test
ISO/DIS 22477	Geotechnical investigation and testing:[26]
	Part 1: Pile load test by static axially loaded compression
	Part 2: Pile load test by static axially loaded tension test
	Part 3: Pile load test by static transversely loaded tension test
	Part 4: Pile load test by dynamic axially loaded compression test
	Part 5: Testing of anchorages (in preparation)
	Part 6: Testing of nailing
	Part 7: Testing of reinforced fill
	Part 8: Pile testing – statnamic testing

For standards under development (of which there are many) and updates to existing standards refer to the CEN website (www.cen.eu) and ISO website (www.iso.org).

Non-contradictory complementary information includes:

BS 1377	Soil testing[27]
BS 5930	Site investigations[28]
BS 6031	Earthworks[29]
BS 8002	Earth retaining structures[30]
BS 8004	Foundations[31]
BS 8008	Construction and descent of machine-bored shafts[32]
BS 8081	Ground anchorages[33]
PD 6694	Structures subject to traffic loading[34]

CIRIA C580 Embedded retaining walls[35]
DMRB UK Design manual for roads and bridges[36]

It is noted that some aspects of these NCCIs may be in conflict with EC7 principles; however until such time as these conflicts are reconciled, EC7 takes precedence.

Other relevant standards published by BSI (www.bsigroup.co.uk) include:
BS 8006 Strengthened/reinforced soils and other fills[37]

Miscellaneous documents include:
Institution of Civil Engineers (ICE) *Specification for piling and embedded retaining walls*[38]
MCDHW – *Manual of contract documents for highway works*[39]

For exhaustive consultation, reference to the bibliography sections of the national annexes should be made, as should reference to the respective websites of CEN, ISO and BSI.

1.6 Terminology

It is important to understand the notation and terminology used in the Eurocode system of documents from the outset of design. A list of notation used in the Eurocodes and this *Manual* is given in the preliminary pages of this *Manual*. Terminology used in the Eurocodes and this *Manual* is given below.

Action (*F*)
– Set of forces (loads) applied to the structure (direct action).
– Set of imposed deformations or accelerations caused for example, by temperature changes, moisture variation, uneven settlement or earthquakes (indirect action).

Actions are permanent (*G*), variable (*Q*) or accidental (*A*). In many cases the word 'load' can be substituted for 'direct actions'. 'Indirect actions' are imposed deformations resulting from, *inter alia*, uneven settlement or ground movement relative to a foundation.

Effect of action (*E*) – effect of action (or action effect) on structural members, (e.g. internal force, moment, stress, strain) or on the whole structure (e.g. deflection, rotation).

Permanent action (*G*) – action that is likely to act throughout a given reference period and for which the variation in magnitude with time is negligible, or for which the variation is always in the same direction (monotonic) until the action attains a certain limit value.

Variable action (*Q*) – action for which the variation in magnitude with time is neither negligible nor monotonic.

Accidental action (*A*) – action, usually of short duration but of significant magnitude, that is unlikely to occur on a given structure during the design working life.

Geotechnical action – action transmitted to the structure by the ground, fill, standing water or groundwater. Where a structure interacts with the ground, the ground will exert force on the structure whether it be in an 'at-rest' condition or resulting from movement of the ground resulting in contact pressures being lower or higher than 'at-rest' conditions. The action can be formed of a normal pressure, a shear stress or a combination of the two. Depending on circumstances and its use within calculations, the force may be considered to be an action or an action effect within the Eurocode.

Fixed action – action that has a fixed distribution and position over the structure or structural member such that the magnitude and direction of the action are determined unambiguously for the whole structure or structural member if this magnitude and direction are determined at one point on the structure or structural member.

Single action – action that can be assumed to be statistically independent in time and space of any other action acting on the structure.

Static action – action that does not cause significant acceleration of the structure or structural members.

Dynamic action – action that causes significant acceleration of the structure or structural members.

Quasi-static action – dynamic action represented by an equivalent static action in a static model.

Axes

The definitions of axes used in this *Manual* are shown in Figure 1.3a. It should be noted that Eurocode definitions differ from those used in traditional practice as in superseded British Standards. The key point to note here is that when exchanging actions between codes (e.g. Figure 1.3a for EC7 and Figure 1.3b for EC3) a clear protocol must be used to ensure that axes are not accidentally rotated.

Characteristic value of a material or product property (X_k or R_k)

Value of a material or product property having a prescribed probability of not being attained in a hypothetical unlimited test series. This value generally corresponds to a specified fractile of the assumed statistical distribution of the particular property of the material or product. A nominal value is used as the characteristic value in some circumstances. It should be noted that this is not the 'geotechnical' definition.

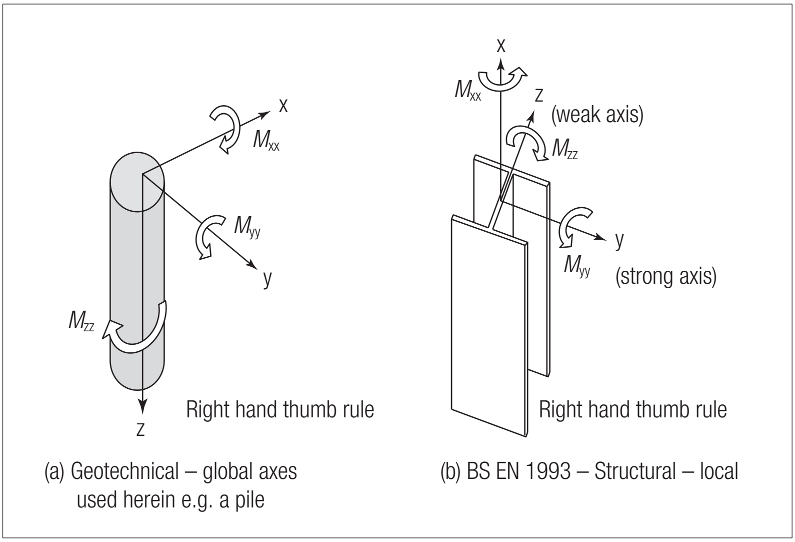


Fig 1.3 Notation of axes

Characteristic value of an action (F_k)
Principal representative value of an action. In Clause 4.1.2 of EC0[3] the characteristic value is generally related to the 5% (or 95%) fractile or for climatic actions a return period of 1 in 50 years.

Note in so far as a characteristic value can be fixed on statistical bases, it is chosen so as to correspond to a prescribed probability of not being exceeded on the unfavourable side during a 'reference period', taking into account the design working life of the structure and the duration of the design situation.

Leading value of a variable action (Q_k)
Value of an action that for a particular load case is considered to occur at its maximum value (with partial factor applied for ULS or characteristic value for SLS). Other variable actions will be accompanying actions.

Accompanying value of a variable action (ψQ_k)
Value of a variable action that accompanies the leading action in a combination. It should be noted that the accompanying value of a variable action may be the combination value, the frequent value or the quasi-permanent value.

Combination value of a variable action ($\psi_0 Q_k$)
Value chosen such that the probability that the effects caused by the combination will be exceeded is approximately the same as caused by the

characteristic value of an individual action. It may be expressed as a determined part of the characteristic value by using a factor $\psi_0 \leq 1$.

Frequent value of a variable action ($\psi_1 Q_k$)
Value determined so that either the total time, within the reference period, during which it is exceeded is only a small given part of the reference period, or the frequency of it being exceeded is limited to a given value. It may be expressed as a determined part of the characteristic value by using a factor $\psi_1 \leq 1$.

Quasi-permanent value of a variable action ($\psi_2 Q_k$)
Value determined so that the total period of time for which it will be exceeded is a large fraction of the reference period. It may be expressed as a determined part of the characteristic value by using a factor $\psi_2 \leq 1$.

Representative value of an action (F_{rep})
Value used for the verification of a limit state. A representative value may be the characteristic value (F_k) or an accompanying value (ψF_k).

Design value of an action (F_d)
Value obtained, usually, by multiplying the representative value by the partial factor γ_f. It is also possible to directly assess the value of the 'design value' where factoring the characteristic value is not considered appropriate. It should be noted that the product of the representative value multiplied by the partial factor γ_F ($=\gamma_{S;d}\gamma_f$) may also be designated as the design value of the action.

Combination of actions
Set of design values used for the verification of the structural reliability for a limit state under the simultaneous influence of different actions.

Characteristic value of geotechnical parameters
Unlike characteristic values of parameters of structural materials which are specified by the designer, characteristic values of geotechnical parameters are assessed by the designer based on geotechnical data. The characteristic value of a geotechnical parameter (Clause 2.4.5.2 of EC7[1]) is the value that is a cautious estimate of the value affecting the occurrence of a limit state. Within typical UK practice the choice of a characteristic value of a geotechnical parameter is not based on statistics but on an engineering appreciation of the ground strength or stiffness which can be mobilised in the limit state being considered. Critically, the characteristic value is not a unique value for a given material but assessed relevant to the limit state being considered. The topic is discussed in Section 4.2.

Characteristic value of a geometrical property (a_k)
Value usually corresponding to the dimensions specified in the design. Where relevant, values of geometrical quantities may correspond to some prescribed fractiles of the statistical distribution.

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.

Construction works
Everything that is constructed or results from construction operations.

Type of construction
Indication of the principal structural material, e.g. reinforced concrete construction, steel construction, timber construction, masonry construction, steel and concrete composite construction.

Method of construction
Manner in which the execution will be carried out, e.g. cast in place, prefabricated, cantilevered.

Construction material
Material used in construction work, e.g. concrete, steel, timber, masonry.

Type of building or civil engineering works
Type of construction works designating its intended purpose, e.g. dwelling house, retaining wall, industrial building, road bridge.

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

Design criteria
Quantitative formulations that describe for each limit state the conditions to be fulfilled.

Design situations
Sets of physical conditions representing the real conditions occurring during a certain time interval for which the design will demonstrate that relevant limit states are not exceeded.

Transient design situation
Design situation that is relevant during a period much shorter than the design working life of the structure and which has a high probability of occurrence (e.g. during construction or repair).

Persistent design situation
Design situation that is relevant during a period of the same order as the design working life of the structure.

Accidental design situation
Design situation involving exceptional conditions of the structure or its exposure, including fire, explosion, impact or local failure.

Design working life
Assumed period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary. Materials should be specified such that predictable deterioration is accounted for in the design (e.g. loss of section for sheet piles due to corrosion). The design life may rely on appropriate maintenance provisions subject to the agreement of the employer and the overseeing authority.

Design value of a geometrical property (a_d)
Generally a nominal value. Where relevant, values of geometrical quantities may correspond to some prescribed fractile of the statistical distribution.

Design value of a material or product property (X_d or R_d)
Value obtained by dividing the characteristic value by a partial factor γ_m or γ_M, or, in special circumstances, by direct determination.

Execution
All activities carried out for the physical completion of the work including procurement, the inspection and documentation thereof (execution includes both on and off site fabrication).

Ground
Soil, rock and fill in place prior to the execution of the construction works.

Hazard
An unusual and severe event, e.g. an abnormal action or environmental influence, insufficient strength or resistance, or excessive deviation from intended dimensions.

Limit states
States beyond which the structure no longer fulfils the relevant design criteria.

Ultimate limit states
States associated with collapse or with other similar forms of structural failure.

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

Irreversible serviceability limit states
Serviceability limit states where some consequences of actions exceeding the specified service requirements will remain when the actions are removed.

Reversible serviceability limit states
Serviceability limit states where no consequences of actions exceeding the specified service requirements will remain when the actions are removed.

Load arrangement
Identification of the position, magnitude and direction of a free action.

Load case
Compatible load arrangements, sets of deformations and imperfections considered simultaneously with fixed variable actions and permanent actions for a particular verification.

Maintenance
Set of activities performed during the working life of the structure in order to enable it to fulfil the requirements for reliability.

Nominal value
Value fixed on non-statistical bases, for instance on acquired experience or on physical conditions.

Nominal value of a material or product property (X_{nom} or R_{nom})
Value normally used as a characteristic value and established from an appropriate document such as a European Standard or Prestandard.

Reference period
Chosen period of time that is used as a basis for assessing statistically variable actions, and possibly for accidental actions.

Reliability
Ability of a structure or a structural member to fulfil the specified requirements, including the design working life, for which it has been designed. Reliability is usually expressed in probabilistic terms.

Repair
Activities performed to preserve or to restore the function of a structure that fall outside the definition of maintenance.

Resistance
Capacity of a member, component, cross-section of a member, or component of a structure, to withstand actions without mechanical failure e.g. bending resistance, buckling resistance, tension resistance and geotechnical bearing capacity, pile capacity, passive resistance etc.

Serviceability criterion
Design criterion for a serviceability limit state.

Strength
Mechanical property of a material indicating its ability to resist actions, usually given in units of stress.

Stiffness
Material resistance against deformation.

Structural analysis
Procedure or algorithm for determination of action effects in every point of a structure.

Structure
Organised combination of connected parts designed to carry loads and provide adequate rigidity.

Structural member
Physically distinguishable part of a structure, e.g. a column, a beam, a slab, a foundation pile.

Form of structure
Arrangement of structural members (e.g. a frame or a cantilever retaining wall).

Structural system
Load bearing members of a building or civil engineering works and the way in which these members function together.

Structural model
Idealisation of the structural system used for the purposes of analysis, design and verification.

Units
For geotechnical calculations, the following units or their multiples are recommended:
– force kN
– mass kg
– moment kNm
– mass density kg/m^3
– weight density kN/m^3
– stress, pressure, strength and stiffness kN/m^2
– coefficient of permeability m/s
– coefficient of consolidation m^2/s

1.7 Structural and geotechnical interaction

The interaction between structural and geotechnical disciplines in terms of design responsibility is not defined in the Eurocodes. However, given the increased complexity of design as will become apparent as the reader progresses through EC7 or this *Manual*, it is considered that a framework of interaction between geotechnical and structural engineering actions should be put in place. Figure 1.4[40] provides a representative outline of the division of tasks that can be used as a starting point. Further comments on this topic are given in Section 4.3.

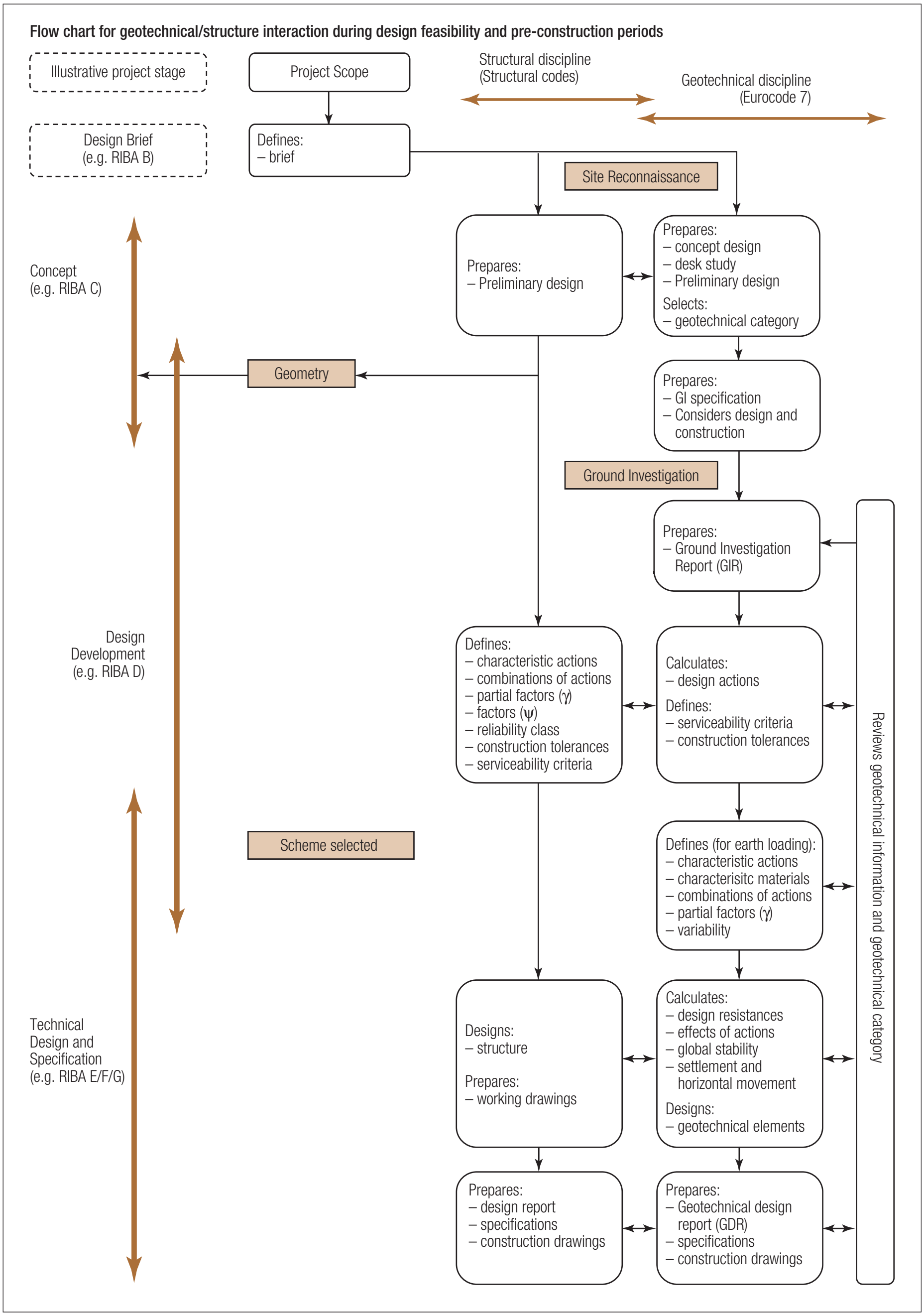


Fig 1.4 Structural and geotechnical interaction based on RIBA (2008) Outline Plan of Work

1.8 Risk registers

The term 'risk register' is not part of the EC7 lexicon. Risk control is however very much part of the Eurocode philosophy. A useful tool for risk control is clearly the use of risk registers. These may be populated under a number of headings:

- Designers risk register associated with technical aspects of the project.
- Designers risk register associated with health and safety issues (see Section 10.2).
- Risk register associated with sustainability aspects of the project.

It is noted that CDM (2007)[41] requirements play a very important part in the design process. Whilst not related to EC7 the designer must also follow these regulations. Comments are provided in Section 10.2.

1.9 Summary

Important observations from this chapter include the following:

- Geotechnical design involves ground (soil, rock, water) as the principle material. Ground is inherently variable and to account for this three Geotechnical Categories (GC1 to GC3) have been introduced (see Section 2.9). Use of this *Manual* is appropriate for GC1 and GC2 when used by an engineer with appropriate experience and competence in ground engineering; this *Manual* does not cover GC3 situations.

- Axes used in geotechnical and structural design vary (see Figure 1.3). This must be remembered when interchanging actions between codes.

- The geotechnical design process relies on ground investigations. These investigations take time to procure and carry out, and need to be allowed for in the project programme to inform the design process. An illustrative programme showing geotechnical investigations is presented in Figure 1.4[40].

- Documents relevant to geotechnical design to EC7 extend far beyond EC7.

- In comparison to traditional design, the Eurocodes use significantly different terminology.

2 Basis of design

2.1 Introduction

Geotechnical design in the UK has traditionally used a working stress approach with global factors of safety (e.g. for the design of spread footings, piles and gravity retaining walls, though not for embedded retaining walls). This approach is in contrast to the design methodology in the Eurocodes where partial factors are applied to actions or their effects (i.e. loads or load effects) and to soil strength or resistance (i.e. capacity). The Eurocode design philosophy provides designers with a methodology in which partial factors make allowance for variable uncertainty (actions, material strengths and ground resistances etc.) and for the likelihood of an event occurring. The result is that structures designed to the Eurocodes should have an acceptable and consistent reliability of performance for each of the limit states considered.

Prior to embarking further into the chapter a short introduction to terminology is provided in order to set the context for what comes next.

A 'limit state' (see Section 1.6) is the condition where a structure no longer fulfils the relevant design criteria that have been set for it. Limit states are usually verified by means of calculation (e.g. 'design action effect' ≤ 'design resistance') (see Sections 2.4 and 2.11).

When designing a structure there will be a wide variety of conditions which the design needs to address and for which the structure will need to perform adequately. Depending on the likelihood of the condition being considered the required performance may vary. These differing conditions are called 'design situations' (see Section 2.5). They include everyday loading, for which the structure should not show distress, up to accidental situations, for which the structure may only need to remain stable during and immediately after an event in order to protect life. This process allows each design situation to be guarded against in a manner that is proportionate to the likelihood and severity of the limit state being considered. The magnitude of the partial factors used is dependent on a number of factors including: the required 'reliability' of the structure (see Section 2.6), the complexity of the structure and the ground conditions as defined by one of three 'Geotechnical Categories' (see Section 2.9).

This *Manual* introduces the concepts presented in EC0[3] and its UK National Annex[42] in order to set the context for EC7. Readers will need to familiarise themselves with these documents as it is not the intention of this *Manual* to cover them in an exhaustive manner.

2.2 Building regulations

The designs of all significant structures built in the UK are required to satisfy the Building Regulations as implemented in England, Wales, Scotland and Northern Ireland. These regulations and their supporting documents (published by government ministries) are being updated to reflect the Eurocode documents.

2.3 Limit states

Two categories of limit state are identified in Section 3 of EC0[3]: the ultimate limit state (ULS) and the serviceability limit state (SLS). Unlike working stress design where the designer is concerned with what is expected to happen, limit state design directs the designer to the unexpected (i.e. what might go wrong) by means of factoring actions and strengths to guard against the limit state being exceeded.

Ultimate limit states concern the safety of people and structures whereas serviceability limit states concern structural function under normal use, comfort and appearance (e.g. deflection and cracking). Ultimate limit states should be extremely unlikely states while serviceability limit states should be rare rather than likely states.

Ultimate limit states that may be relevant and require verification in geotechnical design include:
– loss of rigid body equilibrium (e.g. toppling of a column or of a block or rock)
– excessive deformation (e.g. movement resulting in excessive damage to a structure)
– formation of a failure mechanism (e.g. a slope failure, a bearing capacity failure of a foundation or rotation of a retaining wall)
– rupture (e.g. structural failure of a retaining wall or anchor).

Serviceability limit states that may be relevant and require verification include:
– deformation (e.g. movements resulting in loss of, or reduction in, function)
– vibration (e.g. resulting in loss of function)
– damage (e.g. up to a level not resulting in ULS failure).

In considering serviceability limit states, a further requirement is that reversible and irreversible states be distinguished (see definitions in Section 1.6). In general terms, reversible states can be thought of as ‘elastic behaviour’ and irreversible as limited ‘plastic behaviour’. In geotechnical design for typical UK structures (assuming relatively low live loading) there is a limited benefit in differentiating between reversible and irreversible states (c.f. a bridge with large traffic and wind loading where there is a need to differentiate between load combinations where the ground remains in the elastic range and for more extreme load combinations where limited plasticity may be acceptable).

In practice, observation based on experience often allows identification of the governing limit state which enables other limit states to be verified by a control check (e.g. a simple hand calculation or comparison to previous work). Furthermore, verification of either ULS or SLS may be omitted if there is sufficient evidence to demonstrate that one limit state is satisfied by the other (usually limited to prescriptive design of relative simple structures, see Clause 2.5 of EC7 Part 1[1]).

2.4 Limit state verification methods

Structures should be designed (see Section 2 of EC0[3]) and built in such a way that they will, with appropriate reliability and cost, resist without damage the actions imposed during both the construction phase and the design life; they should also satisfy serviceability criteria and have adequate durability. Furthermore, structures should be designed and built in such a way that they will not be damaged, to an extent disproportionate to the cause of damage, by events such as explosion, impact or human error (i.e. structures must be appropriately robust).

Verification that limit states are not being exceeded can be performed in one or more of the following ways:

– *Design by calculation (see Section 2.11)*
  Design by calculation uses an arithmetical approach to the comparison of actions and resistances, mainly using a partial factor method (see Section 6 of EC0[3] and Section 2.4 of EC7 Part 1[1]).

– *Design by prescriptive measures (see Section 2.12)*
  Design by prescriptive measures relies on past experience of similar structures in similar ground conditions. Partial factors are not used as the design is typically based on design details taken from standard charts or guides (e.g. standard sections for a modular retaining wall) or by comparison of working loads and allowable resistances.

– *Design by the observational method (see Section 2.13)*
  Design by the observational method is a hybrid approach requiring a combination of calculation and comparable experience and monitoring to optimise construction whilst ensuring robust in-service performance. The benefit of using this approach is the combined effects of adopting less cautious ground parameters and possibly by using lower partial factors against failure during the period of monitoring. The method relies on being able to predict and react against a limit state being exceeded.

– *Design by experimental models and load tests (see Section 2.14)*
Design by experimental models and load tests include, for example, centrifuge modelling and load tests on piles.

Accepting these four ways of verifying limit states, the resulting design should also be checked against comparable experience wherever possible.

Of these design options, design by calculation ranks foremost in EC7; the remaining options get less attention.

Within this *Manual* design by prescriptive measures is discussed in relation to preliminary design and design of Geotechnical Category 1 structures (see Section 2.9) while design by calculation is addressed in relation to detailed design. Design by experimental models and the observational method are mentioned only in passing as they are best exercised by a geotechnical specialist.

2.5 Design situations

Design situations can be thought of as snapshots in time of a structure during its construction and design life for which relevant limit states need to be verified. They are selected by considering the circumstances under which a structure is required to fulfil its function and are required to be sufficiently severe and varied so as to encompass all reasonably foreseeable conditions. For each design situation it is necessary to identify the relevant load case(s) and verify that no relevant limit state is exceeded. Design situations are introduced in Section 2.2 of EC7 Part 1[1].

Design situations are classified as:
– persistent (normal conditions)
– transient (temporary conditions)
– accidental (exceptional conditions).

The formulation of design situations and their related limit states for geotechnical design should include consideration of the following:
– actions
– overall stability and ground movements
– disposition and classification of the various zones of soil, rock and elements of construction
– dipping bedding planes in ground strata
– mine workings, caves or other underground structures
– inter-bedded hard and soft strata
– faults, joints and fissures
– possible instability of rock blocks
– dissolution cavities and continuing dissolution processes
– environmental conditions (e.g. scour, erosion, excavation, chemical

corrosion, weathering, freezing, long duration droughts, variations in groundwater levels and gases)
– subsidence
– sensitivity of the structure to deformations
– effect of a new structure on existing structures, services and the local environment
– nature and size of a structure, and its elements
– conditions with regard to its surroundings
– ground conditions
– groundwater conditions
– influence of the environment (hydrology, surface water, seasonal changes of temperature and moisture).

2.6 Reliability

Reliability is achieved through design in accordance with the Eurocodes, appropriate execution and quality management. Different levels of reliability may be selected as a function of the required performance. Readers are directed to Section 2.2 and Annex B of EC0[3] for further information. Table 2.1 provides a qualitative introduction to levels of reliability where the levels of checking and inspection are summarised for different classes.

Table 2.1 Reliability levels and typical checking/inspection requirements

Consequence class (human life, economics, environmental, social) (CC)	Low	Medium	High
	CC1	CC2	CC3
Illustrative building type	Agricultural building	Typical framed residential/office building	Public building/concert hall/unusual structures
Reliability Class (RC)	RC1	RC2	RC3
Design Supervision Level (DSL)	DSL1	DSL2	DSL3
Checking[a]	Self check	Independent person and agreed procedure	Third party check
Inspection Level during execution (IL)	IL1	IL2	IL3
Inspection routine	Self inspection	Inspection to agreed procedure	Third party inspection

Notes

a It should be noted that checking requirements vary according to the location and situation of the structure. For example, structures which impact on the Highways Agency will need to be checked according to HA requirements which vary from those above in Table 2.1 in certain aspects.

b Further details in Annex B of EC0[3].

2.7 Design working life

Table 2.2 may be used as a guide to the selection of a design working life. The actual design working life should be agreed with the client instructing the works.

In some instances a design life of less than 10 years will be appropriate. This is particularly relevant to temporary structures where design life may be measured in months or a reduced number of years. An example of this would be temporary propping for a permanent retaining wall or temporary ground anchors.

Table 2.2 Guide to design working life

Design working life (years)	Example structure
10	Temporary structures
10–30	Replaceable parts of structures
15–25	Agricultural or similar structures
50	Buildings/common structures
120	Monumental buildings
Note Taken from Table NA.2.1 of NA to EC0[42].	

2.8 Durability

Consideration of durability is a key part of design. A designer must assess the significance of the environment on the structure and on how the structure, or element of the structure, will behave with time. The result of the assessment may include the need for regular inspection and maintenance (e.g. retaining wall drainage) or it may result in the specification of sacrificial material thicknesses (e.g. sheet piles). Durability assessment is necessary to enable provisions to be made for protection or adequate resistance during the design working life.

Material deterioration can be estimated by calculation, experiment and/or previous experience. Aspects that should be considered when assessing durability include:
– potential use of the structure
– design criteria
– material characteristics
– environmental conditions
– previous experience in similar circumstances

– type of structural system
– nature of element (structural or geotechnical)
– workmanship and construction control
– protective measures
– relevant construction material standards
– inspection and maintenance requirements and how they are addressed within CDM constraints (see Section 10.4.2).

For materials in the ground or in groundwater the following issues should be considered:

Concrete
– Aggressive agents, e.g. acids, sulphate salts (see Section 4.4.2).

Steel
– Chemical attack where ground permeability permits seepage of groundwater and oxygen.
– Corrosion on areas exposed to free water, e.g. splash zone (see Section 4.4.3).
– Pitting attack for steel in fissured or porous concrete.

Timber
– Fungi, bacteria, insect attack and the like.

Synthetic fabrics
– UV light, ozone, combined temperature and stress, chemical degradation.

2.9 Geotechnical categories

The complexity of a geotechnical design (or parts of a design) needs to be identified with its associated risks in order to establish minimum requirements for the scope of geotechnical investigations, design calculations and construction control. This is achieved by introducing three Geotechnical Categories as shown in Table 2.3 (see also Clause 2.1 (10) of EC7 Part 1[1]).

The designer needs to select a category for the site and structure. It should be recognised that the initial, or revised, category can be changed. A site or structure that is initially assessed to be GC1 can very easily move to GC2 as more information becomes available as it can from GC2 to GC3 if, for example, a shallow tunnel was identified below the building footprint. The category can be reduced if situations indicate this to be appropriate. Different parts of a site or structure can be in a different category so long as the resulting design is compatible from area to area. The designer and those executing the design must always consider and reconsider data as it becomes available during investigation, design and execution. Only in this manner can geotechnical categorisation have real value and in doing so, safeguard engineers from complacency.

Table 2.3 Geotechnical categories

Category	Description
GC1	Small and simple structures. Qualitative investigation and analysis. Negligible risk. Straightforward ground conditions. Routine design and construction methods. No excavation below the water table unless comparable local experience indicates it will be straightforward.
GC2	Conventional structures. Quantitative investigation and analysis. Normal risk. No difficult soil and site conditions. No difficult loading conditions. Routine methods.
GC3[a]	Large and unusual structures. Abnormal risk. Difficult ground. Difficult loading conditions.
Note **a** This *Manual* considers GC1 and GC2 but not GC3.	

2.10 Partial factor method

2.10.1 Procedure

EC7 requires that the assessment of limit states follows a clear procedure. There is a general progression from observed/measured values for actions, materials and resistances, to characteristic values and thereafter to design values. The progression from one value to another can be based on non-arithmetic assessment while in other cases the progression is required to be numeric. With a bias towards pile design (especially column on 'resistances'), the general progression from observed/measured data to design values is illustrated in Table 2.4 (see also Figure 3.1).

2.10.2 Single source

A key rule when considering the application of partial factors is the 'single source' rule[43] applied to actions. This rule dictates that the same partial factor should be applied to an entire element irrespective of the whether parts of the element are variably destabilising and stabilising. The classic example of this is a balanced cantilever where one arm stabilises the destabilising action of the other arm (and *vice versa*). The single source rule prevents the actions generated by the symmetrical arms being factored differently for all but one limit state calculation which explicitly looks at such situations (EQU limit state, see Section 2.11.3). In geotechnical considerations the single source rule requires that the density of soil, or water, in the part of a failure mechanism that is stabilising need not be treated differently to soil that is in the destabilising part of the mechanism. See also notes for Tables 2.6 to 2.10 prior to Table 2.6a.

Table 2.4 Outline of partial factor method

Value	Material	Action	Resistance
Observed or measured value	X_{meas}	–	R_{meas}
Calculated value	–	–	R_{cal}
Correlation factor	Yes[a]	–	ξ
Derived value	$X_{derived}$	–	–
Assessment of characteristic value	Cautious estimate	–	–
Model factor	–	–	$\gamma_{R;d}$
Characteristic value	X_k	F_k	$R_k = R_{meas}/\xi$[b] $R_k = R_{cal}/\gamma_{R;d}$[c]
Partial factor	γ_M	γ_F	γ_R
Additional reduction factor	–	ξ (permanent) ψ (variable)	–
Design value	$X_d = X_k/\gamma_M$[d]	$F_d = \xi\,\psi\,\gamma_F\,F_k$[d]	$R_d = R_k/\gamma_R$

Notes

a Alternatively the derived value may be obtained from a measured value directly or by theory.
b This approach is used when the characteristic resistance is obtained from a series of pile load tests or from calculations using profiles of geotechnical data (e.g. Cone Penetration Test (CPT) data).
c This approach is used in conventional pile design using geotechnical parameters. The R_{cal} is the calculated value of ultimate pile resistance using pile design equations.
d Alternatively (not usually) the design value may be derived directly from the characteristic value without using partial factors.
e Subscripts: cal = calculated, d = design, F = action, k = characteristic, M = material, meas = measured, R = resistance, R;d = model factor related to resistance.

2.11 Design by calculation

2.11.1 Introduction

In the main, design by calculation is the most used method in EC7 for verifying that limit states are not exceeded. It involves actions, material properties, geometrical data, serviceability criteria and calculation models. Avoidance of limit states is verified by the partial factor method. In certain design situations, design by prescriptive means is also a viable alternative to design by calculation where behaviour of the ground and structure is well understood (see Section 2.12).

In design by calculation, calculation models are required to meet the following criteria:
– describe the assumed ground behaviour for the limit state under consideration
– may be analytical, semi-empirical and/or numerical
– be either accurate or conservative
– empirical models must be relevant to the existing ground conditions.

Where there is uncertainty in the accuracy, safety or performance based on calculations using partial factors, model factors may be used to ensure that the design calculations are either accurate or conservative. Model factors are introduced to reduce the design strength or resistance of the ground, to increase loads or to adjust the results of a calculation. Model factors should consider the range of uncertainty in the results of a model and known systematic errors associated with a model. Where model factors are not specified in national annexes, they may be introduced following agreement with the client and relevant authorities (see Section A.6 of NA to EC7 Part 1[44]).

The UK National Annex to EC7 specifies model factors for resistance when using the 'design by calculation' approach for pile design (see Chapter 7) and also, albeit not covered in this *Manual*, for pile design based on dynamic load testing. Elsewhere it can be assumed that the partial factors presented are adequate and implicitly incorporate a model factor to provide the necessary safety for buildings designed using conventional calculation methods.

Soil-structure interaction and strain compatibility are important issues. Materials that exhibit brittle or strain-softening behaviour should be given particular attention. Stress-strain relationships and ground stress states should be sufficiently understood to verify performance.

Design by calculation covers both ULS and SLS design.

It should be remembered that the quality of geotechnical investigation and data obtained from it is usually more important than the precision of the calculation model (the calculation model must always be appropriate) or the value of the partial factors.

2.11.2 Basic variables

2.11.2.1 Actions

General

Definitions are provided in Section 1.6; the following provides an overview. An action F is a force or imposed deformation applied to a structure and is classified primarily by its variation in time as follows:
– Permanent G: once applied the action remains unchanged.
– Variable Q: can be applied and removed and reapplied.
– Accidental A: an extreme and unusual, but imaginable, event.

These can be further divided by origin, space or nature as follows:

Origin:
– Direct: force applied directly to the structure (e.g. a column load).
– Indirect: imposed deformation due to temperature change, moisture change or uneven settlement. Also seismic loading (not covered in this *Manual*).

Space:
– Fixed: an action that has a fixed and unambiguous distribution and position on the structure.
– Free: an action that may have various spatial distributions over the structure.

Nature of response:
– Static: behaviour when the action is applied gradually, not causing significant acceleration to the structure or element of the structure.
– Dynamic: behaviour when the action is applied rapidly, causing significant acceleration to the structure or element of the structure.

The characteristic value F_k of an action is either a mean, upper, lower or nominal value. In general terms, the design value F_d is expressed as:

$$F_d = \gamma_F \psi F_k$$

where:

γ_F	is a partial factor for all actions
ψ	is a reduction factor which is only applied to accompanying variable actions
ψF_k	is also known as the representative action, F_{rep}.

Values for γ_F are presented in Section 2.11.3.7. For permanent and variable actions γ_F is renamed γ_G and γ_Q respectively. For the leading variable action $\psi = 1.0$; other values for ψ (less than unity) are presented in Section 2.11.3.7.

While beyond the scope of this *Manual*, it should be noted that F_d can also be directly assessed using values of γ_F as a guide to the required level of safety.

Permanent actions
Generally, the characteristic value of a permanent action depends upon its coefficient of variation. If the coefficient of variation is small then a single mean value G_k may be used, otherwise upper $G_{k;sup}$ and lower $G_{k;inf}$ values are required. It is noted that self-weight may adopt a single value using nominal dimensions and mean unit masses.

Where a distinction between favourable and unfavourable is made for permanent actions, γ_G is subdivided as $\gamma_{G;inf}$ or $\gamma_{G;stb}$, and $\gamma_{G;sup}$ or $\gamma_{G;dst}$ respectively.

Variable actions
The characteristic value of a variable action Q_k is either an upper, lower or nominal value.

When combining variable actions (see Sections 2.11.3.2 and 2.11.4.2) that can occur simultaneously the following reduction factors are applied:
– $\psi_0 Q_k$: combination value
– $\psi_1 Q_k$: frequent value
– $\psi_2 Q_k$: quasi-permanent value.

The ψ_0 factor reduces the magnitude of characteristic actions due to the reduced likelihood of simultaneous occurrence. The ψ_1 factor reduces the magnitude of characteristic actions so that they represent a value which is exceeded occasionally. The ψ_2 factor reduces the magnitude of characteristic actions to a value which is exceeded for a large fraction of the design life.

Where a distinction between favourable and unfavourable is made for variable actions, γ_Q for favourable actions is always 0.0. For some limit states (EQU, UPL and HYD as defined in Section 2.11.3) the factors are termed $\gamma_{Q;stb}$ and $\gamma_{Q;dst}$, respectively, with $\gamma_{Q;stb} = 0.0$.

Accidental actions
For accidental actions the design value A_d is used.

Further information on actions and their values can be found in EC0[3] and EC1[4].

Actions generated by geotechnical elements
Geotechnical actions include:
– self-weight of soil, rock and water
– stresses in the ground
– earth pressures
– free-water pressures
– groundwater pressures
– seepage forces
– loads from structures
– surcharges
– removal of load or excavation of ground
– traffic loads
– movements caused by mining or other caving or tunnelling activities
– swelling and shrinkage caused by vegetation, climate or moisture changes
– movements due to creeping or sliding or settling ground masses
– movements due to degradation, dispersion, decomposition, self-compaction and solution
– movements and accelerations caused by explosions, vibrations and dynamic loads
– temperature effects, including frost heave
– ice loading
– prestress in struts
– downdrag.

Certain aspects of geotechnical actions require special consideration:
- Even in relatively straightforward design, geotechnical actions may change during a design as a function of ground stiffness/strength (e.g. the design of propped embedded retaining wall in clay where excess pore-water pressure dissipates).
- Also of importance is the way in which free water or groundwater is handled in calculations. Unlike soil which has factored strength parameters, there is no consistent way to factor water and this makes the choice of the characteristic or design water level particularly important in ensuring safety.
- The manner in which ground reacts to locally applied actions in a model may result in actions being changed elsewhere in the model. This would occur in a soil-structure interaction model (e.g. a pile raft with non-linear piles) or in the case of actions which repeat or vary in intensity causing locally increased settlements.

All these aspects, and more, need to be considered when assessing actions generated or influenced by soil, rock or water.

Actions presented in EC7 for which there is no value in EC1 may be independently derived. The values of these actions and their factors should be agreed with the client and relevant authorities.

2.11.2.2 Walls subjected to traffic loading

Published Documents (PDs) are not standards but are prepared to provide guidance and can take the form of NCCI (non-contradictory complementary information) documents. Document PD 6694-1:2011[34] *Recommendations for the design of structures subjected to traffic loading to BS EN 1997-1:2004* is one such document and has been prepared to provide guidance on how traffic loading on the ground surface impacts on structures. Of particular importance to typical UK buildings is the design of retaining walls which support highways. The guidance in PD 6694-1 is thorough and makes careful consideration of both individual wheel or axle loads and average UDL loading in clearly identified lanes of traffic. The drawback with the approach is that, while being thorough, it is far more complicated than conventional design based on surface UDL loading (e.g. of the now superseded 10kN/m^2 for HA loading and 20kN/m^2 for HB loading as contained in DMRB[36] document BD 37/01 *Loads for highway bridges* which was typically used when designing retaining walls). A further complication highlighted in the introduction to PD 6694-1 is that retaining walls with traffic loading should be designed with bridge partial factors (rather than those for buildings) unless agreed otherwise with the overseeing authority.

Design which is compliant to PD 6694-1[34] requires that the full geometry of the wall and carriageway be considered when deriving the various combination of surface loading required. Research has been carried out on surcharge loading (e.g. Shave et al[45]) which provides a commentary on PD 6694-1.

It is noted that the requirements of PD 6694-1[34] provide rigorous design guidance for situations where traffic can access close to the back of the retaining wall resulting in localised high pressures acting on the retaining wall; such pressure will normally result in higher loads on short sections of wall than equivalent UDLs. In this situation the wall will need to be designed to redistribute concentrated loads to adjacent sections of wall in order to provide a robust design. In situations where traffic loading is kept away from the back of the retaining wall equivalent UDLs are likely to be appropriate for design. It is recommended that the design loading be agreed with the overseeing authority at an early stage in the design process.

2.11.2.3 Materials

General

The properties of materials are generally represented by characteristic values X_k. A characteristic value is a cautious estimate of the value of the parameter when considering a specific limit state.

Design values X_d, when not directly assessed, are obtained from:

$$X_d = X_k / \gamma_M$$

where:
γ_M is the partial factor for materials (as per Section 2.10.1).

Where design values are directly assessed, the values of γ_M are a guide to the required level of safety. Values for γ_M are presented in Section 2.11.3.7.

For guidance on choosing structural material properties such as upper and lower characteristic strengths and accounting for the effects of fatigue, the reader should refer to Sections 4.2 and 6.3.3 of EC0[3].

Geotechnical aspects

Geotechnical parameters (derived values) for soil and rock are obtained from test results and other relevant data, interpreted appropriately for the limit state under consideration.

The following should be considered when establishing geotechnical parameters:
– relevant published and well recognised information
– the value compared with relevant published data and local experience
– the potential variation
– results of nearby large scale field trials and measurements
– correlations from more than one type of test
– potential deterioration in ground properties during the design life.

Geotechnical parameters obtained from test results may differ from those governing the behaviour of a geotechnical structure. The following factors may be contributory:
– stress and strain level and mode of deformation
– soil and rock structure
– time effects
– softening due to percolating water

– softening due to dynamic actions
– brittleness or ductility
– installation method for structure
– workmanship for made ground or improved ground
– construction activities
– geotechnical test methods and sample disturbance.

The way in which derived values are converted to characteristic values for geotechnical materials is introduced in Chapter 4. However, the method in which derived values are obtained is beyond the scope of this *Manual*; advice from a geotechnical specialist should be sought.

2.11.2.4 Geometrical data

General

Geometrical data is represented by characteristic or design values. The characteristic value a_k is generally taken to be the dimension specified in a design while the design value a_d is generally a nominal value a_{nom}. In practice the characteristic value and design value are the same unless a particular limit state is sensitive to deviations in geometrical data. For this situation:

$$a_d = a_{nom} \pm \Delta a$$

where:
Δa represents the change to account for geometrical deviations.

Possible deviations from the assumed directions or positions of actions are to be considered. It should be noted that partial factors for actions and materials include allowances for minor variations in geometrical data.

Geotechnical aspects

Geotechnical geometrical data includes: ground surface level and slope, water levels, strata interface levels, excavation levels and the dimensions of structures. Characteristic values may be measured, nominal or upper and lower estimates.

Values for Δa for spread foundations when assessing the effects of eccentric loading and for retaining walls for the assessment of overdig are presented in Chapters 6 and 8 of this *Manual*.

Other examples of construction tolerance include pile position and inclination below pile mat level as well as retaining wall installation deviations on water proofing for a secant pile wall and generally for architectural space within a basement.

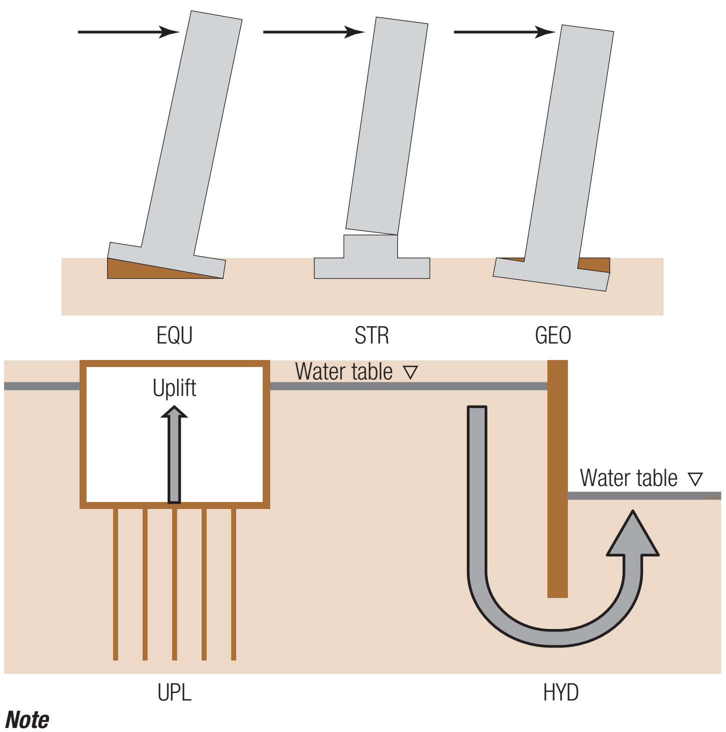
2.11.3 Ultimate limit states

2.11.3.1 Introduction

Verification that each of the following ultimate limit states is not exceeded is required, when relevant:

– EQU – 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.
– STR – internal failure or excessive deformation of the structure or structural elements in which the strength of structural materials is significant in providing resistance.
– GEO – failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance.
– UPL – loss of equilibrium of the structure or the ground due to uplift by water pressure or other vertical actions.
– HYD – hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients (groundwater flow).

These limit states are illustrated in Figure 2.1.



Note
EQU: rotation of the tower about the toe of the foundation (ground is strong)
STR: structural failure of the tower leg (or of the pad foundation)
GEO: bearing capacity of the foundation soil (or sliding on the pad to soil interface)
UPL: flotation of the box held down by weight and vertical tension piles
HYD: failure of the soil in heave due to seepage pressure

Fig 2.1 Illustrative distinctions between EQU, STR, GEO, UPL and HYD limit states

Table 2.5 Ultimate limit states

Limit state	Comments
EQU	– EQU is not usually applicable to typical UK buildings which are generally not prone to geometric instability. Typical non-building examples include balanced cantilever bridge decks, gravity retaining walls on strong rock or toppling of blocks of rock in rock cuttings. For a discussion on the application of EQU the reader is referred to Schuppener et al[43] and Simpson[46].
STR/GEO	– STR and GEO are the most easily understood states. – STR is generally taken to relate to structures and their material strengths (e.g. reinforced concrete or sheet pile structural design). – GEO is generally taken to relate to the ground and its strength (e.g. soil shear strength and geotechnical failure). – In design both are assessed for compatibility between structure and ground.
UPL and HYD	– UPL and HYD address the effects of water on structures and the ground. – For UPL, disturbing actions are generally obtained from water pressures in static water and are used to assess the vertical stability of a structure (e.g. a submerged basement box) to ensure that it does not fail in heave or buoyancy. – HYD on the other hand looks at the movement of water through a soil mass to ensure that the soil mass is stable against hydraulic heave, internal erosion or piping.
Note See EC7[1], Clause 2.4.7.1.	

The STR and GEO limit states use the same two sets of partial factors (i.e. consideration of STR limit state is checked with the same sets of partial factors as the GEO limit states). All other limits states are treated with a unique set of partial factors.

In Table 2.5, each limit state is considered in general terms.

2.11.3.2 Combination of actions

For each load case being assessed, actions that are considered to occur simultaneously need to be combined as below.

For persistent and transient design situations (see Section 2.5) the combination of actions can be expressed as:

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i \geq 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad \text{(Eqn. 2.1)}$$

Clause 6.4.3 of EC0[3] allows alternative combinations of actions for STR/GEO as in Eqns 2.1a and 2.1b below. When this option is taken then the more onerous of the two must be used in place of Eqn 2.1. It is noted that for new designs Eqn 2.1 is recommended (not a Eurocode recommendation); however Eqns 2.1a and 2.1b may be used where deemed beneficial by the designer.

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} \psi_{0,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad \text{(Eqn. 2.1a)}$$

$$\sum_{j \geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad \text{(Eqn. 2.1b)}$$

For accidental design situations the combination of actions can be expressed as:

$$\sum_{j \geq 1} G_{k,j} + P + A_d + \psi_{1,1} Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i} \qquad \text{(Eqn. 2.2)}$$

where:
A_d is the design value of an accidental action
$G_{k,j}$ is the characteristic values of permanent actions
$Q_{k,1}$ is the characteristic value of leading variable action
$Q_{k,i}$ is the characteristic value of accompanying variable action
P is the prestress
$\gamma_{G,j}$ is the partial factor on permanent characteristic actions
γ_P is the partial factor on prestress action
$\gamma_{Q,1}$ is the partial factor on leading variable characteristic action
$\gamma_{Q,i}$ is the partial factor on accompanying variable characteristic actions
ξ_j is a reduction factor for unfavourable permanent actions
$\psi_{0,1}$ is a combination factor for leading variable action
$\psi_{0,i}$ is a combination factor for inferior variable actions
$\psi_{1,1}$ is a combination factor for frequent value of a variable action
$\psi_{2,i}$ is a combination factor for quasi-permanent value of a variable actions

In the equations above there is distinction between the leading variable action and the accompanying variable actions. This allows realistic combinations of actions to be applied where more than variable actions may exist at any one time without being overly conservative.

The assessment of actions is presented in EC1[4]. The exception to this is where actions are generated by a soil mass, e.g. in the case of a retaining wall design or downdrag on a pile. In such situations the designer assesses soil density, water pressures and ground shear strength to derive pressure distributions or shear forces.

2.11.3.3 Verification of EQU

For the verification of the EQU limit state the following inequality applies:

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

where:
$E_{dst;d}$ is the design value of the destabilising actions
$E_{stb;d}$ is the design value of the stabilising actions
T_d is the design value of the shearing resistance on the perimeter of a block of ground or structure being destabilised

In all instances the design values of action effects are functions of the effects of stabilising and destabilising actions ($\gamma_F \psi F_k$), material strengths (X_k/γ_M) and geometry ($a_{nom} \pm \Delta a$).

Partial factors are presented in Section 2.11.3.7.

2.11.3.4 Verification of STR and GEO

For the verification of the STR and GEO limit states the following inequality applies:

$$E_d \leq R_d$$

where:

E_d is the design value of the effect of actions
R_d is the design value of the resistance to the effect of actions

Partial factors γ and factors to convert characteristic values to representative values ψ are applied to either actions F or the effects of actions E, and partial factors γ are applied to material properties X as follows:

$$E_d = E\{\gamma_F \psi F_k ; X_k/\gamma_M ; a_{nom} \pm \Delta a\} \text{ or } E\{\psi F_k ; X_k/\gamma_M ; a_{nom} \pm \Delta a\}\gamma_E$$

Partial factors are applied to resistances R as follows:

$$R_d = R\{\gamma_F \psi F_k ; X_k/\gamma_M ; a_{nom} \pm \Delta a\}/\gamma_R$$

Within EC7, the verification of STR and GEO may be carried out in one of three different ways, namely Design Approaches 1, 2 and 3. The national standard body in each nation is required to decide which Design Approach to use and formalise this in their National Annex to EC7 Part 1[44]. In the UK National Annex, Design Approach 1 (DA1) is used. Each Design Approach applies different combinations of partial factors (see Section 2.11.3.7) to Actions A, Materials M and Resistances R.

Design Approach 1 for general design cases (other than piles) requires that the following two combinations are verified:
– DA1, Combination 1: A1 + M1 + R1
– DA1, Combination 2: A2 + M2 + R1

where:

A1 signifies Set 1 partial factors on actions
M2 signifies Set 2 partial factors on materials etc.

The value of the partial factor for resistance (Set R1) is associated with spread foundations, retaining structures and slopes/overall stability (earth resistance) and has a value of 1.0.

DA1 for the design of axially loaded piles (and also ground anchorages the design of which is under review, see Section 9.6) requires that the following

two combinations are verified:
– DA1, Combination 1: A1 + M1 + R1
– DA1, Combination 2: A2 + (M1 or M2) + R4

Specifically related to pile design 'R4' is Set 4 partial factors on pile resistances given in Table 7.11 (values greater than unity) with R1 factors given in Table 7.9 (values equal to unity).

For DA1 Combination 2 design of piles there are two options for the partial factor on materials, namely M1 or M2. Where the ground material strength provides resistance to the applied actions then value M1 is used. The resistance calculated is then factored down using Set R4 factors to obtain the design resistance. In contrast to this the Set M2 factors are used where the ground material strength is detrimental to pile performance as in the case of negative skin friction. In this instance Set M2 is used to increase the ground strength as the negative skin friction is a disturbing effect, and this action is then added to the externally imposed design action calculated using Set A2 factors.

In general, partial factors are applied to actions rather than the effects of actions for Combination 1. Application of partial factors to the effects of actions in Combination 1 may be appropriate where factoring of actions leads to unreasonable or physically impossible design values. For Combination 2, partial factors are always applied to actions.

Economy in terms of design effort can be achieved by undertaking the verification of one combination where it is obvious that it governs.

Background to DA1 and its two combinations can be found in Simpson[47].

2.11.3.5 Verification of UPL – failure by uplift

For the verification of the UPL limit state the following inequality applies:

$$G_{dst;d} + Q_{dst;d} \leq G_{stb;d} + R_d$$

where:

$G_{dst;d}$ is the design value of the destabilising permanent action
$Q_{dst;d}$ is the design value of the destabilising variable action
$G_{stb;d}$ is the design value of the stabilising permanent vertical actions for uplift verification
R_d is the design value of the resistance to an action (e.g. side shear on a basement box)

Partial factors are presented in Section 2.11.3.7.

2.11.3.6 Verification of HYD – failure due to moving groundwater

For the verification of the HYD limit state the following inequality applies:

$$u_{dst;d} \leq \sigma_{stb;d}$$

or

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

where:

$u_{dst;d}$ is the design value of the destabilising water pressure
$\sigma_{stb;d}$ is the design value of the stabilising total stress in the ground
$S_{dst;d}$ is the design value of the destabilising seepage force in the ground
$G'_{stb;d}$ is the design value of the stabilising permanent vertical actions for heave verification (submerged weight)

Partial factors are presented in Section 2.11.3.7. Exercise care when choosing where to apply the partial factors.

2.11.3.7 Partial factors

This section deals with partial factors on material strengths; partial factors used in the calculation of design bearing resistance of piles (axial loading) are presented in Chapter 7.

The UK National Annexes to EC0[42] and EC7 Part 1[44] provide partial factors for ultimate limit states EQU, STR/GEO, UPL and HYD. The partial factors for actions and materials are presented in Tables 2.6 to 2.9 for general design usage.

Table 2.10 provides values for combination factors ψ_0, ψ_1 and ψ_2 which are applied to variable actions when deriving combinations of actions.

Notes for Tables 2.6 to 2.10:

- The value of a partial factor is usually taken as being a multiplier for actions and a divider for material strengths. In certain situations the factor on material strength can be taken as a multiplier if this is more onerous (e.g. a pile being laterally pushed through soil).
- For accidental design situations the value of the partial factor for all unfavourable and permanent favourable actions in Eqn 2.2 is 1.0. For accompanying actions reduction factors are also used. For materials and resistances in accidental design situations, the partial factors γ on material strengths/resistances should be chosen according to the particular event being considered; possible use of Set M (EQU) for materials may be appropriate but is not included in EC7.
- When dealing with groundwater the design effect of groundwater can be obtained by defining the characteristic groundwater level and then applying a partial factor to the resulting pressure distribution (i.e. designing with the effects of groundwater pressures). It is also acceptable to choose the design water level directly and forego use of the partial factors. When choosing a design water level directly, the appropriate partial factor together with an assessment of the characteristic water pressure may provide a guide to the choice of the directly selected design water level. See Chapter 4 for further comment.
- Where overturning instability of a structure could occur without the resistance of the ground being exceeded (e.g. on rock foundation), the tabulated partial factors (EQU, STR/GEO) can result in a foundation with seemingly small dimensions that may be out of keeping with pre-EC7 design in the UK. In such situations, consideration may be given to higher partial factors as part of the design to EC7.
- In cases where favourable and unfavourable permanent actions can be traced to a single source, a single partial factor should be applied to both favourable and unfavourable components of the action. It will be necessary to check whether the unfavourable or favourable partial factor is more onerous. This principle is commonly employed when assessing the action of groundwater and self-weight of the ground (see Section 2.10.2).
- Less severe values of the tabulated partial factors may be used for temporary/transient design situations where the likely consequences justify it. However, agreement with the client and relevant authorities may be required. Furthermore, parameters used for temporary design situations may also be assessed to be different to those in the permanent design situation.

Table 2.6a EQU partial factors – actions (for design of buildings)

Set	Permanent actions		Variable actions		
	Unfavourable $\gamma_{G,j}$	Favourable $\gamma_{G,j}$	Unfavourable (leading) $\gamma_{Q,1}$	Unfavourable (accompanying) $\gamma_{Q,i}$	Favourable $\gamma_{Q,1}$ or $\gamma_{Q,i}$
A (EQU)[a] For use with Eqn 2.1[b]	1.1	0.9	1.5	1.5	0.0
Alternative[c]	1.35	1.15	1.5	1.5	0.0

Notes

a Terminology is taken from EC0 but is not included in EC7.
b Set A in EC0.
c Possible alternative to separate verification of EQU and STR/GEO provided that partial factors of unity applied to permanent actions do not yield a more onerous outcome under EQU. The partial factors are applied to STR and GEO limit states as an alternative to Combination 1 verifications.
d Factors from NA to EC0[42] Table NA.A1.2 (A).

Table 2.6b EQU partial factors – ground properties

Set	Materials			
	$\gamma_{\varphi'}$	$\gamma_{c'}$	γ_{cu}	γ_{qu}
M (EQU)[a]	1.1[b]	1.1	1.2	1.2

Notes

a Terminology is taken from EC0 but is not included in EC7.
b Applied to tan φ'.
c Factors from NA to EC7 Part 1[44] Table A.NA.2.

Table 2.7a STR/GEO partial factors – actions (for design of buildings)

Set	Comment	Permanent actions (or effects)		Variable actions (or effects)		
		Unfavourable $\gamma_{G,j}$	Favourable $\gamma_{G,j}$	Unfavourable (leading) $\gamma_{Q,1}$	Unfavourable (accompanying) $\gamma_{Q,i}$	Favourable $\gamma_{Q,1}$ or $\gamma_{Q,i}$
A1[a]	For use with Eqn 2.1	1.35	1.0	1.5	1.5	0.0
	For use with Eqn 2.1a	1.35	1.0	1.5	1.5	0.0
	For use with Eqn 2.1b	$\xi_j\gamma_{G,j} = 1.25$[c]	1.0	1.5	1.5	0.0
A2[b]	For use with Eqn 2.1	1.0	1.0	1.3	1.3	0.0

Notes

a Set B in EC0.
b Set C in EC0.
c $\xi_j = 0.925$.
d Factors from NA to EC0[42] Tables NA.A1.2 (B) and NA.A1.2 (C).

Table 2.7b STR/GEO partial factors – ground properties

Set	Materials			
	$\gamma_{\varphi'}$	$\gamma_{c'}$	γ_{cu}	γ_{qu}
M1	1.0[a]	1.0	1.0	1.0
M2	1.25[a]	1.25	1.4	1.4

Notes

a Applied to tan φ'.
b Factors from NA to EC7 Part 1[44] Table A.NA.4.

Table 2.8a UPL partial factors – actions (for design of buildings)

Set	Permanent actions		Variable actions		
	Unfavourable $\gamma_{G,j}$	Favourable $\gamma_{G,j}$	Unfavourable (leading) $\gamma_{Q,1}$	Unfavourable (accompanying) $\gamma_{Q,i}$	Favourable $\gamma_{Q,1}$ or $\gamma_{Q,i}$
A (UPL)[a] For use with Eqn 2.1	1.1[b]	0.9	1.5	1.5	0.0

Notes
a Terminology is taken from EC0 but is not included in EC7.
b Does not account for uncertainty in the level of groundwater or free water. This may be significant where a structure is partially submerged. It may be more appropriate to directly assess the design value or apply a safety margin to the characteristic value rather than blindly factor the characteristic water level. See Chapter 4 for further information.
c Factors from NA to EC7 Part 1[44] Table A.NA.15.

Table 2.8b UPL partial factors – ground properties

Set	Materials			
	$\gamma_{\varphi'}$	$\gamma_{c'}$	γ_{cu}	γ_{qu}
M (UPL)[a]	1.25[b]	1.25	1.4	–

Notes
a Terminology is taken from EC0 but is not included in EC7.
b Applied to tan φ'.
c Factors from NA to EC7 Part 1[44] Table A.NA.16.

Table 2.9 HYD partial factors – actions (for use in Eqn 2.1)

Permanent actions		Variable actions		
Unfavourable $\gamma_{G,j}$	Favourable $\gamma_{G,j}$	Unfavourable (leading) $\gamma_{Q,1}$	Unfavourable (accompanying) $\gamma_{Q,i}$	Favourable $\gamma_{Q,1}$ or $\gamma_{Q,i}$
1.35	0.9	1.5	1.5	0.0

Note
Factors from NA to EC7 Part 1[44] Table A.NA.17, where $\gamma_{G,j} = 1.335$.

Table 2.10 Factors for variable loads – combination factors

Imposed loads in buildings[a]	ψ_0	ψ_1	ψ_2
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	1.0	0.9	0.8
Category F: traffic area: vehicle $\leq$ 30kN	0.7	0.7	0.6
Category G: traffic area: 30kN $<$ vehicle $\leq$ 160kN	0.7	0.5	0.3
Category H: roofs	0.7	0	0
Snow loads: altitude $>$1000m asl	0.7	0.5	0.2
Snow loads: altitude $\leq$1000m asl	0.5	0.2	0
Wind loads on buildings	0.5	0.2	0
Temperature (non-fire) in buildings	0.6	0.5	0

Notes
a See also EC1[4].
b Factors from NA to EC0[42] Table NA.A1.1.

2.11.4 Serviceability limit states

2.11.4.1 Introduction

Serviceability limit states are those concerned with the day-to-day functioning of a structure, the comfort of people using the structure and how the structure appears. A distinction must be made between reversible and irreversible states (often referred to frequent and infrequent load combinations); see Table 2.11 where the consideration for each state is provided.

For each serviceability limit state it is verified that either:

$$E_d \leq C_d$$

or

that a sufficiently small amount of the ground strength is mobilised, provided that a value of deformation is not required and there is comparable experience

where:
E_d is the design value of the effect of an action (e.g. settlement)
C_d is the limiting (or acceptable) design value of the effect of an action (e.g. settlement)

Verification of the serviceability limit states is based on criteria relating to deformation, vibration and damage. Deformation and vibration are defined in

Table 2.11 Examples of serviceability limit states

Serviceability limit state	Example criterion
Water ingress into basement	Basement grade[a]
Floor deflection	Limit on vertical deflection
Footing movement	Limit on differential settlement
Vibration from adjacent tunnel	Limit on vibration dose value for humans[b]
Cracks in façade	Limit on crack width and angular distortion resulting from foundation movement
Retaining wall deflection causing cracking in external roads or adjacent buildings	Limit on construction induced movements

Notes
a CIRIA Report 139[48] discusses basement grades.
b BS 6472[49] provides guidance.

terms of intended use, in relation to the limit state being considered. Definition of damage should consider its effect on durability as well as aesthetics.

In the case of finishes to a structure (e.g. cladding), it is reasonable to assess only those actions or effects of actions (e.g. settlement) which will occur after the finish is fixed to the structure when checking if the limit state is exceeded. Such considerations should include time dependent behaviour of the ground where settlement continues long after the application of an action.

Serviceability criteria are formulated for each project and agreed with the client and relevant authorities.

The value of partial factors γ are usually taken as 1.0.

Examples of serviceability limit states and criteria are included in Table 2.11.

2.11.4.2 Combination of actions

Design value of the effect of actions E_d
E_d is determined for a particular design situation on the basis of a relevant combination of actions which should be appropriate to the serviceability criterion being verified. Actions resulting from imposed deformations are to be considered where relevant. The combinations in Table 2.12 can be used (these combinations are from EC0, Clause 6.5.3[3]).

The recommended values of characteristic actions (surcharge pressures, densities etc.) are provided in EC1[4]. Actions generated in the ground are addressed in EC7.

When calculating a value for E_d in terms of differential movement of foundations, the following are to be taken into account:
– the occurrence and rate of settlements and ground movements
– variability of ground conditions

Table 2.12 Serviceability load combinations

Combination	Normal/intended use
Characteristic: $\sum_{j \geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i > 1} \psi_{0,i} Q_{k,i}$	Function and damage to structural/ non-structural members (Irreversible limit states)
Frequent: $\sum_{j \geq 1} G_{k,j} + P + \psi_1 Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i}$	User comfort, use of machinery, etc. (Reversible limit states)
Quasi-permanent: $\sum_{j \geq 1} G_{k,j} + P + \sum_{i \geq 1} \psi_{2,i} Q_{k,i}$	Long-term effects and appearance of the structure

– load distribution
– method and sequence of construction
– stiffness of the structure.

Limiting design value of the effect of actions C_d
For the design of foundations, the value of C_d should be established for foundation movement. Settlement (total and relative), rotation, tilt, relative deflection, relative rotation, horizontal displacement and vibration amplitude should be considered.

When selecting a value for C_d for foundation movement the following should be considered:
– the confidence with which the value can be specified
– the occurrence and rate of ground movements
– the type of structure, construction material, foundation and ground
– the mode of deformation
– the proposed use of the structure
– utility movement limit criteria.

Movement/deformation of foundations resulting in deformation of a supported structure is to be limited so that a limit state is not exceeded in the structure as a result of the movement/deformation of the foundation. Limiting design values for movement/deformation are to be agreed during the design of the supported structure with the client and relevant authorities. Where appropriate movement in the structure should be added to movement generated in the ground.

In the absence of specified limiting values the following values given in Annex H to EC7 Part 1[1] may be used for normal, routine and relatively uniformly loaded structures:
– Maximum acceptable relative rotation ranges from 1/300 to 1/2000 with a commonly acceptable value of 1/500 in sagging mode.

– Maximum acceptable relative rotation ranges from 1/600 to 1/4000 with a commonly acceptable value of 1/1000 in hogging mode.
– Normal structures with isolated foundations can often accept total settlements of up to 50mm (or greater if rotation and tilt remain acceptable and utility/service connections are not significantly affected). It is necessary to check that differential settlements are acceptable, i.e. a differential settlement of 50mm between columns would likely be unacceptable.

Further discussion of serviceability limit states is presented in Section A1.4 of EC0[3].

2.12 Design by prescriptive measures

This method of design may be used where there are no appropriate calculation models or where calculation is not necessary or generally not possible. It may also be suitable for some aspects of durability design for which calculations are not appropriate.

Design by this method should involve comparable experience, conventional approaches and conservative design rules. The specification and control of materials, workmanship, durability and maintenance should also form part of the design. Examples of design by this method include:
– the use of presumed bearing resistance tables/charts for shallow footings such as those presented in the Building Regulations
– use of standard details.

The use of this method should be agreed, where appropriate, with the client and relevant authorities.

2.13 Design by the observational method

The observational method of design can be appropriate where a range of ground properties are plausible, thus enabling possible economies in construction cost to be achieved by means of monitoring and, where appropriate, modifying the construction sequence. The implementation of the observational method is a specialised area and it will only be dealt with briefly in this *Manual*.

This method requires that the designer considers a range of possible behaviours from which a range of construction strategies are developed. As construction progresses monitoring is undertaken, the results are compared with predicted values and action plans are implemented as appropriate. Within the remit of structures covered by this *Manual*, it is most usually applied to temporary works associated with retaining walls where the level and number of props may be altered to ease construction.

The following requirements must be met:

Before construction:
– Establish acceptable limits of behaviour.
– Assess the range of possible behaviour and show that the acceptable probability of the actual behaviour falling within this range.
– Prepare a monitoring plan that is capable of showing whether the actual behaviour is within acceptable limits in such a manner that contingency plans can be implemented successfully.
– Ensure that the response time of monitoring equipment and procedures for analysing the results are rapid enough to cope with the on-going behaviour of the structure.
– Prepare contingency plans that can be implemented when behaviour of the structure is outside acceptable limits.

During construction:
– Undertake monitoring as planned.
– Assess monitoring results and implement contingency plans if acceptable limits of behaviour are exceeded.
– Replace or extend monitoring equipment if the data is unreliable, not appropriate or too infrequent.

Guidance on the observational approach can be found in CIRIA Report No. 185[50].

The use of this method should be agreed, as appropriate, with the client and relevant authorities.

2.14 Design based on experimental models or load tests

Load tests and scale model testing (in contrast to theoretical derivation) may be used to justify a particular design or to complement other methods of design such as those discussed above. A sample of the actual construction, a part scale model or a full scale model of the actual construction may be used.

Testing may be used to justify lower 'model factors' or partial factors. The main example of this is in the design of piles where preliminary pile load testing may be used to reduce the magnitude of the model factor and where load tests on working piles may be used to reduce the magnitude of partial factors used to calculate the design resistance from the characteristic resistance.

In order to use the results from testing in design, the differences between the test and actual conditions in terms of ground and groundwater conditions, duration and scale must be considered and incorporated.

Further guidance on classifying tests, planning testing and deriving parameters can be found in Annex D of EC0[3] and also Clause 7.5 of EC7 Part 1[1].

2.15 Summary

- Each site, structure or part of a structure needs to be assigned a Geotechnical Category.

- Reliability, durability and design life are key considerations.

- Design to EC7 must explicitly consider both the ultimate limit state (ULS) and the serviceability limit state (SLS) unless it can be shown that one is satisfied when considering the other; this usually is only possible for relatively simple structures and design situations.

- Limit states are verified for each design situation.

- Design may be carried out using variable levels of sophistication. Where ground conditions and loading conditions are relatively simple it is possible to design using prescriptive measures (well defined 'rules of thumb'); such design would usually incorporate both ULS and SLS requirements. For more complicated situations consideration of ULS and SLS by calculation (comparing action effects and resistances) is usually required.

- Design by calculation uses the partial factor method. This requires factoring of material properties and actions when deriving design actions and resistances.

- ULS design by calculation includes five sets of ULS partial factors, all must be satisfied by:
  EQU For consideration of stability of a structure with emphasis on geometry above geotechnical or structural material strength.
  STR For consideration of stability with emphasis on structural material strengths.
  GEO For consideration of stability with emphasis on geotechnical material strengths.
  UPL For design against uplift generated by water or heave pressures.
  HYD For design of stability of structures against the flow of water.

  For design in the UK STR/GEO follow Design Approach 1. STR and GEO limit states both need to satisfy Combinations 1 and 2.

- SLS design must consider the function of the building and the appearance of the building.

3 Geotechnical investigation and reporting

3.1 General

3.1.1 Scope

The purpose of this chapter is to provide an introduction to geotechnical investigation and reporting. The chapter is not an exhaustive discussion on geotechnical investigations but contains adequate details to provide the reader with an appreciation of the content of EC7. It is expected that non-geotechnical specialist engineers will consult with a suitably experienced geotechnical engineer prior to commissioning a ground investigation or report. Further information can be found in EC7 Part 2[2].

> “you pay for a site investigation whether you have one or not”[51]

3.1.2 Ground properties

In Section 2.10 the concept of the process of obtaining geotechnical measurements and converting them for use in design was introduced. The aim of geotechnical investigation is to obtain information at the first tier: test results and other relevant data. See also Figure 3.1.

3.1.3 Planning

The general aims of geotechnical investigation are:
– to ensure that relevant information and data are available when needed and are adequate to manage risks
– to establish ground and groundwater conditions/properties and collect other relevant data
– to confirm or invalidate the selected Geotechnical Category.

A phased approach to geotechnical investigation comprised of the following parts should be adopted:
– preliminary ground investigation
– design ground investigation
– control ground investigation.

Geotechnical investigation at each phase is to:
– be commensurate with the anticipated type and design of construction
– take into account the findings of earlier phases of investigation

– take into account the anticipated ground conditions
– provide information on ground and groundwater relevant to the design and construction of the proposed works
– be flexible with an ability to adapt as new information becomes available
– be undertaken and reported in accordance with internationally recognised standards.

An appropriate quality assurance system must be established.

Whilst not addressed in EC7 the following is advised. All ground investigations should be carried out in a safe manner. Part of this includes the pre-assessment of the ground conditions at the site in terms of contamination as it may impact on the ground investigation operatives and supervisory staff and on the environment (e.g. aquifers). At an early stage in GI planning the historical data on the site must be used to assess contamination and other risks to allow appropriate site procedures to be put in place to manage these risks. The site is often categorised as 'green', 'yellow' or 'red' as a function of contamination risk and the need to introduce increasingly rigorous levels of contamination management. As with all risk based approaches, if conditions other than those assumed are found the risk assessment must be reassessed. Equally important is consideration of overhead and buried services.

3.1.4 Responsibilities

Though not discussed in EC7, it is helpful to provide some recommendations relating to the division of tasks with respect to ground investigation at the preliminary and design stages such that appropriate control and continuity is maintained. This is presented in Table 3.1.

Table 3.1 Guidance for ground investigation responsibility (not EC7)

Stage	Task	GC1	GC2	GC3
Preliminary investigation	Desk study	D or C	D	D
	Specification of ground investigation	D or C	D	D
	Supervision of ground investigation	C	D or C	D
	Factual report	D or C	D or C	D or C
Design investigation	Specification of ground investigation	D or C	D	D
	Supervision of ground investigation	C	D or C	D
	Factual report	C	C	C
Ground Investigation Report (i.e. factual report and evaluation)		C	D or C	D

Note
GC = geotechnical category (see Section 2.9), D = designer, C = ground investigation contractor.

3.2 Specification

Specification is discussed in Chapter 10.

3.3 Preliminary investigation

3.3.1 General

A preliminary investigation is required for all works where design to EC7 is carried out. The objectives are:
– to assess site suitability
– to assist feasibility studies
– to assess development effects
– to assist planning of further investigation
– to identify sources of material
– to identify possible foundation solutions
– to identify problems.

3.3.2 Desk study and site walkover

A desk study (see Section 2.7 of EC7 Part 2[2]) is the first task in geotechnical investigation and comprises the collection of all relevant available information such as: topography, geology, geomorphology, stratigraphy, groundwater, strength/stiffness, contamination, historical development and hazards. This information may be obtained from a number of sources such as geological memoirs and data from geological societies/bodies, mapping bodies, local libraries, commercial data providers, government bodies, and publications. Desk studies also may include information on other ground related topics such as archaeology and underground utilities.

The desk study should be complemented by a site walkover (field reconnaissance) which will allow collection of data not typically available through the desk study, such as: condition of existing structures, questioning of residents, site access, evidence of groundwater and evidence of instability.

In order to obtain the full benefit of a desk study it should be undertaken as early as possible in a project's timeline.

Further guidance on sources of information for desk studies can be found in TRL Report 192[52].

3.3.3 Intrusive investigation

Should desk study investigation provide insufficient information in the early stages of a project, it may be necessary or prudent to undertake an initial phase of intrusive investigation.

Investigation could comprise of boreholes, probing or trial pits and be broad and general or local and specific. However, the scope should reflect the project's risk profile and be limited to the collection of information that is necessary at the project's preliminary stage.

3.4 Design investigation

3.4.1 General

Design investigation is required and the objectives are to:
– provide information to satisfy the preliminary investigation objectives
– provide information for adequate design of temporary and permanent works including method of construction
– identify potential construction difficulties
– satisfy party wall matters where relevant.

3.4.2 Investigation

3.4.2.1 Introduction

Investigation, though not necessarily intrusive, is to:
– identify the nature and properties of all relevant ground strata and groundwater
– establish parameters necessary for design prior to the start of detailed design
– be carried out at least through the geological formations assessed to be relevant
– establish the location and capacity of nearby groundwater abstraction/ dewatering wells
– comprise drilling, excavation, measurement and testing, if relevant.

3.4.2.2 Programme

A field investigation programme should contain information on the required investigation points, sampling, testing, measurement, equipment and standards.

3.4.2.3 Investigation points

The location, spacing and depth of investigation points should take into account the preliminary investigation data and be appropriate to the geological conditions, the size of the structure, the engineering problems involved and the extent of local comparable experience. The following issues should be kept in mind with respect to the location of investigation points:
– The ability to assess stratigraphy and variations in stratigraphy, and groundwater will be dependent upon the distribution of investigation points.
– Appropriate distribution of investigation points across a structure will facilitate design.
– Offsetting of investigation points on linear structures is necessary to provide a three dimensional picture of the ground and groundwater.
– Where topography is sloping or steps, investigation points outside the slope or step will facilitate assessment of overall stability.

– Inappropriately positioned investigation points may pose a hazard to the completed structure (e.g. trial pits which are excavated below formation level of spread foundations in competent strata).
– Investigation points at a distance from the immediate structure being considered will facilitate the assessment of how the structure affects surrounding property.
– If groundwater investigation points are durable and accessible beyond the design investigation they will assist in construction and future monitoring.

The minimum spacing of investigation points for typical buildings should be a grid pattern of 15 to 40m and for linear structures a spacing of 20 to 200m.

The minimum depths for investigation points are presented in Table 3.2; in some instances the proposals in EC7 Part 2[2] have been simplified, with Table 3.2 suggesting marginally deeper investigations. When choosing the depth of investigation, requirements for temporary works should also be considered.

3.4.2.4 Sampling

The purpose of sampling is to obtain ground or groundwater for identification, description, laboratory testing and retention. The scope of sampling is dependent upon the aims of the geotechnical investigation, the geology and the complexity of the proposed development. However, at least one borehole or trial pit must be undertaken with appropriate sampling.

Soil and rock samples fall into one of five quality classes (1 is high quality, 5 is low quality) depending upon the method of collection and storage. The soil or rock properties that can be determined from a particular sample are determined by the quality class of that sample. The relationships between quality classes, illustrative soil/rock properties and common sampling techniques are shown in Table 3.3.

Groundwater samples can be collected from exploratory holes and standpipes. Methods of extraction include: pumping, water samplers or vacuum bottles.

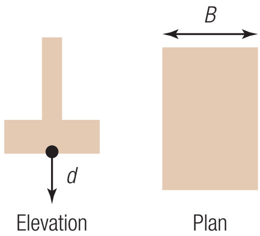
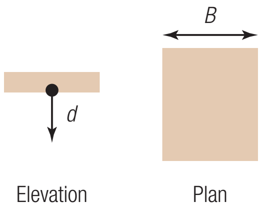
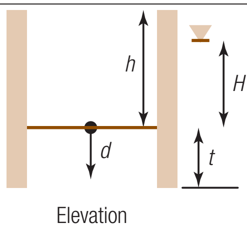
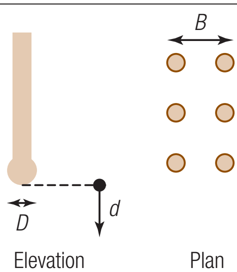
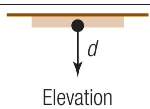
Further information on soil/rock sampling methods and groundwater sampling/measurement can be found in EN ISO 22475-1:2006[24].

3.4.2.5 Laboratory testing

Laboratory measured values should be compared with expected values. The expected test values should be based on experienced inspection of samples, tactile testing and review of field logs in light of the geological setting and comparable experience. Blind reliance on laboratory tests results should not be made.

The scope of laboratory testing will be dependent upon ground homogeneity, comparable experience, design method and type of construction.

Table 3.2 Ground investigation geometry

Structure	Definitions	Minimum depth[a] (m)
Shallow footing		$d \geq \max(6, 3B)$
Raft		$d \geq 1.5B$
Excavation		If $H < 0$, $d \geq (0.4h, t + 2)$ If $H \geq 0$, $d \geq \max(H + 2, t + 5)$[b]
Pile		$d \geq \max(B, 5, 3D)$ where: B is the width of the pile group D is the pile base diameter
Road		$d \geq 2$

Notes

a For large buildings deeper investigations will be required. For competent rock strata at formation level a reduction in depth of some investigation points may be possible. For soft strata at formation level deeper investigation will likely be required. These are guide values only.

b Simplified, based on Annex B of EC7 Part 2[2].

Table 3.3 Sampling rules

Quality class	1	2	3	4	5
Ground property that can be determined (non-exhaustive):					
Layer sequence	✓	✓	✓	✓	✓
Strata boundaries (broad)	✓	✓	✓	✓	–
Strata boundaries (fine)	✓	✓	–	–	–
Grading	✓	✓	✓	✓	–
Atterberg limits, moisture content	✓	✓	✓	✓	–
Density, permeability	✓	✓	–	–	–
Compressibility, shear strength	✓	–	–	–	–
pH, sulphate, chloride, carbonate, organics	✓	✓	✓	✓	✓
Compaction	✓	✓	✓	✓	✓
Quality delivered – sampling by drilling (non-exhaustive):					
Triple-tube rotary coring (cohesive soil)	✓	–	–	–	–
Double-tube rotary coring (rock)	✓		–	–	–
Hammer driven clay cutter (cohesive soil)	–	✓	–	–	–
Hammer driven clay cutter (non-cohesive soil)	–	–	✓	–	–
Double-tube rotary coring (soil)	–	–	✓	–	–
Cable percussion/shell and auger	–	–	–	✓	–
Auger	–	–	–	✓	
Quality delivered – sampling by sampler (non-exhaustive):					
Thin-walled sampler	✓	–	–	–	–
Thick-walled piston sampler	–	✓	–	–	–
U100 (thick-walled open-tube sampler)	–	–	✓	–	–
SPT (Standard Penetration Test) sampler	–	–	–	✓	–
Window sample	–	–	–	–	✓

Note
The sampling methods described above have been abbreviated. Reference should be made to EN ISO 22475-1:2006[24] for specific details.

Samples for laboratory testing need to be representative of the strata being examined. In order to satisfy this requirement (and in addition to field logging) common classification tests (as shown in Table 3.4) can be used to characterise strata. Samples should be selected so that they are evenly distributed throughout the strata being considered.

A broad classification of geotechnical tests for the derivation of soil parameters that provide measurement of strength and stiffness are presented in Table 3.5.

Table 3.4 Summary of typical laboratory tests for index properties

Test	Soil			Rock
	Clayey	Silty	Sandy/ Gravelly	
Water content	✓	✓	–	✓
Bulk density	✓	✓	–	✓
Min/max density	–	–	✓	–
Atterberg limits	✓	✓	–	–
Grading	✓	✓	✓	–
Undrained shear strength	✓	–	–	–
Permeability	✓	✓	–	–
Sensitivity	✓	✓	–	–
Porosity	–	–	–	✓
Unconfined compressive strength	–	–	–	✓
Young's modulus and Poisson's ratio	–	–	–	✓
Point Load Index	–	–	–	✓

Suggestions for the minimum number of tests that may be appropriate for one soil stratum are presented in Table 3.6 based on Annexes of EC7 Part 2[2]. Designers should decide whether these are sufficient for their particular project as it is noted that the lack of definition of a 'stratum' means that a thick 'stratum' would be severely under investigated. For example, a site underlain by a large thickness of London Clay would usually require more than one to three undrained shear strength tests.

Table 3.5 Summary of typical laboratory tests for strength and stiffness parameters

Parameters	Soil test			
	Clay	Silt	Sand	Gravel
Compressibility (m_v, C_c, C_s, c_v, λ, κ)	OED	OED	–	–
Stiffness (E, G, G_{max})	TX	TX	TX	TX
Undrained shear strength (c_u)	TX, DSS	TX, DSS	–	–
Drained shear strength (c', φ')	TX, SB	TX, SB	TX, SB	TX, SB
Residual strength (c'_r, φ'_r)	RS	RS	RS	RS

Notes

a TX = triaxial, SB = shear box, OED = oedometer, RS = ring shear box (or reverse SB), DSS = direct simple shear.

b E and G may vary with both stress and strain magnitudes.

c Taken from Table 2.3 EC7 Part 2[2].

Table 3.6 Summary of suggested (minimum) testing frequency per stratum

Test	Minimum suggested testing frequency per stratum based on knowledge/experience of stratum	
	No experience[a]	Extensive experience[a]
Grading	4–6	2–4
Water content	All samples of quality class 1 to 3	
Density	3–4	1–2
Undrained shear strength (c_u)	3–6	1–3
Drained shear strength (φ')	2–4	1–2
Permeability	3–5	1–3
Unconfined compressive strength	2–6	0–2
Compressibility (E_{oed})	2–4	1–2
Atterberg limits	3–5	1–3

Note

a These are based on Annex M of EC7 Part 2[2] recommendations. It is considered that they are an absolute minimum and in nearly all instances more tests would be carried out especially where strata are more than a few metres thick.

3.4.2.6 *In situ* testing

Field testing may be used to replace sampling and laboratory testing provided they are calibrated against boreholes, trial pits or appropriate local experience. The choice of field test will be dependent upon the geology, the construction, the design method and the geotechnical parameter desired. An overview of the applicability of common *in situ* tests to the determination of various geotechnical parameters is presented in Table 3.7.

3.5 Control investigation

Control investigation is discussed in Chapter 10.

3.6 Ground investigation specification

The preparation of a specification for ground investigation will use the information above to define an appropriate scope (types of investigation, spacing and depth of investigation points, and appropriate laboratory testing). The specification is only part of the requirements of a tender package; the complete package of documents usually includes the following elements:

– Short description of works.
– Form of tender and appendices.
– Standard conditions of contract and amendments.
– Specification (technical requirements).
– Schedules (employer's requirements etc.).
– Bill of quantities with preamble.
– Technical annexes.

There are two approaches that can be taken when procuring a ground investigation. The first option is for the engineer to provide a designed ground investigation and request the GI contractors to complete the rates column in the bill of quantities and form of tender; a 'Full Specification' approach. Alternatively the engineer can provide a full description of the proposed development works and require the GI contractor to fill in both quantities and rates in the bill of quantities as well as the form of tender document; a 'Performance Specification' approach. These two approaches are summarised in Table 3.8.

Table 3.7 Summary of typical *in situ* tests

In situ test	Soil/rock type			Layer identification			Shear strength			Compressibility			Permeability			Density			Pore-water pressure		
	F	C	R	F	C	R	F	C	R	F	C	R	F	C	R	F	C	R	F	C	R
Cone penetration test	1	1	3	1	1	–	1	2	–	2	1	–	2	3	–	2	2	–	2	2	–
Pressuremeter test	3	3	3	3	3	3	1	1	–	1	1	–	3	–	–	–	–	–	3	–	–
Flexible dilatometer test	3	3	2	3	3	3	–	–	–	1	–	1	–	–	–	–	–	–	–	–	–
Flat dilatometer test	2	2	–	1	2	–	1	2	–	1	2	–	–	–	–	2	2	–	–	–	–
Standard penetration test	1	2	–	2	2	–	3	2	–	2	2	–	–	–	–	2	2	–	–	–	–
Dynamic probing	3	3	–	2	1	–	3	2	–	2	2	–	–	–	–	–	2	–	–	–	–
Vane shear test	–	–	–	–	–	–	1	–	–	–	–	–	–	–	–	–	–	–	–	–	–
Plate load test	–	–	–	–	–	–	1	1	2	1	1	–	–	–	–	–	–	–	–	–	–
Permeability test	–	–	–	–	–	–	–	–	–	–	–	–	1	1	1	–	–	–	–	–	–

Notes
a F = fine grained (clay/silt), C = coarse grained (sand/gravel), R = rock.
b 1 = high applicability, 2 = medium applicability, 3 = low applicability.
c Taken from Table 2.1 EC7 Part 2[2].

Table 3.8 Ground investigation procurement routes

Documentation	Full specification	Performance specification
Short description of development works and proposed investigation	General details provided	Full development details provided including location of proposed buildings and details (loads, basement depth etc.)
Form of tender and appendices	Full contract details provided	Full contract details provided
Conditions of contract and amendments to conditions of contract	Full details provided	Full details provided
Specification (technical requirements)	Full details provided	None
Schedules	– Site constraints provided – 3rd party restrictions provided (e.g. interaction with services, adjacent land owners, environmental restrictions) – Engineers/employers facilities stated – Particular technical requirements peculiar to the project	– Site constraints provided – 3rd party restrictions provided (e.g. interaction with services, adjacent land owners, environmental restrictions) – Engineers/employers facilities stated
Bill of quantities with preamble	Full details provided	Full details in preamble and base bill of quantities with anticipated types of investigation
Technical annexes	Full details provided	Limited
Approach to tender	Check for conforming tender turns	For competitive tenders the performance specification approach is unlikely to reward the diligent contractor and it is more likely to result in inadequate site investigation data

It is noted that the Performance Specification route may not reward diligent contractors and may result in inadequate site investigation data. Furthermore, it should be recognised that the cheapest tender return for either route may not offer the best value for money or the best value for the project.

The documents in Table 3.8 can be prepared using existing reference information as given in Table 3.9.

It is critical to note that use of the Performance Specification route should be done in a manner in which the GI contractor demonstrates that their proposal is

Table 3.9 Ground investigation documentation

Documentation	Resource
Description of development works and proposed investigation	Provide in covering letter
Form of tender and appendices	ICC Infrastructure Conditions of Contract: Ground Investigation Version (2011)[53] Amendments may be required to maintain the conditions of contract up to date with legislation and codes of practice as they evolve, as would be covered in amendments to conditions of contract
Conditions of contract	
Specification (technical requirements)	Site Investigation in Construction – UK Specification for Ground Investigation (2012)[54]
Schedules	
Bill of quantities with preamble	
Technical annexes	

compliant with the requirements of EC7 Parts 1[1] and 2[2]. An inadequate ground investigation with a low price should never be seen as value for money:

> "the bitter taste of poor quality remains much longer than the sweet taste of a low price".

The final comment on specification and scoping of ground investigation is related to price certainty, or lack of it. Ground investigation is by its nature looking for the unknown based on a view of what is likely. It is reasonable to assume that as the investigation proceeds unexpected conditions will be revealed which require further investigation, in order to adequately inform the ground model. Such additional investigation work will result in additional cost and may also result in extensions of programme. These uncertainties are usually accommodated by having a schedule of rates and illustrative quantities allowing a re-measured final cost to be arrived at, along with a contingency sum (typically 10% of the tender sum) to accommodate small changes in scope. Programme changes can be accommodated by the allowance of extensions of time or mobilisation of additional plant. The cost mechanism allows for the correct price to be paid for the work carried out while the contingency sums protect the client from the inconvenience of small cost over-runs. However, where significant unknown features are found during the investigation works it is likely that costs will significantly exceed the contract sum. At the limit, additional investigations (with associated remobilisation costs) may be needed. Such risk should be explained to the client at the start of the ground investigation process.

3.7 Ground Investigation Report

3.7.1 General

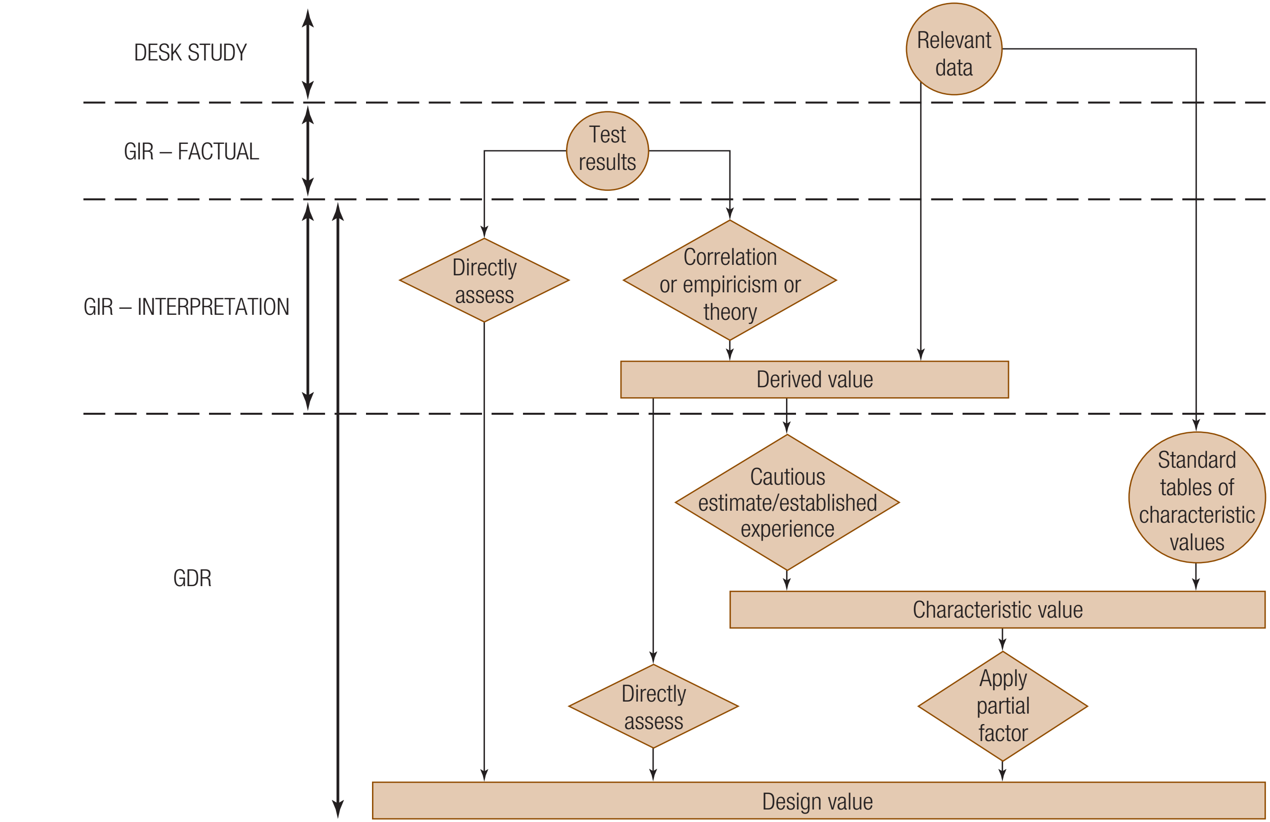
The Ground Investigation Report (GIR) records the results of ground investigation, forms part of the Geotechnical Design Report (GDR) (see Section 3.8) and contains the following:
– a presentation of appropriate and relevant geotechnical data
– a geotechnical evaluation of the data, stating any assumptions and known limitations
– proposals for necessary further work including justification and programme.

While the Ground Investigation Report may include derived values, it may be appropriate to only present test results. Figure 3.1 illustrates a possible division of tasks with respect to the presentation of ground parameters based upon the Ground Investigation Report having two sections: a factual section written by the ground investigation contractor and a derivation section written by the designer (or the ground investigation contractor if appropriate). Alternatively, the derivation section could be deferred to the Geotechnical Design Report discussed below (Section 1.6 of EC7 Part 2[2]).

3.7.2 Presentation

The presentation of geotechnical investigation information in the Ground Investigation Report includes:
– a factual account of all field and laboratory investigations
– documentation of the methods, procedures and results
– reporting in accordance with relevant EN and ISO standards.

In addition, the following are included as relevant:
– the purpose, scope and dates of investigation
– names of relevant parties
– field reconnaissance notes
– site history, geology
– survey data, aerial photograph information
– local experience
– sample handling procedures
– equipment used
– quantities of field and lab work
– supervisor field observations
– groundwater measurements
– borehole logs, core photographs, subsurface formation descriptions
– presence/possibility of radon
– frost susceptibility data
– field and laboratory results in appendices.



Note GIR – Geotechnical Investigation Report, GDR – Geotechnical Design Report

Fig 3.1 Reporting of ground parameters

3.7.3 Evaluation

The evaluation of geotechnical investigation information and its documentation in the Ground Investigation Report includes, as appropriate:
– an evaluation of field and laboratory results
– a review of the field and laboratory results and other information (including consideration of limitations in the data and the effect of sample treatment)
– detailed description of the strata including physical properties, stiffness and strength
– review of derived values
– comments on anomalies
– assumptions made during interpretation of data (e.g. sampling method, groundwater level)
– differences between desk study findings and field/laboratory investigations
– proposals for further field and laboratory work including justification, programme and aims
– tables and graphs of results
– tables of geotechnical parameters for each stratum (derived or other)
– geotechnical sections
– correlations and their applicability, when used to establish derived values
– appropriate treatment of outliers (data points outside the general scatter which may be of significance) in the data
– comparison of results with published values and investigation of anomalies
– justification for merging strata with similar characteristics or averaging ground that varies significantly due to fine layering
– justification for using linear interpolation between investigation points.

3.8 Geotechnical Design Report

3.8.1 General

Designers are required to prepare a Geotechnical Design Report (GDR) which should include where appropriate:
– assumptions made
– data used
– methods of calculation adopted
– results of safety and serviceability verification
– a plan for construction supervision, maintenance and monitoring
– cross-referencing to the Geotechnical Interpretative Report and other documents containing more detail
– a description of the site, its surroundings and the ground conditions
– a description of the proposed construction and actions
– design values of soil and rock properties (justified where appropriate)
– reference to codes and standards used
– statements on the suitability of the site with respect to the proposed construction and the level of acceptable risks

– geotechnical design calculations and drawings
– design recommendations
– sequence of construction assumed
– required maintenance and monitoring
– specification of the manner in which load tests are to be carried out.

The Geotechnical Design Report need only be as detailed as is necessary and the format can be selected by the designer.

See also Section 1.3.

3.8.2 Construction supervision, maintenance and monitoring plan

Specification of supervision of construction and monitoring is required, and is to include:
– items to be checked during construction
– the purpose of each set of observations or measurements
– the locations and frequency for measurement and observation
– the methods for evaluating results (and their expected ranges)
– identification of those parties responsible for measurements, observations, interpretation of results and maintenance of monitoring equipment
– acceptable limits for results to be obtained by supervision
– the type, quality and frequency of supervision
– the type, quantity and frequency of field and laboratory testing
– key assumptions
– risk assessment and contingency measures (as appropriate)
– the duration that monitoring is to continue after the end of construction
– checks to verify that the weight density of backfill is no worse than that used in design

Specification of maintenance should provide information on:
– critical parts of the structure which require regular inspection and/or maintenance (e.g. drains)
– form of monitoring required
– frequency of monitoring
– remedial works in case monitoring results require action
– works that require a design review before being undertaken.

The requirements of the plan should be commensurate with:
– the degree of uncertainty in the design assumptions
– the complexity of the ground and loading conditions and construction processes
– the potential risk of failure during construction (including impact on third parties)
– the feasibility of implementing changes during construction.

It is noted that monitoring does not necessarily imply that instruments should be used, as monitoring could include observation by a competent site engineer.

The requirements for supervision, monitoring and maintenance of a completed structure must be communicated to the client or the owner.

3.9 Summary

– The aim of geotechnical investigation is to ensure that sufficient and appropriate information is available when needed to manage risks.

– Investigation should be phased.

– A desk study and intrusive ground investigation are required.

– There are minimum requirements with respect to the number of boreholes, trial pits etc., the depth to which they reach, and the number and type of laboratory tests.

– Ground investigation needs to be appropriately specified and procured.

– The division of tasks between designer and ground investigation contractor needs to be clear.

– A Ground Investigation Report and a Geotechnical Design Report are required and there are minimum requirements with respect to content.

4 Introduction to design

4.1 Introduction

This chapter builds on Chapter 2, where the basis of design was presented, by providing guidance on the choice of characteristic values of geotechnical parameters (Section 4.2). Comments are also provided on the interaction between structural (EC2 and EC3) and geotechnical (EC7) engineering (Section 4.3) which build on the initial comments in Section 1.7. The final section of this chapter deals with common environmental issues such as sulphate attack on concrete and corrosion of steel piles (Section 4.4).

4.2 Geotechnical parameters

4.2.1 General

The development of geotechnical design parameters is comprised of three options as per Table 4.1.

Table 4.1 Options for determining design values

	Option 1	Option 2	Option 3
Step 1:	Evaluating test results and other relevant data to obtain derived values of geotechnical data	Evaluating test results to obtain a characteristic value of the test results	
Step 2:	Interpreting the derived values to select characteristic values of strength and stiffness (and other parameters as appropriate)		
Step 3:	Factoring characteristic values to obtain design values		Directly assess the design value

At Step 1 the properties of soil and rock (geotechnical parameters) are obtained from test results either directly or indirectly (correlation, theory, empiricism) and from other relevant data as illustrated in Figure 3.1. Consideration of how to obtain derived values at Step 1 is beyond the scope of this *Manual*. For further information on derived values the reader is referred to Chapter B16 of *Manual of Geotechnical Engineering* published by the Institution of Civil Engineers[55] as well as other commentaries and handbooks on EC7 that continue to be published.

Steps 2 and 3 are discussed below.

4.2.2 Characteristic value – sources of information

When selecting a characteristic value the designer must consider information from the following primary sources and their reliability:
- Geological and hydrogeological data from desk study data (e.g. maps, memoirs, reports and previous projects etc.) (see Section 3.3).
- Potential for variation from existing data.
- Comparable experience.
- The volume of ground which governs the limit state being considered.
- Field investigations/sampling and laboratory testing and the appropriateness/extent of such investigations (see Section 3.4).
- Ability of the structure to accommodate variation in ground conditions (ductility of the structure).

Characteristic values are described in Section 2.4.5 of EC7 Part 1[1].

4.2.3 Characteristic values and limit states – soil/rock

Within this discussion the term 'limit state' is used in the context of a design situation and is not specific to either ultimate limit state (ULS) or serviceability limit state (SLS) for which different partial factors are applied to the characteristic value to obtain the design value.

The characteristic value of a geotechnical parameter is defined in EC7 Part 1[1] in a qualitative manner as "a cautious estimate of the value affecting the occurrence of the limit state". In terms of historical practice, the characteristic value can be considered to be similar to the term "moderately conservative" in CIRIA C580[35]. In order to choose a characteristic value of a geotechnical parameter from a set of data the designer must consider the following:
- *The limit state that is being assessed:* For a particular set of data there may be, in fact will nearly always be, more than one characteristic value as a function of the limit state being assessed. An example of this is the difference between a limit state when soil strength is a benefit (e.g. for a bearing pile in clay) and when the soil strength is a potential disbenefit (e.g. when soft soil results in down-drag on a bearing pile). Hence, the designer must identify the characteristic value for a specific limit state and not make the characteristic value a material constant.

– *The mechanism by which the ground behaves when mobilising the limit state being assessed:* The ground parameter (e.g. strength, stiffness, permeability etc.) that controls the limit state will be related to a particular mechanism that allows the assessment, usually a calculation, to be carried out. The choice of mechanism (e.g. a slip circle in homogenous soil or a failure wedge in rock) needs to consider the ground structure and variability in order to identify realistic design calculations. Thereafter, the choice of characteristic value must consider the mechanism to ensure that the characteristic value is appropriate to the way in which the ground resistance will be mobilised.
– *The volume of ground controlling the limit state:* The volume of ground affecting the occurrence of a limit state should be considered. For example, where the volume of ground is small then the characteristic value should be the lower (or upper if appropriate) bound value of the parameter being considered. However, where the volume of ground is large then the way in which soil behaviour will be mobilised should be considered. Given these conditions for determining the characteristic value the designer must understand how the limit states being considered will come to exist; this requires the failure mechanism to be understood for ULS design considerations and how movements will occur for SLS considerations.
– *The characteristic value may vary with time (e.g. a swelling clay loses undrained shear strength) or with progressive failure (soil strength reducing below peak strength):* The chosen value should be a cautious estimate of that which controls the specific limit state being considered at a specific time.

While the characteristic value is defined in EC7, the method for choosing the characteristic value is not so well described. In choosing a characteristic value the reader could commence with an assessment of the best estimate value of the parameter for the limit state being considered, along with an assessment of how the data varies from this best estimate. From this assessment, using an engineer's judgement, it is possible to move towards the characteristic value in a qualitative assessment such that the subsequent calculation of characteristic resistance will be a moderately conservative assessment of the actual resistance that could be mobilised.

4.2.4 Characteristic values and limit states – groundwater

In a similar manner to above, the choice of groundwater conditions should also be considered with the criterion of "a cautious estimate of the value affecting the occurrence of the limit state"[1]. For groundwater it is the groundwater level, or range of levels, that needs to be considered as well as the potential for water to flow, generating hydraulic gradients.

When choosing the groundwater level for use in the design process it is recommended that the following is adhered to:
– The groundwater level prior to any works taking place should be based on the monitored water level plus allowance for seasonal variations that are

beyond the data set of recorded levels. This data allows a level to be set that may be considered to be the existing expected, or most probable, level. This level should then be adjusted to allow for adverse variations that are reasonably possible during the design life in order to arrive at the characteristic groundwater level. Following identification of this characteristic groundwater level it is necessary to assess if the construction being proposed, or others that are known about, will change the groundwater level at the site of the structure during the execution phase of the project or during the operational phase of the project in order to arrive at the future characteristic groundwater level (e.g. the construction of a large basement in an area of groundwater flow resulting in a local obstruction to flow that needs to be included in the design).

- For design to STR/GEO limit states, the general assumption for urban UK development assumes that during the operational phase of a retaining wall the wall should be designed for a characteristic groundwater level no lower than 1m below ground level in keeping with common UK practice[56] to allow for burst water mains and local flooding etc. In addition EC7 suggests that where the soil is of low permeability (silts and clays) the water level should be taken at ground level unless a reliable drainage system is installed or infiltration is prevented (Clause 9.6(3)P of EC7 Part 1[1]). These groundwater levels and the water pressures resulting from them are not the design values but the characteristic values. When this approach is taken, the water can be assumed to generate a permanent action which will attract permanent rather than variable partial factors when calculating design actions[57].
- For design against UPL (uplift, see Section 2.11.3) it is necessary to take a more cautious approach to water due to the nature of the limit state being considered, water pressure being one of the main design considerations in UPL. This more cautious approach leads to a ‘worst credible’ water level being adopted for use as a characteristic value in the design process. In this case, unless there is very good reason not to, the water level should be taken at ground level in relatively flat areas and above this level if there is a change of artesian pressures.
- Where there is evidence of significant groundwater flow or more complex hydrogeological conditions (hillsides, multi layered aquifers, areas undergoing dewatering by third parties etc.) then specialist advice should be obtained to better define the potential variation in groundwater level and the impact of groundwater on proposed development.
- The characteristic water level during the construction phase can be closer to the measured level with allowance for any damming effect that a basement construction may cause and seasonal effects etc.
- It is possible to assess the effects of groundwater level directly by addressing the factors which control the characteristic groundwater level and then adding a margin of safety to this level to arrive at the design level. Groundwater pressures calculated from this design level are not factored further (they are design values). Such an approach is less common in urban design in the UK but may be considered where a groundwater table close to ground level is considered unforeseeable.

The above discusses groundwater levels for design of a structure against instability and structural failure (ULS limit states). When considering waterproofing (an SLS consideration) it is likely to be appropriate to detail the structure such that groundwater and/or dampness are excluded from entering the building from ground level downwards.

4.2.5 Characteristic value – examples

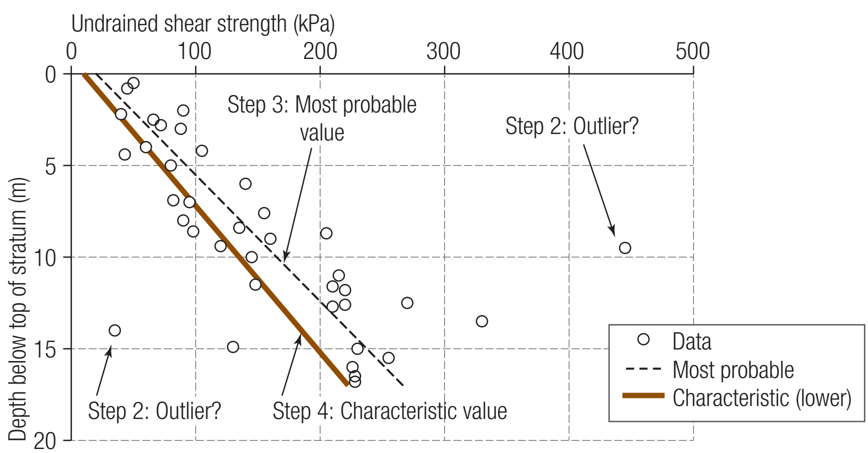
4.2.5.1 Options

The following options provide a means by which the characteristic can be identified:

- Choose a characteristic value by means of a line drawn by eye through the data set – a qualitative assessment using an engineer's judgement.
- Choose a characteristic value by using statistical analysis of ground data.

4.2.5.2 Characteristic values – qualitative assessment

Historically, the use of a line drawn by hand through a data set is the most common method of assessing representative values of soil parameters in the UK. An example of this approach using undrained shear strength data c_u of stiff clay is shown in Figure 4.1. The most probable line is shown for the



Note

Step 1: Plot data with depth (or level if more appropriate)
Step 2: Identify data points which are considered non-representative due to sample disturbance, local inclusions (e.g. Clay stones etc)
Step 3: Draw most probable value according to limit state and performance of the ground (this may not be the mean line)
Step 4: Assess characteristic design line, in range of 15–25% of data outside the line

Fig 4.1 Assessment of characteristic design line – typical case

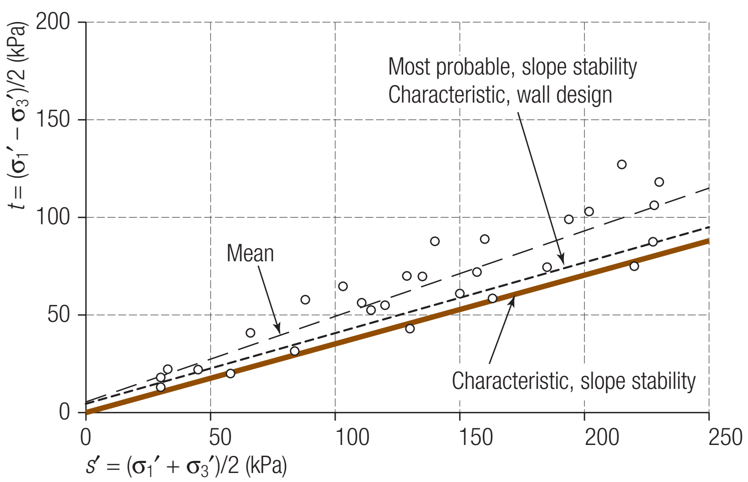


Fig 4.2 Assessment of characteristic design line – design case dependent

situation where the soil mass governs the limit state, i.e. the most probable line, representing most probable ground behaviour, is comparable with the mean line through the data points. The characteristic line is shown for the (typical) case where soil strength is beneficial. The lines are both 'eyed-in' and it is likely that other engineers would have chosen marginally different lines to those shown. It is considered that such a situation is acceptable so long as the data used to draw the line(s) through has been checked for appropriateness and correctness to prevent gross error, and that the characteristic line passes the requirements for "a cautious estimate of the value affecting the occurrence of the limit state".

The second example, Figure 4.2, shows the qualitative assessment of a characteristic design line using triaxial shear strength data. The parameters φ' and c' are required for two different design cases:
– for the calculation of earth pressure coefficients K_a and K_p as used in retaining wall design
– for the long-term design of a slope in clay.

The example shows the data set has a reasonable scatter with samples exhibiting lowest strengths being dominated by pre-existing fissures and those with highest strengths being intact samples. The data plot shows three lines. The mean line is the arithmetic mean through the data; the only judgement in drawing this line is the consideration of the need to remove outliers – in this instance none were considered present. The middle line is judged to be the characteristic line for retaining wall design where the presence of the retaining wall constrains the failure surface in the soil thereby ensuring that the mass strength of the soil is mobilised; the line has approximately 15 to 25% of data points below it. The lowest line is the characteristic design line for slope stability where the failure mechanism, in this hypothetical case, is free to mobilise the weakest soil due to the build-up of the strata; and where higher strengths are not representative of such a failure mechanism.

This example attempts to illustrate two important points:

(1) Within the limitations of ground investigation, laboratory testing and reporting, the geotechnical data will usually be such that the characteristic value is not an absolute constant but will vary with the limit state being assessed. For example, for slope stability assessment it is not possible to test only samples with fissures orientated in the correct direction to obtain the strength along the critical slip circle. To obtain a relevant data set the data needs to be sorted into relevant and not relevant with outliers removed; experience is needed to do this.

(2) The assessment of the characteristic value, namely the most probable value affecting the occurrence of the limit state, is not the arithmetic mean of the representative data but the line that the engineer judges to be most probable for the limit state being considered.

Further examples of choice of characteristic parameters are presented in Section 7.12.1 and Figure 7.1, for the case of negative skin friction on a pile where the designer must consider the higher rather than the lower characteristic value, and in Appendix A.

As a final note on parameters in this example, specifically c', attention is drawn to the likely large variation in this parameter compared to φ' and the need to apply caution to the selection of the characteristic value. It is also noted that c' can be reduced to zero by means of ground disturbance due to construction processes.

4.2.5.3 Characteristic values – statistical assessment

Whilst part of EC7, it is considered that statistical assessment of characteristic values for geotechnical parameters is not suitable for the typical design of UK buildings.

Statistical methods may be used to accommodate soil variability in design. Any statistical method used should allow local and regional sampling to be differentiated and permit the use of *a priori* knowledge[58]. The difficulty in using statistics is that it is not intended that the characteristic value of the parameter be the five percentile value of the data, but that the resulting stability analysis, based on a large number of calculations which model the ground's variability, has a five percentile chance of exceeding the limit state. In any assessment using such an approach the required partial factors need to be incorporated in the assessment.

When using statistical methods, caution should be exercised to ensure that outliers (which may or may not be significant to the occurrence of a limit state) are considered appropriately and that existing knowledge contained in publications and comparable experience is not ignored. It is considered that statistics should not be applied blindly but rather used as a tool providing guidance.

4.2.6 Design value

A material design value is generally obtained from a characteristic value by the application of a partial factor (i.e. $X_d = X_k/\gamma_M$, see Section 2.10.1 above) or selected directly (i.e. the designer selects a design value directly which must offer the same level of reliability as implied by the partial factors). Values for the partial factor γ_M are presented in Chapter 2.

In cases where factoring a characteristic value results in an illogical design value then the direct selection of a design value may be more appropriate than determining a characteristic value and applying a partial factor. In such situations the value of the partial factor (and where appropriate other factors) that would have been used in the conversion of the characteristic value to design value should be remembered to ensure that appropriate level of safety is applied to the design value.

Direct assessment of design values are likely to be most common in GC1 design but may also be important in GC2 and GC3 designs.

Use of prescriptive design (Section 2.12 above) often relies on the direct assessment of design values without use of partial factors. In such cases the geotechnical parameter on which the design is based must be selected in a manner that is appropriate to the basis of the design (average value of the parameter, moderately conservative value of the parameter etc.) Furthermore, the calculation of the design action must be similarly based on the rules that were adopted at the time the prescriptive design method was formed.

4.3 Interaction with structural design

Initial comments on the relationships between geotechnical and structural design have been presented in Chapter 1 and illustrated in Figure 1.4. For general design of foundations and retaining walls this interaction will largely be one of exchange of actions and resistances. Table 4.2 presents a possible interaction between geotechnical and structural design work for a building project.

For design to the Eurocode set of documents in the UK, structural design must be undertaken in accordance with EC2 to EC6 and EC9 whilst the geotechnical design is undertaken to EC7.

Table 4.2 Design disciplines

Step	Issue	Design disciplines
1	– Calculation of structural actions imposed externally to geotechnical element – Calculated characteristic values of permanent and variable actions	Structural engineering
2	– Agreement of limits states and combinations of actions to be considered and partial factors to be used	Structural engineering with geotechnical engineering input
3	– Calculation of design actions for various limit states	As appropriate to the origin of these actions. Clarity required as to the limit state and partial factors used
4	– Calculation of design foundation resistances	Geotechnical engineering
5	– Assess correctness of geotechnical design	Geotechnical engineering
6	– Provision of resulting ULS design actions (axial load, shear stress and bending moment) and resulting serviceability limit state performance (settlement, heave and lateral movements of structure and adjacent ground/ structures)	Geotechnical engineering
7	– Assessment of structural performance and serviceability performance of structure and associated structures – Assessment of structural dimensions and impact on architectural space as necessary – Party wall issues	Structural engineering with geotechnical engineering input
8	– Assessment of correctness of design calculations based on resulting solution – Assessment of 'buildability' of resulting scheme Note: If changes needed loop back to appropriate step above	Structural engineering and geotechnical engineering as appropriate
9	– Geotechnical Design Report	Geotechnical engineering

4.4 Geo-environmental considerations

4.4.1 Introduction

This section deals with geo-environmental conditions which impact on structural material rather than aspects of contamination, contaminated soil and waste classification. In the main part this topic is not addressed in detail by EC7 over and beyond the need for durability.

4.4.2 Chemical attack on concrete

Durability of structures is as much part of design as stability, strength and serviceability. The following should be considered:

Design of concrete against aggressive ground is addressed in BRE Special Digest 1 (2005) *Concrete in aggressive ground*[59] and BS EN 206[60]. The assessment is based on desk study followed by ground investigation sampling of the ground and groundwater followed by chemical analysis of the samples and formulation of a ground model. Chemical sampling focuses on pH, sulphate and total sulphur testing. The method by which the Design Chemical Class (DC) of the ground and requirements for Additional Protective Measures (APMs) is defined includes a number of steps as shown in Table 4.3.

Table 4.3 Chemical design of concrete

Step	Input	Result
1	Sulphate level in the ground	Design Sulphate Class (DS)
	Potential sulphate level in the ground	
	Natural ground or brownfield site	
2	DS Class	Aggressive Chemical Environment for Concrete (ACEC) Class
	pH of the ground	
	Mobility of groundwater	
3	ACEC Class	Design Chemical Class (DC) plus Additional Protective Measures (APMs)
	Design Life	
4	DC Class	Concrete mix
5	APM	Options to provide enhanced protection

Note
For further information see *Concrete in aggressive ground*[59].

The BRE guide[59] provides rules for identifying the representative values for ground and groundwater sulphate level and pH level.

4.4.3 Corrosion of sheet piling and steel bearing piles

Steel exposed to the environment will corrode unless protected by means of:
– maintained cathodic protection
– maintained painting.

Where such measures are not put in place then allowance in design for reduction in the steel section modulus and moments of inertia is required. EC3 Part 5[6] and its UK National Annex[61] provide guidance on corrosion rates depending on the environmental conditions, as presented in Tables 4.4 and 4.5.

Table 4.4 Recommended value for the loss of thickness (mm) due to corrosion for piles and sheet piles in soils, with or without groundwater

Required design working life	Recommended value for the loss of thickness (mm) due to corrosion for varying periods (years)					
	5	25	50	75	100	125
Undisturbed natural soils (sand, silt, clay, schist, etc.)	0.00	0.30	0.60	0.90	1.20	1.50
Polluted natural soils and industrial sites	0.15	0.75	1.50	2.25	3.00	3.75
Aggressive natural soils (swamp, marsh, peat etc.)	0.20	1.00	1.75	2.50	3.25	4.00
Non-compacted and non-aggressive fills (clay, schist, sand, silt etc.) (for compacted fills divide values by 2)	0.18	0.70	1.2	1.70	2.2	2.70
Non-compacted and aggressive fills (ashes, slag etc.) (for compacted fills divide values by 2)	0.50	2.00	3.25	4.50	5.75	7.00

Notes

a The values given for 5 and 25 years are based on measurements, whereas the other values are extrapolated.

b This Table is derived from EC3 Part 5[6].

Table 4.5 Recommended value for the loss of thickness (mm) due to corrosion for piles and sheet piles in fresh water or in sea water

Required design working life	Recommended value for the loss of thickness (mm) due to corrosion for varying design periods (years)					
	5	25	50	75	100	125
Common fresh water (river, ship canal etc.) in the zone of high attack (water line)	0.15	0.55	0.90	1.15	1.40	1.65
Brackish or very polluted fresh water (sewage, industrial effluent etc.) in the zone of high attack (water line)	0.30	1.30	2.30	3.30	4.30	5.30
Sea water in temperate climate in the high tide splash zone or in the low water zone attack (see comment in following paragraph on ALWC)	0.55	1.90	3.75	5.60	7.50	Protection system required
Sea water in temperate climate in the zone of permanent immersion or in the intertidal zone	0.25	0.90	1.75	2.60	3.50	4.40

Notes

a The highest corrosion rate is usually found in the splash zone or at the low water level in tidal waters. However, in most cases, the highest bending stresses occur in the permanent immersion zone.

b The values given for 5 and 25 years are based on measurements, whereas the other values are extrapolated.

c This Table is derived from EC3 Part 5[6].

Accelerated low water corrosion (ALWC) is not accounted for in Table 4.5. ALWC typically occurs at or below low water level and is caused by bacterial action; corrosion rates can be an order of magnitude greater than those shown in Table 4.5. Should such conditions exist at the development site additional protection measures will need to be provided. CIRIA publication C634 *Management of accelerated low water corrosion in steel maritime structures*[62] may be consulted as an initial investigation into the phenomenon.

4.4.4 Contaminated land

Reference to contaminated land assessment can be obtained from the Environment Agency in the form of Contaminated Land Exposure Assessment (CLEA) and Soil Guideline Values (SGVs). SGVs provide guidance on the acceptable level of contamination for a variety of elements for 'residential', 'allotment' and 'commercial' land uses. It is recommended that a geo-environmental specialist be consulted when dealing with contaminated or potentially contaminated sites.

4.5 Illustration of design process

Refer to Appendix A.

4.6 Summary

- The selection of geotechnical parameters follows a staged process from 'raw data' to 'derived value' to 'characteristic value' to 'design value'.

- The 'characteristic value' is a cautious estimate of the value affecting the occurrence of the limit state being considered and is commensurate with the traditional term 'moderately conservative'.

- The selection of characteristic or design values for groundwater requires careful consideration.

- Statistical methods of selecting a characteristic value can be used but should not be applied blindly.

- Structural and geotechnical designers need to understand their roles and what each requires from the other.

5 Overall stability and impacts on adjacent structures

5.1 Introduction

Within the context of EC7, overall stability (Chapter 11 of EC7 Part 1[1]) is deemed to be addressed by the following limit states:
- Loss of overall stability of the ground (natural slopes, cuttings, embankments); ultimate limit state.
- Ground movement which is not a direct result of the project and which would impact on the project in serviceability or ultimate limit states.

Overall stability is equally well applied to ground without any construction activity (the natural slope) as it is to ground during and post the execution of the project. Overall stability applies to both temporary and permanent conditions.

Whilst not covered in detail in EC7 as part of overall stability, this chapter also introduces a number of design limit states which focus on the effect that a project could have on structures beyond the limits of the construction. Such examples include (Clause 11.2(2) of EC7 Part 1[1]):
- Damage (beyond repair) of adjacent structures, roads or services due to movements in the ground due to the construction – this is an ultimate limit state consideration of associated structures.
- Damage (repairable) of adjacent structures, roads or services due to movements in the ground due to the construction – this can be considered to be a serviceability limit state consideration for associated structures (it is likely not be acceptable).

Damage of adjacent structures can be due to ground movement in shear, settlement or heave, or from unacceptable vibration.

Figures 5.1 to 5.6 show examples of overall stability and also examples of design situations which need to be guarded against when considering structures beyond the limit of the construction.

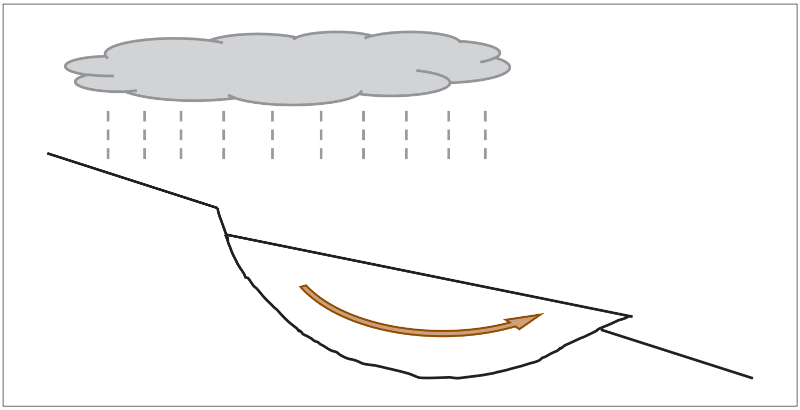


Fig 5.1 Overall instability of natural slope (rain induced?) – strength controlled ULS (GEO)

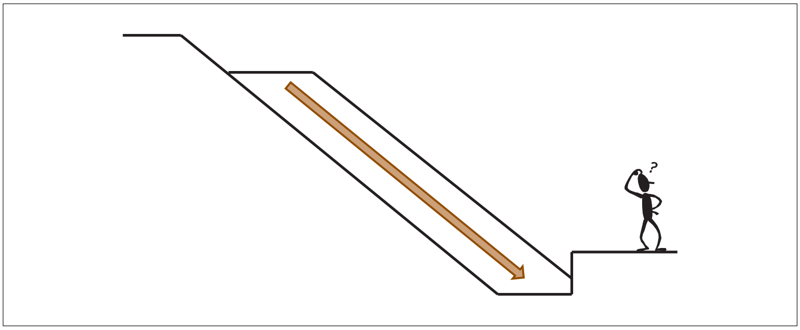


Fig 5.2 Overall instability, slope stability into a newly excavated trench – strength controlled ULS (GEO)

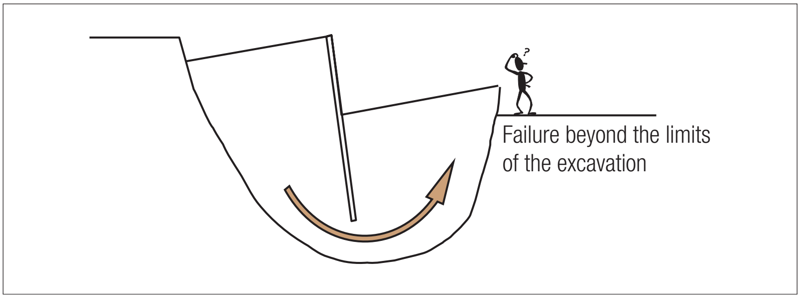


Fig 5.3 Overall instability, rotational failure induced by a retained excavation – strength controlled ULS (GEO)

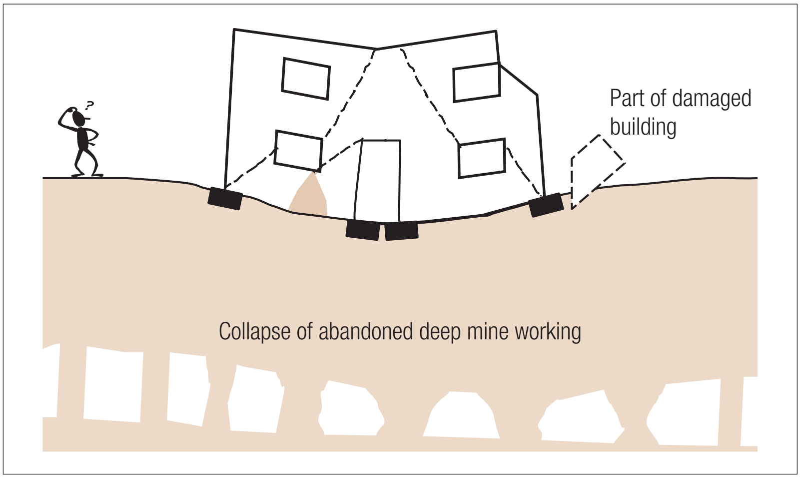


Fig 5.4 Loss of stability of structure due to overall stability of the collapse of deep abandoned mine workings (deformation controlled ULS)

In Figure 5.3 it is clear that failure has occurred even though the wall itself has not failed.

Assessment of geotechnical overall stability is carried out using ULS analyses as presented for GEO in Section 2.11.3.4. The method of calculation is presented in Section 5.3 and may be considered to be appropriate to GC2 designs.

The second example of overall stability, related to ground movement, as distinct to identifiable shear failures is shown in Figure 5.4. This example

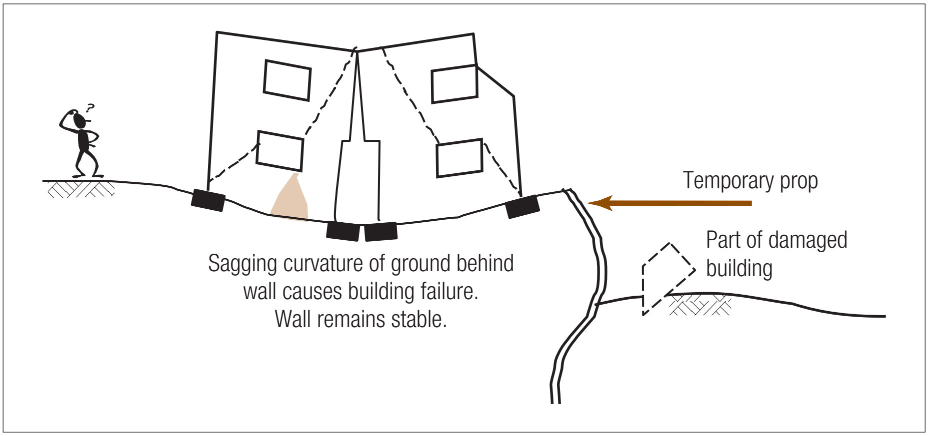


Fig 5.5 Loss of stability of structure behind a retaining wall due to excess settlement (deformation controlled ULS)

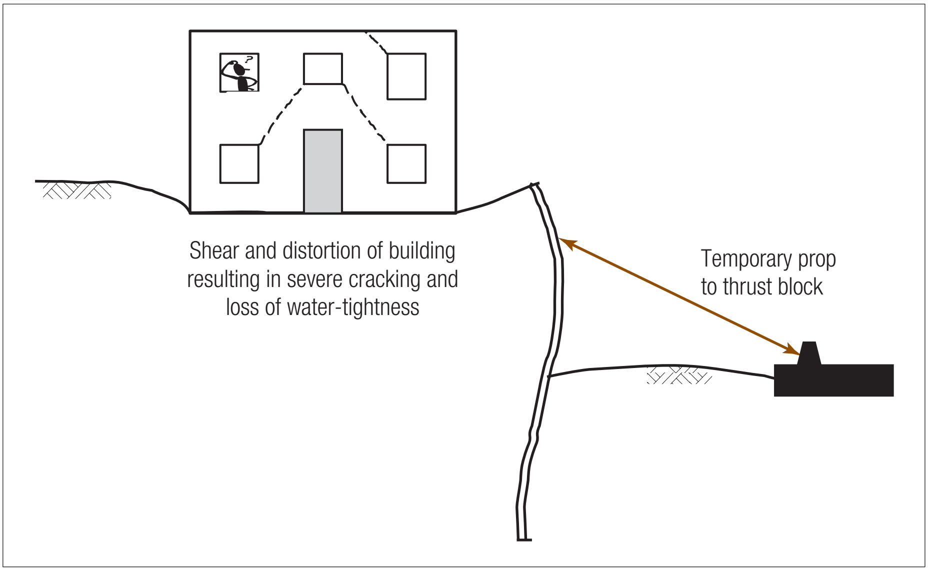


Fig 5.6 Loss of function of structure behind a retaining wall due to excess settlement (deformation controlled SLS)

shows how collapse of abandoned deep mine workings cause a failure to the overlying structure; the structure does not necessarily trigger the event. Such an example could also have SLS implications if movements were small.

Whilst the situations shown in Figures 5.5 and 5.6 are not examples of overall stability in the context of EC7 they are presented as examples of where construction work can have implications beyond the perimeter of the site.

These examples (Figures 5.5 and 5.6) are deformation controlled. Full assessment of such effects can only be fully carried out as part of the detailed design of the geotechnical construction being considered. Preliminary assessment may be required during planning to ensure that the proposed construction is viable; this may be a requirement of planning approval. During the detailed design, ground movement behind retained excavations or earthworks should include an assessment of the impact on adjacent structures; such assessments often benefit from surveys of the impacted structures to identify how they will respond to the predicted movements. These assessments are typically carried out using SLS design ground parameters (c.f. assessment of shear failure analysis which uses ULS design parameters as applicable to Figures 5.1 to 5.3).

5.2 Overall stability – introduction to geotechnical design

Adequate assessment of geotechnical slope stability can only be carried out once the following information is available:
– slope geometry
– ground and groundwater conditions including stratigraphy, structure of the ground and strength
– the potential failure mechanisms of the structure (slope, cutting, retained excavation etc.)
– applied actions (e.g. foundation loads).

During construction of all slopes and trenches, checks must be made to allow identification of unexpected conditions (unexpected groundwater conditions, pre-existing fissures with or without water, soil strata different from that assumed etc.); the observations should be compared to the Geotechnical Design Report findings. Where conditions differ from those assumed a reassessment of stability must be carried out.

For pre-existing slopes (natural in the main part), geotechnical assessment should be carried out as part of detailed design where there is reasonable concern that the stability may have an impact on the construction or permanent performance of the project. Assessment of slope stability for existing slopes should consider the presence of pre-existing shear planes within these slopes as they may have reduced shear strength. The assessment should consider the development site as well as the zone beyond the site which may affect or be affected by the development.

As a final note to these comments, the issue of access and safety must be addressed. Unsupported trenches, even if shallow, should not normally be entered. For all trenches and slopes, regular inspection must be carried out during construction site works to ensure that movements are not occurring that would suggest an increased risk of collapse or detrimental effect on adjacent structures. Full and up to date guidance from the HSE should be consulted (www.hse.gov.uk).

5.3 Design for overall stability

5.3.1 General

An introduction to overall stability has been presented in Section 5.1 where main forms of overall stability were presented with additional general design considerations; here they are placed into three groupings:

Group 1: Overall stability – shear failure of the ground:
– This includes the stability of slopes which may impact the development even if the development does not change the slope (see Figure 5.1) as well as

overall stability of the construction where structural elements are absent (see Figure 5.2) or not influential in stabilising the construction (see Figure 5.3).
– As a general rule, where live loading is small compared to the weight of the failing mass, overall stability can be checked using Design Approach 1 Combination 2 of STR/GEO consideration for the ULS; this is because the factors applied to the ground strength in Combination 2 have a greater effect than factors applied to the actions in Combination 1. It is important to note that factors applied to the body of failing ground are equally applied to those parts which are destabilising and those parts stabilising (i.e. single source rule). Appendix B presents a sample calculation.

Group 2: Overall stability – ground movements not related to the current construction:
– Constructions which experience excess movement due to issues not related to the construction such that there is a loss of stability (see Figure 5.4) or serviceability. The main means of designing against such situations is to carry out remedial works to prevent the movement from taking place (or build elsewhere).

Group 3: Construction works which cause a limit state failure in adjacent constructions:
– The two examples given in Figures 5.5 and 5.6 are for retaining walls which are supporting an existing building. Movements due to wall installation and excavation result in limit states being reached in the existing adjacent building.
– Similar situations could be envisaged due to construction dewatering causing settlement which impacts on adjacent structures or heavy constructions on soft ground causing ground movements beyond the limit of the site.
– The examples used show buildings being impacted. It is equally possible for roads, utilities or other forms of construction to be impacted; the same considerations apply.

Group 1 includes the conventional forms of overall stability. They can be addressed by means of limit equilibrium analysis in most cases. Typically ULS Design Approach 1, Combination 2 will be the critical case.

Groups 2 and 3 both require an assessment of movement to be carried out to assess the resulting impact on serviceability and ultimate limit states; in these cases stiffness of the ground and magnitude of differential movement are often important.

5.3.2 Assessment of movements

The assessment of movement must include all actions which are planned as part of the proposed construction. These include the construction processes listed in Table 5.1.

Table 5.1 Construction processes and nature of potential movement

Construction process	Nature of potential movement
Foundation construction and loading	– Settlement of structure – Settlement of adjacent structures
Driven pile construction	– Settlement of adjacent structures – Vibration of adjacent structures
Bored pile construction	– Settlement of adjacent structures
Pile loading	– Settlement of structure – Settlement of adjacent structures
Retaining wall installation: secant, contiguous and diaphragm walls	– Horizontal movement and settlement of adjacent ground/structures (CIRIA C580[35])
Retaining wall installation: driven sheet piles	– Settlement of adjacent structures – Vibration of adjacent structures
Excavation within retaining walls	– Horizontal movement and settlement of adjacent ground/structures (CIRIA C580[35])
Open cut excavation	– Settlement/heave of adjacent structures, horizontal movement of adjacent structures
Dewatering	– Settlement of adjacent structures
Note All construction processes listed are likely to result in differential movements in adjacent structures as a function of the distance (and varying distance) of the structure from the point of construction and a function of the structure's detailing (e.g. mixed foundations, service connections etc.)	

The assessment of damage must also consider the existing condition of adjacent structures. Old and historic buildings may have unpredictable behaviour and non-standard detailing (either original construction or modifications since construction) as well as damage which has been covered up or repaired.

5.3.3 Group 1 – geotechnical collapse

Group 1 Overall Stability (ULS), as above, is usually addressed by comparing design soil strength which provides resistance and design actions based on STR/GEO Design Approach 1 Combination 2 parameters:

$$E_d \leq R_d$$

where:
E_d is the design value of the effect of actions
R_d is the design resistance

The comparison of actions (or effects of actions) and resistances are generally carried out in units of force (e.g. a failure wedge analysis) or moment (e.g. a slip circle analysis), as appropriate to the mechanism of failure being considered.

Choices of characteristic soil strength parameters and geometric data need to be consistent with the requirement that the characteristic value is a cautious estimate of the value controlling the occurrence of the limit state. In addition to the considerations given in in Section 4.2 the following comments are provided:

– *Short and long-term conditions*: both short term and long-term stability should be considered for overall assessment. As a general rule short-term conditions are usually critical for cases of net loading (e.g. embankment loading). Long-term conditions are usually critical for cases of net unloading (e.g. excavations). Unfortunately cases occur which combine components of loading and unloading (e.g. an embankment formed of compacted stiff clay on a clay formation) and result in the need to consider both conditions unless one is clearly more critical than the other. (Assessment of movement in short and long-term conditions, SLS, is likely necessary irrespective of the controlling ULS condition.)
– *Groundwater regime*: when choosing a characteristic groundwater level it should be a cautious estimate of the groundwater level that could occur; this may vary with time. It should be based on monitoring data, natural variation beyond the monitored data and changes resulting from the construction. Critically, soil heterogeneity should be considered as this may have a large influence on the resulting long-term groundwater pressure regime.
– *Slope height and angle*: allowance for planned (cleaning of drainage runs) and unplanned excavations (overdig during construction) at the toe of a slope/retaining wall should be made (see Section 8.13).
– *Applied loading*: surface loading should be a characteristic assessment of anticipated loading with appropriate partial factors. Historically it is usual to assume a surcharge based on HA/HB loading for roads. However (see Section 2.11.2.2), new guidance exists in the form of PD 6694-1[34] where highway loading is more rigorously defined. Prior to embarking on a design which attracts highway loading it is recommended that the required loading is agreed with the overseeing authority.

Design against overall stability associated with limiting the strength of the ground rather than failure of structures supported by the ground can be assessed with factors as presented in Tables 2.7a and 2.7b and as repeated in Tables 5.2 and 5.3.

Table 5.2 STR/GEO (Set A2) partial factors (actions) – buildings as applied to slopes

Factors on actions (Set A2)	Permanent γ_G		Variable γ_Q		
	Unfavourable $\gamma_{G,j}$	Favourable $\gamma_{G,j}$	Unfavourable (leading) $\gamma_{Q,1}$	Unfavourable (accompanying) $\gamma_{Q,i}$	Favourable $\gamma_{Q,1}$ or $\gamma_{Q,i}$
γ	1.0	1.0	1.3	1.3	0.0
Note Factors from NA to EC0[42], Table NA.A1.2(C).					

Table 5.3 STR/GEO (Set M2) partial factors (materials) – buildings as applied to slopes

Material factors (Set M2)	$\gamma_{\varphi'}$	$\gamma_{c'}$	γ_{cu}	γ_{qu}
γ_m	1.25	1.25	1.4	1.4
Note Factors from NA to EC7 Part 1[44], Table A.NA.4.				

When using these factors the ground and groundwater are treated as permanent actions (there is no distinction in value of favourable and unfavourable partial factors); surcharges are typically variable actions.

It is usual to carry out overall stability checks for geotechnical collapse using either limit equilibrium analysis (slope stability analysis) or finite element analysis; finite element analysis is beyond the scope of this *Manual*. When limit equilibrium analysis is used it is typical to use proprietary computer software for slope stability calculations. When using such software the methods of analysis should satisfy the requirement that overall moment and vertical stability of the sliding block is checked and that if horizontal equilibrium is not checked then inter-slice forces should be assumed to be horizontal. The following methods are a sample of those which satisfy these requirements:
– Bishop method[63] with either horizontal or inclined inter-slice forces
– Janbu method[64] with inclined inter-slice forces
– Morgernstern and Price method[65]
– Spencer method[66].

Design for the less usual form of overall stability (see Figure 5.2) using EQU should use the partial factors in Table 2.6a and 2.6b.

5.3.4 Groups 2 and 3 – movement controlled

Limit states which are reached as a result of movement include consideration of both ULS and SLS events. When considering such events a movement assessment must be carried out to allow categorisation of the expected movement:
– Acceptable movement: no limit state exceeded.
– Unacceptable movement: SLS limit state reached.
– Unacceptable movement: ULS limit state reached.

Assessment of limit state failure due to excessive movement requires calculation of movements associated with the proposed construction or from other effects. Examples of excessive movement are shown illustratively in Section 5.1 (see Figures 5.1 to 5.6).

When assessing movement related limit states, calculation of displacements should be based on design values equal to the characteristic values; the parameters should not typically be factored when calculating the displacement. Depending on the situation, recourse to EC0[3] can be made where a partial factor for the effects of uneven settlement γ_{Gset} is presented for ULS situations; it has a value above unity for the STR/GEO Design Approach 1 Combination 1 (A1) partial factors (i.e. when material strengths are unfactored). For this case the suggested γ_{Gset} value in EC0[3] is 1.2 or 1.35 depending on the situation (linear or non-linear analysis of the structure); this factor is applied to the effects of movement. Choice of strength and stiffness values should be the characteristic lower values while detrimental surcharges should be the characteristic upper values. Comparison of computed movements should be made with case history data to identify that the predicted modes of behaviour are reasonable.

It is the combination of structural stiffness, ground strength and stiffness, and the resilience of the structure being impacted that will result in the limit state being exceeded. It is necessary that each impacted structure is considered independently according to its condition and detailing when planning a proposed construction. Initial assessment of the likely consequences of movement to low rise structures can be made using damage classification charts or tables. The damage classification presented in Burland[67] presents a framework for assessing damage to masonry structures. Categorisation of damage is based on ease of repair with illustrative crack sizes presented as a secondary means of visualising damage. The assessment of damage is based on the calculated deflection ratio (sagging or hogging of the building) together with the calculated horizontal strain. The assessment is usually carried out assuming there is no structural effect on the calculated ground movement. The six 'damage' categories are presented in Table 5.4.

When considering the acceptable movements for defining the onset of a limit state it is necessary to consider the function of the building and its use. Hence in Table 5.4 there is overlap between damage categories and limit states. These are provided for a typical building with a typical usage and by no means provide definitive classification of serviceability and ultimate limit states.

There are a range of similar damage classification systems (e.g. Institution of Structural Engineers guide *Subsidence of low rise buildings*[68]) which are based on the system in Table 5.4 but which have subtle changes. It is suggested that Table 5.4 be used and that the reference is quoted to prevent confusion in terms of damage category meaning. As in all cases the original paper should be consulted.

One of the most relevant examples of damage caused by construction for typical UK buildings is that of retaining wall and basement construction in urban areas where services, structures, and at the limit tunnels, may exist within the zone of influence of the excavation. Initial assessment of likely movement resulting from a basement excavation could use published information such as CIRIA C580 *Embedded Retaining Walls*[35] (normalised wall installation and excavation movements). Thereafter, assessment of

Table 5.4 Limit states and BRE damage categorisation

'Damage' category[a]	Possible limit state being infringed	Ease of repair	Illustrative crack size on vertical elements
Category 0 (Negligible)	None?	N/A	Hairline cracks <0.1mm
Category 1 (Very slight)		Repair during normal decoration.	Fine cracks in finishes <1mm
Category 2 (Slight)		Cracks easily filled. Some repointing externally.	Typical cracks between 1mm–5mm
Category 3 (Moderate)	Serviceability? (loss of water tightness, unacceptable appearance, doors/window sticking)	Opening up and patching. Repointing with small amounts of brickwork replaced.	Typical cracks up to 15mm, several cracks up to 3mm
Category 4 (Severe)		Breaking out and replacing sections of walls, especially at openings.	Typical cracks up to 25mm
Category 5 (Very severe)	Ultimate? (in a dangerous condition, risk of collapse)	Major repair requiring partial or complete rebuilding.	Typical cracks >25mm

Notes

a When using this table the reader should be aware of the subjective nature of the assessment ('ease of repair' and 'crack width') and the need to proceed with caution. The table cannot be used for buildings with unusual detailing.

b Table taken from Burland[67].

movement can be carried out using retaining wall design software or numerical modelling using characteristic parameters.

Assessment of damage to infrastructure and utilities is equally important and should be considered as part of the design.

5.4 Vibration and damage

While not an issue with regard to the overall stability of adjacent structures, it is necessary to consider vibration in design and construction planning. Vibration is often related to construction processes (driven piling, use of vibratory sources for casing extraction etc.) as well as from vibratory sources in complete buildings (working machinery). Guidance on levels of vibration causing damage to structures, or having impact on third parties, is given in CIRIA Technical Note TN142[69]. There are references to additional sources of information in the CIRIA document. BS 5228-2:2009[70] also provides useful information and is the basis of the data in Tables 5.5 and 5.6. The term 'ppv'

Table 5.5 Guidance on effects of vibration levels

Vibration level (ppv)	Effect
0.14mm/s	Vibration might be just perceptible in the most sensitive situations for most vibration frequencies associated with construction. At lower frequencies, people are less sensitive to vibration.
0.3mm/s	Vibration might be just perceptible in residential environments.
1.0mm/s	It is likely that vibration of this level in residential environments will cause complaint, but can be tolerated if prior warning and explanation has been given to residents.
10mm/s	Vibration is likely to be intolerable for any more than a very brief exposure to this level.

Note
This Table is taken from BS 5228-2:2009[70].

Table 5.6 Limits for transient vibration, above which cosmetic damage could occur

Type of building	Peak component particle velocity in frequency range of predominant pulse	
	4Hz to 15Hz	15Hz and above
Reinforced or framed structures Industrial and heavy commercial buildings	50mm/s at 4Hz and above	50mm/s at 4Hz and above
Unreinforced or light framed structures[b] Residential or light commercial buildings[b]	15mm/s at 4Hz increasing to 20mm/s at 15Hz	20mm/s at 15Hz increasing to 50mm/s at 40Hz and above

Notes
a Values referred to are at the base of the building.
b At frequencies below 4Hz, a maximum displacement of 0.6mm (zero to peak) is not to be exceeded.
c This Table is taken from BS 5228-2:2009[70].

or 'peak particle velocity' is used as a measure of the severity of vibration (amplitude and frequency); it is defined as the instantaneous maximum velocity reached by a vibrating element as it oscillates about its rest position (BS EN 5228-2:2009[70]). Measurements are often taken in different directions (vertical and horizontal).

As well as noise and vibration causing disruption to third parties and damage to structures, vibration can also cause permanent ground movements. Such ground movements are most likely to happen in loose and very loose soils (natural or more commonly made ground). Assessment of potential ground movement is a specialist task and recourse to a geotechnical specialist is recommended in the following instances:
– Where ground bearing structures are founded on loose or very loose coarse grained soils.

- When pile construction (with percussive or vibratory source) or other vibratory equipment is being considered on sites with adjacent structures (on or off the site) where these are founded on loose and very loose coarse grained soils (beware also differential settlement between service connections and structure).

5.5 Illustration of design process

Refer to Appendix B.

5.6 Summary

- Overall stability includes more than just the ability of a structural element to stand up; in particular it includes the following:
  - Loss of overall stability of the ground (natural slopes, cuttings, embankments) or associated structures (retaining walls/foundations) – this is a ULS consideration.
  - Excessive movements in the ground due to shear, settlement or heave that is not related to the project construction – these can be both SLS and ULS considerations.

- While not overall stability, is it also necessary to consider how proposed construction will impact on adjacent structures, namely:
  - Damage (beyond repair) of adjacent structures, roads or services due to movements in the ground due to the construction works – this is a ULS consideration of associated structures but requires prediction of movement to allow the assessment to be made.
  - Damage or loss of serviceability of adjacent structures, roads or services due to movements in the ground – this is a serviceability limit state consideration of associated structures.

- The assessment of a site and proposed construction includes the setting of the site, the structure being built, and the impact that this has on the adjacent structures. The assessment includes assessment of limiting strength and of limiting movements; it covers both serviceability and ultimate limit states.

6 Spread foundations

6.1 Introduction

In Chapter 2 four approaches to design were presented (by calculation, by prescriptive methods, by the observational method, and by experimental models and load tests). The design of spread footings in the UK typically falls into the categories of design by prescriptive methods and design by calculation (Clause 6.4(5)P EC7 Part 1[1]). However, EC7 also gives an option to design by an indirect method using field or laboratory measurements or observations; this method is addressed below in Section 6.6 for GC1 designs.

The reader should consider the contents of Chapter 7 on piled foundations and in particular Section 7.2 when identifying whether spread foundations or pile foundations are the most appropriate foundation system.

Spread foundations include pads, strips and rafts (but not piled rafts). Sections 6.2 to 6.5 below provide information on key considerations that must be addressed when designing spread foundations. These are followed by a section on prescriptive design (Section 6.6) which is also useful for preliminary design. Sections 6.8 and 6.9 provide information on design by calculation for ultimate and serviceability limit states respectively. Sections 6.10 and 6.11 complete the chapter and deal with interaction with structural design and executions standards. A design example is given in Appendix C.

While specialist ground improvement techniques are not addressed in detail in this *Manual* (see Section 9.4) they can be used in combination with spread footings to provide an economic foundation solution where spread footings on their own are non-viable and where recourse to piled foundations is not necessary. Underpinning is discussed briefly in Section 7.16.

6.2 Limit states

At the commencement of a design a list of limit states must be compiled, this list should be reviewed during the design for completeness. Table 6.1 presents the limit states applicable to spread footing design that should be considered as a minimum.

Table 6.1 Limit states to be considered as a minimum – spread foundations

– Loss of overall stability – Bearing resistance failure – Combined failure in the ground and structure – Movement leading to collapse or adverse appearance/reduced usability (including nearby structures or services) – Structural failure due to foundation movement – Excessive settlement – Unacceptable vibrations – Excessive heave – Failure by sliding – Punching failure, squeezing
Note Combinations of these limit states must be considered if relevant.

6.3 Design situations

In addition to the design situations presented in Chapter 2, the following additional design situations should be considered for spread footing design:

– Variation of ground conditions (soil properties, water levels, pore-water pressures) with location and anticipated variation of ground conditions with time.
– Variation in actions (vertical, moment, torsion and horizontal/shear) and how they are combined.
– Ground-structure interaction analysis where structural stiffness is similar to ground stiffness.

It is also necessary to consider the effects of chemicals and corrosion on foundation material durability. Details for design of concrete against sulphate attack are presented in Section 4.4.2.

6.4 Design considerations

In addition to the design considerations presented in Chapter 2, the additional design considerations in Table 6.2 must be addressed for spread footing design.

Table 6.2 Minimum design considerations – spread foundations

– Depth of adequate bearing stratum (do soft layers exist at depth?) – Variations in water level and its effect on excavation during construction – Effect of installation on nearby structures – Effect of future groundwork (e.g. service excavations) on foundation stability and movement – Integrity of foundation once installed – Environmental risk associated with contamination of aquifers and linking aquifers – Soluble materials within the ground – Changes in shallow ground conditions with time and depth (summer/winter effects of desiccation and swelling, frost depth[b], tree actions on clays, transmitted hot or cold temperatures, scour)[72] – Ground movements and reductions in ground strength due to seepage, climate or construction
Notes **a** Combinations of these limit states must be considered if relevant. **b** For frost damage to occur, the soil must be frost susceptible and the foundation un-insulated within the frost depth – refer to BS EN ISO 13793[71] for frost protection measures.

6.5 Actions and geometry

The basic actions presented in Chapter 2 are to be considered.

When considering the geometry of spread foundations, consideration of economic excavation, setting out tolerances, requirements for working space, size of the supported structure and other practical matters is required.

6.6 Design by prescriptive methods – pads and strips

6.6.1 Introduction

EC7 permits the use of presumed bearing resistance where there is adequate case history data to justify such a method. The method provides allowable bearing resistances (or pressures) which are generally assumed to satisfy both ULS and SLS requirements for conventional structures. The allowable bearing pressures should be compared with unfactored loads from the structure.

The following is based on current practice as would be applied to situations where explicit assessment of settlement would not usually be required to validate the design. The design rules presented are based on the now superseded British Standard as follows:
– BS 8004[31] Table 1 for design of spread footings on sand and clays (silts and gravels).

Similar rules are also available in BS 8004[31] as follows:
– BS 8004 Table 2 for design of spread footings on chalk and also CIRIA C574 *Engineering in chalk*[73].
– BS 8004 Figure 1 and Table 4 for design of spread footings on rocks (excluding chalk and Mercia Mudstone).
– BS 8004 Table 3 for design of spread footings on Mercia Mudstone.

The 'presumed bearing values' quoted in BS 8004[31] are compared to unfactored dead plus live loads (giving the representative load). If design resistances are to be assessed for comparison with a ULS design action (F_d, which includes partial factors on actions using Design Approach 1 Combination 2 factors as per Section 2.11.3.4 above) then adjustments can be made to the values quoted in BS 8004 to accommodate the difference in design methodologies.

Before presenting prescriptive design rules, it is noted that there are limitations to their usage. Examples of where prescriptive design should not be used are summarised below in Table 6.3. The list is not exhaustive and the designer must check that the design being undertaken is appropriate.

6.6.2 Clay soils – GC1 design

Prescriptive design of pads and strips bearing on clay strata is well documented for strata with uniform or increasing strength with depth and where the strength of the formation level is of medium strength or better (medium strength has a measured undrained shear strength in the range of 40 to 75kN/m^2). Where conditions are more complex then prescriptive design may not be applicable and the designer should consider design by calculation (see Section 6.8). Table 6.3 provides a list of situations where prescriptive design would likely be inappropriate.

For GC1 structures and for initial assessment of foundation size based on average undrained shear strength and unfactored loads the presumed bearing resistance q_a can be obtained from Figure 6.1 which is based on BS 8004[31] suggested values. This figure also accommodates the EC7 requirement that if the footing has a bulked factor of safety of three or more then no settlement analysis will be required for conventional structures. The choice of undrained shear strength should be in keeping with BS 8004 and can be taken to be a moderately conservative value. In Figure 6.1 the 'presumed bearing resistance' can be compared with the unfactored load (vertical action).

Figure 6.1 has been revised in Figure 6.2 to be in keeping with EC7 terminology whilst maintaining a bulked factor of safety of three thereby limiting the need for calculation of settlement. In Figure 6.2 the design resistance, R_d is the design load with case A2 (Table 2.7a) partial factors applied to the actions and the undrained shear strength, $c_{u;k}$ is the characteristic undrained shear strength of the clay.

Table 6.3 Limitations for use of prescriptive spread foundation design

Situation	Reason for not using prescriptive design
Large loads	Where large loads are present requiring pad or strip foundations of more than 3m nominal width then assessment of settlement becomes more important as does the structural design of the foundation.
Inclined and eccentric loads	Where loads are inclined or eccentric to the centre of the footing then the design resistance will reduce due to geometric effects. Inclined loads also require a check for sliding.
Low strength soils (measured $c_u < 40kN/m^2$ for clays or SPT $N < 10$ for coarse grained soils)	– Risk of large settlement – Need early consideration of temporary works and construction plant stability.
Soil strata strengths reduce with depth below formation level	Such situations occur where weathered or desiccated crusts overlie weaker soils and where soft clays underlie sand and gravel strata. The risk of punching failure needs to consider the gradient of strength below the formation level as does the risk of larger than anticipated settlements.
Location near existing, planned or recently removed trees in clay strata	Need to consider ground heave/settlement to arrive at depth of foundation to overcome effects of trees[72].
Locations with water bearing layered soils	In layered water bearing sand (or silt) and clay soils the risk of rapid softening of the clay layers is increased due to short drainage path lengths where footings are placed in excavations or trenches.
Excavation below groundwater level in coarse grained soils	Where excavation is carried out beneath the water, softening/disturbance of the formation level results in uncertain settlement of the footing. Placement of concrete may also be difficult leading to structural problems – solution may be temporary dewatering.
Uneven or sloping ground	Where the spread footing is located close to a slope or an excavation, then the assumptions on which prescriptive design is based may no longer apply. In addition, overall stability may also be the controlling factor in design and stipulation of a bearing pressure should not be finalised until the full design is developed.
Heavily fissured clay	Where the clay is heavily fissured and disturbed, design needs to consider the mass strength rather than the intact strength that would typically be measured by small scale laboratory or field tests. An assessment of how the fissuring would affect ultimate bearing resistance would be required to allow the presumed bearing pressure to be assessed.
Displacements	Where structures are unusually sensitive to movement then prescriptive design is unlikely to be appropriate.

It is intended that the performance of foundations designed using Figures 6.1 and 6.2 will be similar.

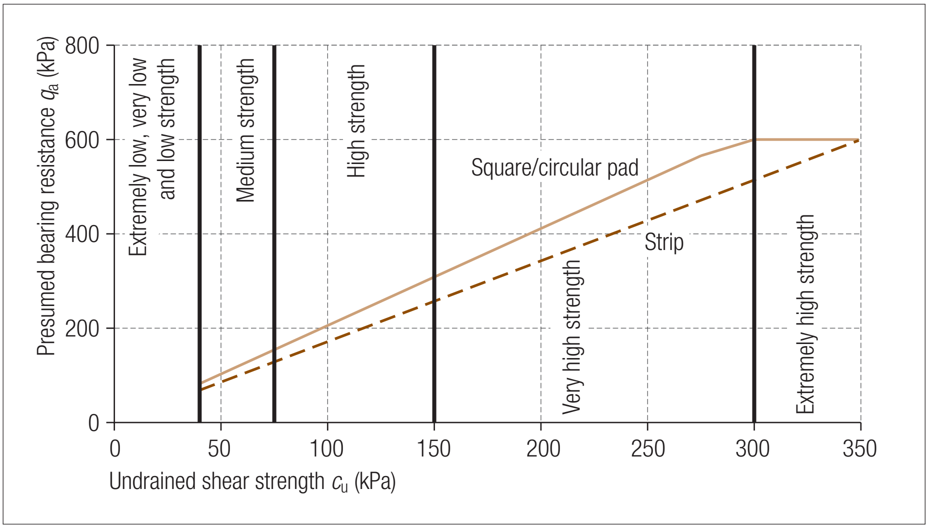


Fig 6.1 Prescriptive spread footing design for GC1 structures.

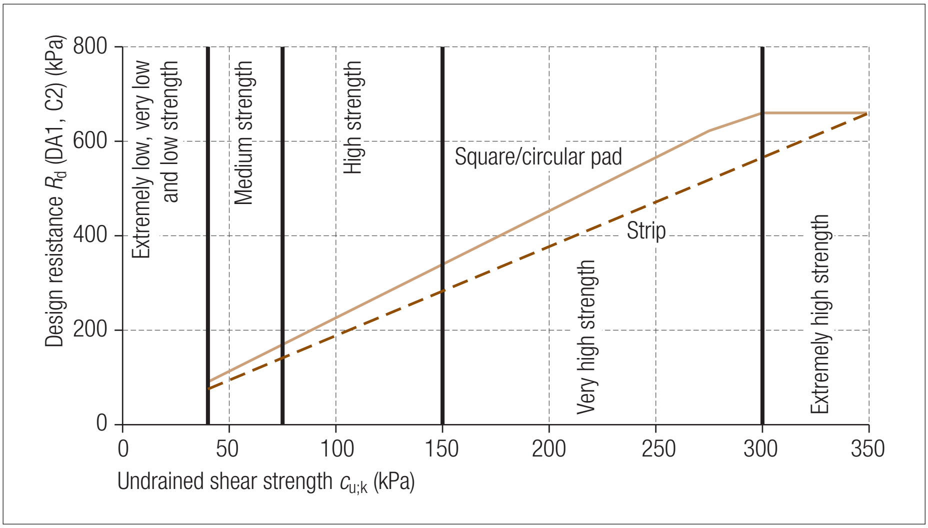


Fig 6.2 Spread footing design to EC7 – clays (bulk factor of safety 3.0)

The reader may note that the value for EC7 design resistance is higher than the BS 8004[31] presumed bearing value for a given soil strength, c_u or $c_{u;k}$. The reasons for this are that:

- The design resistance presented in Figure 6.2 includes Design Approach 1, Combination 2 factors as well as a check that there is a bulk factor of safety of 3.0 on the representative load thereby reducing the need for an explicit settlement calculation.
- The design resistance R_d in Figure 6.2 has a partial factor of 1.3 on variable actions which is absent for BS 8004 presumed bearing pressure assessment shown in Figure 6.1.

Whilst not necessary for GC1 designs, an assessment of settlement should be carried out for GC2 designs. This settlement assessment will use SLS partial factors on actions and unfactored ground stiffness values.

6.6.3 Sand and gravel soils – GC1 design

Where sand and gravel soils are encountered as competent strata without substantial clay layers close to the formation level and where ground conditions are seen to improve with depth, then the following approach to preliminary sizing of spread footings for GC2 structures and for design of GC1 structures may be taken.

The initial approach to the design of footings on sand and gravel is an empirically based relationship between allowable bearing resistance and SPT N value.

$q_a = f\,N_{60}$ for footings constructed above the water table

$q_a = 0.5f\,N_{60}$ for footings constructed at the water table

where:

q_a is the allowable or presumed bearing resistance in kN/m^2

N_{60} is the SPT N value (assuming 60% energy efficiency – see BS EN ISO 22476 Part 3[25])

f is an empirical factor as presented in Table 6.4.

Table 6.4 Values of f and q_a for spread footing design on sand and gravels

Soil density	SPT blow count	Empirical factor f		
		Sand	Sand and gravel	Gravel
Very loose	<10	N/A	N/A	N/A
Medium dense	10–30	10	20 decreasing to 10 ($f = 25 - N/2$)	20
Dense	30–50	10	10	20
Very dense	>50	$q_a = 500$kN/m^2	$q_a = 500$kN/m^2	$q_a = 1000$kN/m^2

Note
Values quoted are for construction above the water table. Use 50% of values for construction below the water table.

Where the spread footing formation level is below the water table (GC2) then temporary dewatering should be considered to allow excavation and placement of concrete in the dry, thereby negating the need to reduce bearing pressures.

These empirical relationships take account of settlement as well as bearing capacity failure of the footing. The 50% reduction for construction at/below the water table allows for the potential loosening of the formation and the risk of increased settlement at design pressure (i.e. the correction is largely a serviceability limit state precaution).

Design of pad and strips bearing on sand and gravel strata can also be based on characteristic values of φ' and bearing capacity factors as presented in Annex D of EC7 Part 1[1] along with assessment of settlement. This approach is presented below.

6.6.4 Rock

The design of spread foundation on rock must consider the following:
– how the rock mass may deform and what its strength is
– the effect of weak layers, bedding joints, other discontinuities, weathering, decomposition and fracturing
– disturbance of the natural state of the rock by construction activity.

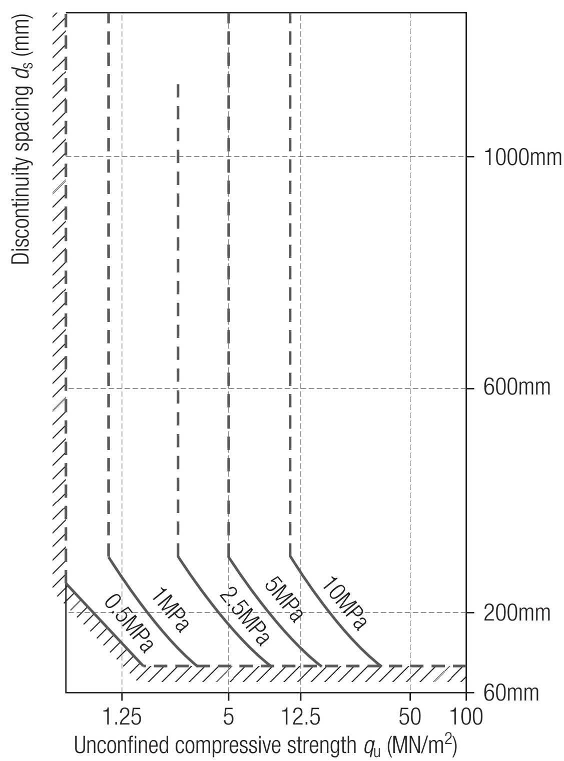
However, spread foundations on rock can normally be designed using presumed bearing resistances and the settlement assessed on the basis of comparable experience related to rock mass classification.

Presumed bearing resistances for vertical loading of square pads are presented in Figures 6.3a–6.3d.

EC7 is silent on the design of spread foundations on rock in terms of design by calculation. While designers are free to choose an appropriate method, it is likely that they will need to undertake a review as to whether additional partial factors are needed. Rock mechanics calculations often involve material parameters other than φ', c', c_u and q_u (e.g. GSI values or Hoek-Brown failure criterion parameters).

6.7 Ground bearing slabs

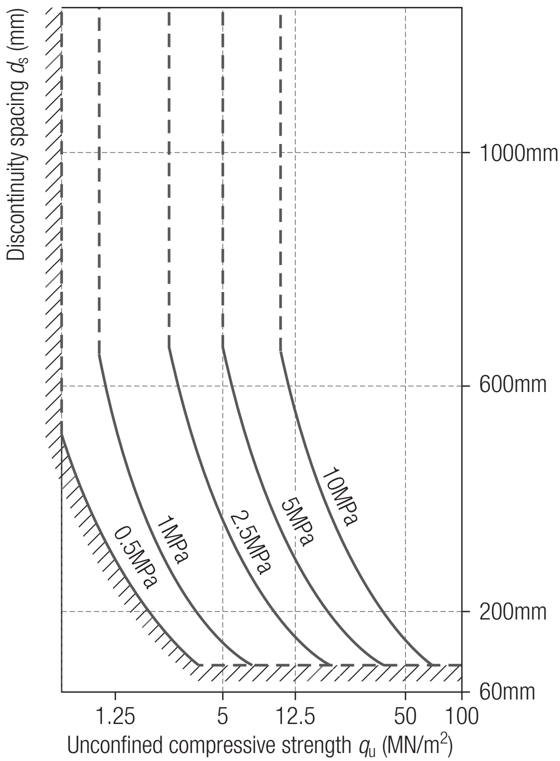
A special case of spread footing is that of a ground bearing slab. Such structures rely heavily on structural design to prevent punching failure of point or concentrated loads and need adequate strength/stiffness to prevent unacceptable differential settlement occurring which would result in a serviceability limit state failure (with a resulting impact on use of the structure). The Concrete Society report *Concrete industrial ground floors*[74] provides a useful reference document for the design of such structures; it may also be used for residential developments. Knapton's *Ground Bearing Concrete Slabs*[75] is also a useful reference document.



Notes

a Values on dotted lines to be reduced by 50% if rock joints are open.
b Solid line is presumed bearing resistance.
c Presumed bearing resistance may be applied to weak and broken rocks provided joints are tight.
d Weak and broken rocks with open or infilled joints should adopt a reduced presumed bearing resistance.
e It is assumed that the structure can tolerate settlements equal to 0.5% of the pad width. Presumed bearing resistance for higher settlements can be derived by direct proportion.
f Presumed bearing resistance in hatched areas is assessed by inspection and/or testing.
g A check should be made to ensure that the concrete strength does not govern.

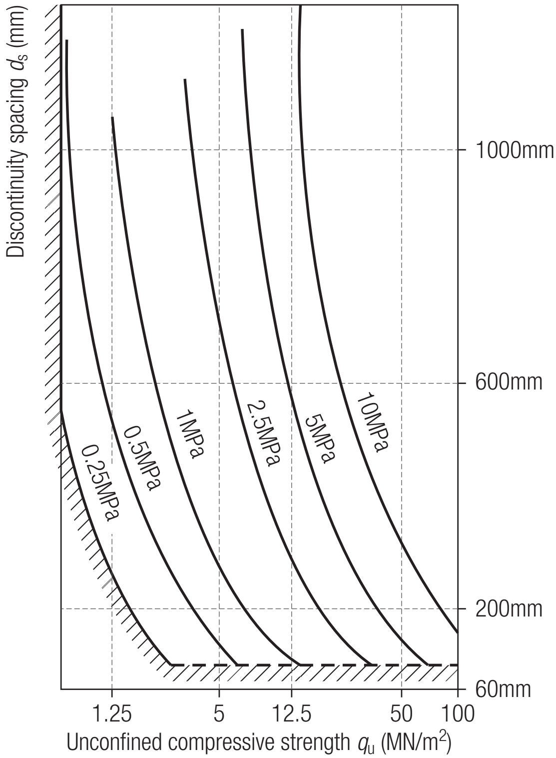
Fig 6.3(a) Presumed bearing resistance for vertical loading of square pads (after EC7): Group A – pure limestone and dolomite, carbonate sandstone of low porosity



Notes

a Values on dotted lines to be reduced by 50% if rock joints are open.
b Solid line is presumed bearing resistance.
c Presumed bearing resistance may be applied to weak and broken rocks provided joints are tight.
d Weak and broken rocks with open or infilled joints should adopt a reduced presumed bearing resistance.
e It is assumed that the structure can tolerate settlements equal to 0.5% of the pad width. Presumed bearing resistance for higher settlements can be derived by direct proportion.
f Presumed bearing resistance in hatched areas is assessed by inspection and/or testing.
g A check should be made to ensure that the concrete strength does not govern.

Fig 6.3(b) Presumed bearing resistance for vertical loading of square pads (after EC7): Group B – igneous rock, oolitic and marly limestone, well cemented sandstone, indurated carbonate mudstone, metamorphic rock (slate and schist with flat cleavage/foliation)



Notes

a Solid line is presumed bearing resistance.

b Presumed bearing resistance may be applied to weak and broken rocks provided joints are tight.

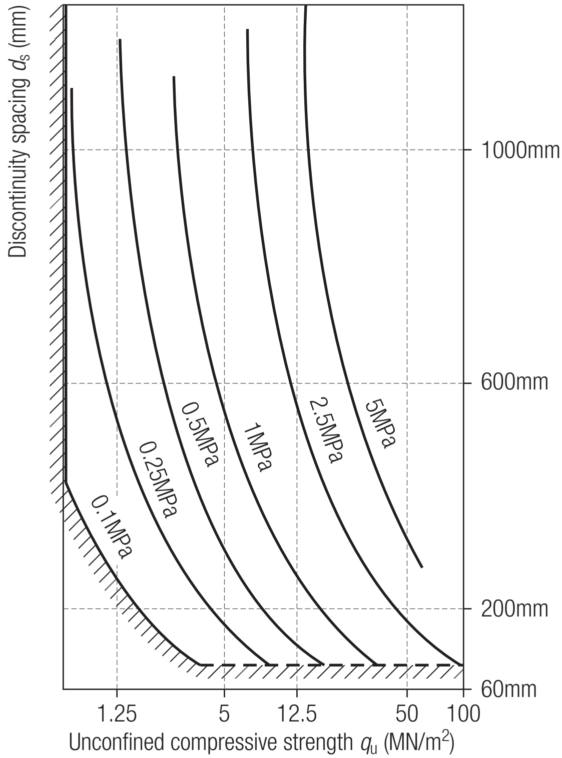
c Weak and broken rocks with open or infilled joints should adopt a reduced presumed bearing resistance.

d It is assumed that the structure can tolerate settlements equal to 0.5% of the pad width. Presumed bearing resistance for higher settlements can be derived by direct proportion.

e Presumed bearing resistance in hatched areas is assessed by inspection and/or testing.

f A check should be made to ensure that the concrete strength does not govern.

Fig 6.3(c) Presumed bearing resistance for vertical loading of square pads (after EC7): Group C – very marly limestone, poorly cemented sandstone, metamorphic rock (slate and schist with steep cleavage/foliation)



Notes

a Solid line is presumed bearing resistance.
b Presumed bearing resistance may be applied to weak and broken rocks provided joints are tight.
c Weak and broken rocks with open or infilled joints should adopt a reduced presumed bearing resistance.
d It is assumed that the structure can tolerate settlements equal to 0.5% of the pad width. Presumed bearing resistance for higher settlements can be derived by direct proportion.
e Presumed bearing resistance in hatched areas is assessed by inspection and/or testing.
f A check should be made to ensure that the concrete strength does not govern.

Fig 6.3(d) Presumed bearing resistance for vertical loading of square pads (after EC7): Group D – uncemented mudstone and shale

Conventional design of ground bearing slabs requires the following base assumption to be valid:

– The ground conditions beneath the slab are relatively uniform across the site.

This requirement would reject sites with variable thickness of soft material beneath the slab and sites with hard and soft spots as would be presented by old foundations and local poorly filled excavations respectively. It may be possible to improve sites by breaking down hard spots, removing soft spots or using ground improvement to increase the strength and stiffness of the ground to enable this requirement to be satisfied.

The design of a slab needs to consider a number of main criteria:

– Structural design should be carried out using modulus of sub-grade reaction *k* models or finite element models. Spring constants should be calculated with a bias towards the shallow soil strata to account for local distribution of column loads through the slab. Attention to soft strata is necessary as these may provide a large proportion of the settlement.
– The value(s) of spring constants *k* should be a conservative estimate of the value; several values may be used to account for variations across a slab or to provide a parametric study of the effect of geotechnical design parameters. Thereafter the design should be carried out in keeping with structural design to EC2[5], using appropriate partial factors on actions and structural materials.
– The appraisal of characteristic values of ground stiffness should be provided by the geotechnical engineer based on ground investigation data including *in situ* and laboratory testing.
– Assessment of bearing stresses at the edge of ground bearing slabs where the risk of bearing capacity failure is possible. Calculated bearing pressures from the Winkler Spring model (modulus of sub-grade reaction springs) should be compared with the allowable bearing pressures in Sections 6.6 (prescriptive design) and 6.8 (design by calculation).
– Long-term settlement (time related) assessment of stiffness should be included in the design.
– Calculation of overall settlement may be carried out using the load distribution provided by the Winkler Spring model and a geotechnical settlement computer program. The resulting settlement profile must be assessed for acceptability for the intended usage; consideration of bending moments resulting from combined calculation models should be made.
– Structural stiffness and properties used in the model must be appropriate to the level of bending and load type (permanent or transient).
– Where differential settlement is a critical consideration (e.g. in the case of warehousing with high level stacking systems), then it is recommended that the geotechnical design be carried out by a geotechnical engineer with relevant experience.

6.8 Design by calculation – ULS

6.8.1 ULS – general

The design of spread foundations by calculation is explicitly considered in EC7 for the ultimate limit state failure modes of bearing resistance and sliding resistance, as presented below. A designer is, however, still required to consider other relevant modes of failure (see Chapters 2 and 4).

6.8.2 ULS – bearing resistance

For bearing resistance it must be shown that:

$$V_d \leq R_d$$

where:
V_d is the design value of vertical load (or component of the total action normal to the foundation base) including the weight of the foundation, the weight of the backfill above the foundation, earth pressures and equilibrium water pressures (upper and lower bound value of water table may be considered)
R_d is the design value of resistance to V_d.

The design bearing resistance should be calculated by a commonly recognised method. The following analytical equations for undrained and drained conditions below may be used.

For undrained conditions

$$R_d / A' = (\pi + 2)\, c_{u;d}\, b_c\, s_c\, i_c + q$$

where:
R_d is the design resistance
A' is the effective base area ($B'L'$)
B' is the effective width of foundation
L' is the effective length of foundation
$c_{u;d}$ is the design undrained shear strength ($c_{u;d} = c_{u;k} / \gamma_{cu}$, see Section 2.11.3.7 for values of γ_{cu})
b_c is the base inclination factor
s_c is the shape factor
i_c is the load inclination factor
q is the overburden pressure at base.

B' (and similarly L') can be calculated as follows:

$$B' = B - 2e_b = B - 2\, M_d / V_d$$

where:
e_b is the eccentricity of vertical load in the direction of the width B

M_d is the design moment acting about the axis perpendicular to the width B, i.e. the moment about axis in direction of L

The base inclination factor is:

$$b_c = 1 - 2\alpha/(\pi + 2)$$

where:
α is the inclination in radians of the foundation base to the horizontal.

The shape factors are:

$s_c = 1 + 0.2\ (B'/L')$ for a rectangular shape

$s_c = 1.2$ for a square or circular shape

The load inclination factor is:

$$i_c = \frac{1}{2}\left(1 + \sqrt{1 - \frac{H_d}{A'\,c_{u;d}}}\right) \qquad \text{with } H_d \le A'\,c_{u;d}$$

where:
H_d is the design resultant horizontal load.

For drained conditions

$$R_d/A' = c'\,N_c\,b_c\,s_c\,i_c + q'N_q\,b_q\,s_q\,i_q + \tfrac{1}{2}\gamma'\,B'\,N_\gamma\,b_\gamma\,s_\gamma\,i_\gamma$$

where:
R_d is the design vertical resistance
A' is the effective base area (B'/L')
B' is the effective width of foundation
c' is the effective cohesion
N_c, N_q and N_γ are the bearing capacity factors calculated using φ'_d (where $\varphi'_d = \text{atan}(\tan\varphi'_k/\gamma_{\varphi'})$, see Section 2.11.3.7 for value of the partial factor
N_c is the factor on soil cohesion
N_q is the factor on the surrounding vertical effective stress at formation level soil
N_γ is the factor on soil buoyant density below footing level
b_c, b_q and b_γ are the base inclination factors
s_c, s_q and s_γ are the shape factors
i_c, i_q and i_γ are the load inclination factors
q' is the design effective overburden pressure at the level of the foundation base
γ' is the design effective weight density of the soil below the foundation level

Table 6.5 Bearing capacity factor values

Deg (°)	N_q	N_c	N_γ [b]	Comment
0[c]	1	5.14	0	Undrained
16	4	11	1	Drained
18	5	13	2	
20	6	14	3	
22	7	16	5	
24	9	19	7	
26	11	22	10	
28	14	25	14	
30	18	30	20	
32	23	35	27	
34	29	42	38	
36	37	50	53	
38	48	61	74	
40	64	75	106	

Notes
a All values rounded down.
b Value of N_γ based on $\delta \geq \varphi'_d/2$.
c Design based on c_u.

The effective dimensions B' and L' can be calculated as for undrained conditions above.

The values of the bearing capacity factors provided in Table 6.5 are based on these equations:

$$N_q = e^{\pi \tan(\varphi'_d)} \tan^2(45 + \varphi'_d/2)$$

$$N_c = (N_q - 1) \cot \varphi'_d$$

$$N_\gamma = 2(N_q - 1) \tan \varphi'_d$$

where:
$\delta \geq \varphi'_d/2$ (for rough base)

The base inclination factors are:

$$b_c = b_q - (1 - b_q)/(N_c \tan \varphi'_d)$$

$$b_q = b_\gamma = (1 - \alpha \tan \varphi'_d)^2$$

where:
α is the inclination of the foundation base to the horizontal (radians).

The shape factors are:

$s_q = 1 + (B'/L') \sin \varphi'_d$ for a rectangular shape

$s_q = 1 + \sin \varphi'_d$ for a square or circular shape

$s_\gamma = 1 - 0.3\ (B'/L')$ for a rectangular shape

$s_\gamma = 0.7$ for a square or circular shape

$s_c = (s_q N_q - 1)/(N_q - 1)$ for rectangular, square or circular shape.

The load inclination factors are:

$i_c = i_q - (1 - i_q)/(N_c \tan \varphi'_d)$

$i_q = (1 - H_d/(V_d + A'c' \cot \varphi'_d))^m$

$i_\gamma = (1 - H_d/(V_d + A'c' \cot \varphi'_d))^{m+1}$

where:
V is the vertical load
H is the resultant horizontal load

$m = m_B = (2 + (B'/L'))/(1 + (B'/L'))$ when H acts in the direction of B'

$m = m_L = (2 + (L'/B'))/(1 + (L'/B')$ when H acts in the direction of L'

In cases where the horizontal load component acts in a direction forming an angle θ with the direction of L', m may be calculated by:

$m = m_\theta = m_L \cos 2\theta + m_B \sin 2\theta$

Note that the partial factor on resistance, $\gamma_{R;v}$, is included in EC7 but has a value of 1.0 in the UK National Annex and has been omitted from the assessment of R_d above.

It is noted that the undrained and drained equations do not include factors for embedment (depth of foundation below ground level) or ground surface inclination. Whilst the omission of depth factors is conservative, the omission of ground inclination factors is not; this is particularly important for the situation where the ground falls away from the foundation. Help in establishing values for these factors may be sought in the work of Hansen[76,77], Vesic[78,79], Chen and McCarron[80] and Tomlinson[81].

For clay soils short and long term values of R_d are to be considered. However, for a conventional spread foundation on stiff clay at ground level (i.e. not in a basement excavation) and where a large bulk factor of safety (e.g. 3.0) has been used (as per that used in Figures 6.1 and 6.2) then there is rarely a need to consider drained bearing resistance. The long-term settlement should, however, be checked under SLS requirements (see Section 6.9.1 below). A drained analysis (or an undrained analysis with reduced shear strength to account for swelling) is appropriate for a spread foundation on clay which will soften as would occur in an excavation or where there is a rise in groundwater level.

For sand and gravel strata it is only necessary to consider drained conditions.

Where there is a defined structure to the ground strata below a foundation such as layering or discontinuities, the calculation model and parameters must include the effects of this structure. For foundations on distinctly layered soils, separate parameters are to be developed for each layer. Calculations for bearing resistance should take account of weaker layers, for example punching failure where a strong layer overlies a weaker layer. Numerical methods may be more appropriate than analytical methods for these situations.

6.8.3 ULS – sliding resistance

For sliding resistance it must be shown that:

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

where:

H_d is the design value of horizontal load (or component of the total action parallel to the foundation base) which includes design values of active earth pressures acting on the foundation

R_d is the design value of resistance on the base of the footing

$R_{p;d}$ is the design value of resistance from earth pressure on side(s) of foundation (see Section 8.15 where interface frictions are discussed).

The design sliding resistance is calculated as follows for undrained and drained conditions respectively:

$$R_d = A_c \, c_{u;d}$$

$$R_d = V'_d \tan \delta_d$$

where:

A_c can conservatively be taken to be the effective base area of the footing ($A' = B'L'$) to maintain compatibility with the vertical bearing capacity which uses effective area rather than the alternative approach which takes A_c to be the area beneath the footing under compression

$c_{u;d}$ is the design value of undrained shear strength (i.e. $c_{u;k}/\gamma_{cu}$)

V'_d is the design value of effective vertical action (or component of the total action normal to the foundation base)

δ_d is the design value of structure-ground interface friction angle (may assume $\delta_d = \varphi_{cv;d}$ (design value of effective critical state angle of shearing resistance) for cast *in situ* concrete or $\frac{2}{3}\varphi_{cv;d}$ for precast concrete).

Note that the partial factor on resistance $\gamma_{R;h}$ is included in EC7 but has a value of 1.0 in the UK National Annex and has been omitted from the assessment of R_d above.

Whilst the prescribed use of φ_{cv} may be logical given its relationship to φ' and ultimate limit states, determining its value by means of laboratory testing is not straightforward. However, some guidance is given in Section 4.8 of PD 6694-1:2011[34] and Section 2.2 of BS 8002[30] along with example values for clays and sands/gravels.

For undrained conditions, if water or air can reach the structure-ground interface then the following inequality which relates to situations where the horizontal load is high compared with the vertical load must also be satisfied:

$$R_d \leq 0.4\,V_d$$

Additional considerations for sliding resistance include:
– The possibility of shrinkage of clay soils away from a foundation due to seasonal movements of the ground or that soil adjacent to a foundation may be removed by human activity or erosion, particularly when a value greater than zero is adopted for $R_{p;d}$.
– Whether H_d and V'_d are dependent or independent actions when determining V'_d.
– Anticipated movements and the life of the structure when calculating design values of resistance.

6.8.4 ULS – geometry

Where the loading lies in the shaded areas shown in Figure 6.4, the design values of actions must be carefully reviewed and construction tolerances taken into account.

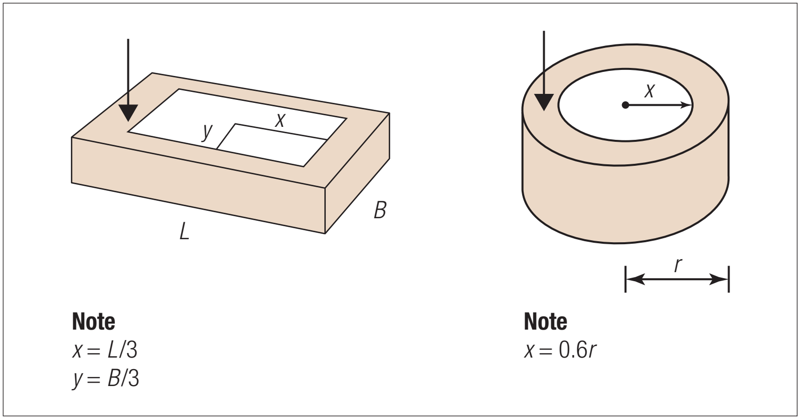


Fig 6.4 Illustration of where loading geometry becomes critical (shaded areas)

Construction tolerances of 0.1m should be considered for 'normal' construction practice; less for tighter control.

6.8.5 ULS – overall stability

Overall stability is addressed in Chapter 5. However, with respect to spread foundations overall stability (with or without the foundations) needs to be checked particularly when near or on a slope, near an excavation or retaining wall, near a water body, or near buried structures.

6.9 Design by calculation – SLS

6.9.1 SLS – general

Further to Section 2.11.4, verification of serviceability limit states is to:
- take into account displacements caused by actions
- consider displacements in light of comparable experience
- involve calculations for settlement estimation in soft clays
- consider total displacement of a foundation and differential displacement of parts of a foundation
- consider the contribution of adjacent loading to stress increase in the ground
- consider the possible range of relative rotations
- take account of load distribution and ground variability when assessing differential settlements and relative rotations
- involve calculations for settlement estimation in firm and stiff clays
- consider that differential settlement normally occurs even if calculations predict uniform settlement.

For conventional structures on clays, the ratio of the bearing capacity R_d to serviceability loading E_d should be checked prior to undertaking settlement calculations:
- If $R_d/E_d \geq 3$ then calculations may not be necessary.
- If $R_d/E_d < 3$ then calculations should be undertaken.
- If $R_d/E_d < 2$ then calculations should consider non-linear stiffness.

However, in general the inequality introduced in Chapter 2 (Section 2.11.4.1) applies:

$$E_d \leq C_d$$

In the following sections, the serviceability limit state is discussed in terms of settlement and heave as these are explicitly addressed in EC7. However, it should be kept in mind that the serviceability limit state can be more than just an assessment of vertical movement (e.g. horizontal movement, vibration).

6.9.2 SLS – settlement

Total settlement (s_t or more generally E_d) is comprised of immediate and delayed components. The components are typically represented as:

$$s_t = s_0 + s_1 + s_2$$

where:
s_0 is the immediate settlement
s_1 is the consolidation settlement
s_2 is the creep settlement.

Note that the additional settlement due to self-weight compaction of soil is to be assessed where applicable, and the possible effects of self-weight, flooding and vibration on fill and collapsible soils, and the effects of stress changes on crushable soils should be considered.

Total (or immediate) settlement can be calculated by selecting appropriate values for stiffness parameters, for example an undrained elastic modulus and Poisson's ratio for short term behaviour for an overconsolidated clay.

The assessment of differential settlement must also be carried out by assessing the likelihood of variability of total settlement s_t due to the following:
- Variable loading across the structure (e.g. core and column loading).
- Variable foundation levels across the structure (e.g. part of the structure with a basement or on piles).
- Variable ground conditions (e.g. infilled excavation beneath part of building footprint).
- Settlement trough resulting from a flexible foundation (e.g. a flexible raft).

Settlement should be evaluated using commonly recognised methods. For instance, the stress-strain method (compute stress distribution, compute strains from stresses and stiffness moduli, integrate strains to compute a settlement) as implemented in commercial geotechnical software may be appropriate for computing total settlement. Ground stiffness models may be either linear (often for over-consolidated soils) or non-linear (often for normally-consolidated soils), as appropriate.

Separation of the consolidation and creep components and their rates typically requires the interpretation of consolidation and *in situ* permeability tests. Caution should be exercised when evaluating these components for organic and soft clays.

The depth of ground considered for calculation of settlement should relate to the foundation size/shape, variation in ground stiffness and spacing of foundations. Typically, stress changes to a depth equal to two to three times the building dimension (i.e. not the individual pad or strip dimension) should be considered in uniform ground (more for very soft soils) or to the depth of a rigid boundary if closer (e.g. competent rock). Nevertheless, the designer should make an assessment as to where stress increase below a foundation is no longer significant.

While not covered in this *Manual*, designers of buildings where there are vibratory sources or dynamic loading should consider how such loading could result in geotechnical movement beyond those considered for monotonic loading. The reader is referred to additional information in *Design and Structures and Foundations for Vibrating Machines*[82] for an introduction to design, CP 2012: Part 1: 1974 *CoP Foundations for Machinery*[83] and DIN 4024: 1988 *Machine Foundations, flexible structures which support machines with rotating elements*[84].

6.9.3 SLS – ground heave

Heave is comprised of immediate and delayed components and is to be characterised as being caused by:
- reduction of effective stress (e.g. unloading, excavation or rise in groundwater level)
- undrained heave (e.g. due to excavation of overlying layers)
- external sources (such as frost action or effects of trees).

6.9.4 SLS – effect of trees

For structures founded on clay soils subject to the effects of trees (proposed, present or recently removed) careful consideration of ground movements must be carried out to address potential future movements (settlement, heave or sideways). Guidance for construction in the UK is given in the NHBC Standard Part 4 *Foundations*[72]. Chapter 4.2 of the NHBC document provides guidance for construction in the vicinity of trees with attention paid to the criteria in Table 6.6.

The NHBC document[72] provides illustrative details of how to accommodate tree induced ground movement on foundations by means of increased depth of foundation and measures that may be used to isolate the foundation from

Table 6.6 Key considerations – building near trees

Type of tree	Different trees have different water demands and thereby have the potential to cause varying levels of suction to varying depth below ground level.
Nature of ground	Clay soils are the key soil type at risk. The NHBC document[72] splits clay soils into 3 categories (high, medium and low volume change potential).
Depth of foundation and distance from tree	The deeper the foundation and the further the foundation is away from the tree the lower the risk.

shallow settlement/heave movements as well as horizontal ground movement and pressures.

6.10 Interaction with structural design

The structural design of spread foundations is covered in the Institution's *Manual for the design of concrete building structures to Eurocode 2*[85].

In addition to the general statements made elsewhere in this *Manual* on design interaction between structural and geotechnical engineering the following issues are relevant to spread foundations:

- The movements of spread foundations are to be considered to ensure they do not lead to an ultimate limit state in the supported structure.
- Foundations and supported structure must be designed to accommodate or resist heave in the ground that has the potential to swell.
- Serviceability must be checked using SLS loading and an appropriate distribution of bearing pressure.
- Allowance should be made for variable ground causing differential settlement unless the structure prevents it.
- The distribution of loads and ground variability is to be considered when assessing differential settlements and relative rotations.
- The stiffness of a spread foundation will affect the distribution of bearing pressure and a suitable method of modelling the distribution should be adopted. For a stiff foundation a linear distribution of bearing pressure may be adopted, while for a flexible foundation, the distribution can be derived using a continuum or spring model. Ignoring structural stiffness tends to lead to over-prediction of differential settlement.
- An estimate of tilt caused by settlement under eccentric loading can be found by adopting a linear bearing pressure distribution and then calculating the settlement at each corner.
- Ground-structure interaction may be generally undertaken using the modulus of sub-grade reaction spring or continuum models (a layered elastic half space representing the ground). When either the ground or the structure is complex more sophisticated models may be necessary (e.g. finite element analysis).
- Table 6.7 illustrates the general hierarchy of ground-structure interaction models and when they may be used appropriately.
- Also refer to Section 7.14 on interaction with tunnels.

6.11 Execution standards

There are no execution standards applicable to spread foundation design.

Table 6.7 Options for design complexity – spread foundations

Ground-structure interaction model	Appropriate use
Rule of thumb design/design charts for spread (pad) footings[85]	Regular geometry spread footings
Flexible raft with uniform or linear stress distributions vs. rigid raft with non-uniform stress distributions[86] using closed form solutions	Design of raft foundation reinforcement (shear and bending moment) allowing for uniform loading and ground stiffness
Iterative solutions using proprietary software incorporating ground (modelled with elastic constants) and structure accounting for structural form	Design of raft foundation reinforcement (shear and bending moment) allowing for non-uniform loading and variable structure/ground stiffness
Numerical analysis of ground and structure in 2D and 3D	Category 3 structures, with high degree of structural complexity and risk

Structural design of spread footings is introduced in the Institution's *Manual on EC2*[85].

6.12 Illustration of design process

Refer to Appendix C.

6.13 Summary

– Spread foundations include pads, strips and rafts.

– The design of spread foundations is generally undertaken using prescriptive methods or by calculation rather than by field testing or observation.

– Various ultimate and serviceability limit states need to be verified. In terms of a vertically loaded pad this could comprise a check for bearing resistance and settlement.

– Rock is only dealt with in terms of prescriptive design in EC7.

– Calculation methods for bearing and sliding resistance on soil are defined.

– Eccentric loading is described in terms of a middle two-thirds rule using design ULS loading rather than the traditional one-third rule using unfactored loading.

– Use of the constant volume angle of friction φ'_{cv} is more common in EC7 than traditional practice.

7 Pile foundations

7.1 Introduction

This chapter deals with the design of load bearing piles and is based on Chapter 7 of EC7 Part 1[1].

In Section 2.4 the four general EC7 approaches to design are presented, namely design by:
– calculation
– prescriptive measures
– the observational method
– experimental models and load tests.

For pile design, EC7 allows design to be based upon one of four sub-approaches:
(1) Empirical or analytical calculation (demonstrated to be valid by means of static load testing in comparable situations).
(2) Static load testing (demonstrated to be consistent with relevant experience by calculation or otherwise).
(3) Dynamic load testing (demonstrated to be valid by means of static load testing in comparable situations).
(4) Observed performance of comparable piles (supported by site investigation and ground testing).

Design of piles in the UK typically falls into the category of design by calculation, usually using empirically based relationships for base and shaft resistance, i.e. item (1) above. This chapter on pile design focuses on this approach but also provides an overview of the other design approaches for pile design. The chapter is divided up as follows:
– Section 7.2 discusses the relative merits of spread footings (Chapter 6) versus piled foundations.
– Sections 7.3 to 7.5 present the key considerations that should be addressed when designing piles to EC7 (fulfilment of these sections should demonstrate appropriate design rigour).
– Section 7.6 introduces pile load testing within the context of EC7.
– Section 7.7 presents preliminary options for pile types and dimensions for different site location and ground conditions.
– Sections 7.8 to 7.16 present detailed design of piles.
– Section 7.17 presents relevant construction standards and tolerances.
– Section 7.18 presents geotechnical aspects of design for varying ground types and pile types. This data will feed into the calculation processes presented in Section 7.9.

– Sections 7.19 and 7.20 complete the chapter with reference to a design example and a summary.

7.2 Spread footing or piled foundation?

The choice of foundation solution for a project will be an iterative process between applied loads, acceptable settlement and required safety. These basic 'engineering' considerations are then joined by considerations of costs, construction issues, environmental impact and constraints, programme implications and sustainability.

When addressing the basic engineering considerations it is necessary to identify the optimum foundation solution by means of experience and preliminary assessment. It is suggested that the order in which foundation types are considered in a design is as shown in Table 7.1, with the likely solution being the earliest choice which works.

Consideration of both typical and alternative foundation solutions may also be appropriate depending on ground conditions and loading conditions.

In order to work through this foundation choice process the following information will need to be collated and appraised as part of the preliminary investigation (desk study) and scheme design:
– Nature of ground, choice of ground parameters which will in turn be used to provide preliminary spread footing and pile design resistances.
– Magnitude of applied actions.
– Specific SLS and performance requirements.
– Location of site and special considerations.

Section 6.6 presents guidance on preliminary design of spread footings. If sensible sized footings can be designed then it is suggested that consideration of spread footings rather than piled foundations be made prior to the adoption of piled foundations.

Further reading on the selection of spread footings or piled foundations can be found in publications such as *Foundation design and construction*[81].

Table 7.1 Choice of foundation

	Typical solution	Alternative solution
First choice	Pad or strip foundations	–
Second choice	Pad or strip foundations associated with ground improvement	Raft with/without ground improvement
Third choice	Piled foundation	Piled raft foundation

7.3 Limit states

Designers are required to compile a list of limit states. Table 7.2 presents those limit states which, as a minimum, are to be considered for pile design.

Table 7.2 Limit states to be considered as a minimum – pile foundations

– Loss of overall stability (surfaces through and below piles) – Compressive or tensile resistance failure for a pile or pile group – Combined failure in the ground and structure, or ground and pile – Excessive settlement, heave or lateral movement – Unacceptable vibration passed into structure – Structural failure in compression, tension, bending, buckling or shear – Failure in uplift (individual and blocks) – Failure due to transverse loading – Movement leading to ultimate or serviceability limit states in a supported or nearby structure or service

7.4 Actions and design situations

Piles can be loaded axially or transversely; soil structure interaction analysis may be necessary to define the full load case. Notwithstanding the actions and design situations presented in Chapter 2, actions due to ground displacement relative to a pile are also to be considered.

Ground can be subject to displacement as a result of consolidation, swelling, adjacent loading, creep, landslide or earthquake and this displacement can lead to downdrag (negative skin friction), heave/stretching or transverse loading on a pile (see Section 7.3.2 of EC7 Part 1[1]).

Where pile design involves the movement of ground resulting in the mobilisation of resistance in the pile there are two options available:
(1) The ground displacement may be used directly as an action (the structure deforms as the ground displaces); or
(2) An upper bound force can be derived from the ground displacement to use as a direct action.

Values of characteristic ground strength and stiffness of moving ground, for use when calculating design ground parameters, are usually upper characteristic values as these will result in larger actions on the pile. It is noted that when using option (2) above to derive an upper bound force, the soil strength and the source of the load should be taken into account.

Further information is presented for downdrag in Section 7.12.1, and transverse loading in Section 7.12.4.

7.5 Design considerations

While the general design considerations have been presented in Chapter 2, Table 7.3 gives those to be considered specifically for piles.

Table 7.3 Minimum design considerations – pile foundations

– How piles and pile groups behave with respect to the strength and stiffness of the structure connecting them. – Duration and variation in time of loading with respect to the selection of calculation method and parameter values. – Planned future excavation/filling or potential changes in the groundwater regime. Effect of trees or their removal. – The choice of pile type, material quality and installation method is to consider: ground/groundwater conditions, obstructions, installation stresses, ability to preserve/check integrity, effects of installation on existing piles and adjacent structures, installation tolerances, chemicals in the ground, connection of different groundwater regimes and handling/transport. – When considering the choice of pile type, material quality and installation method, the following should receive attention: pile spacing, movement/vibration of adjacent structures, type of hammer/vibrator, dynamic stresses, the need to keep drilling fluids at a particular level, base/shaft cleaning, local instability during concreting, soil inclusions, groundwater flowing through wet concrete, extraction of water from concrete by unsaturated ground, effects of chemicals, soil compaction due to driving and soil disturbance due to boring.

7.6 Pile load tests

7.6.1 Static tests

7.6.1.1 Introduction

There are two types of pile load tests generally used in design and verification of piles. First, trial (preliminary) pile load tests are used to confirm or inform design assumptions to allow pile design to be carried out. Second, working pile tests are used to verify that the pile design is compatible with the construction method used and to verify settlements at working load are within anticipated limits.

While detailed discussion of pile load testing specification and interpretation is beyond the scope of this *Manual*, it is necessary to have an appreciation of the subject in order to understand its place in pile design and derivation of pile design methods. For further guidance on load testing, advice should be sought from a geotechnical specialist.

7.6.1.2 Trial piles

Trial pile load tests are undertaken to inform the design process. Trial pile load tests usually require an initial outlay of capital prior to the commencement of construction. This upfront cost allows refinement of the geotechnical pile design and provides an increase in confidence of the resulting design. There is also a cost benefit from pile load testing, as will be seen in Section 7.9.2, by means of a reduction in partial factors used in calculation of the pile design resistance.

EC7 requires that trial pile load tests are undertaken when:
- Using a pile type or installation method for which there is no comparable experience.
- Using a pile which has not been tested under comparable ground and loading conditions.
- Loading a pile in such a way that theory and experience do not provide sufficient confidence in the design.
- Observations during installation indicate a strong and unfavourable deviation in pile behaviour from that anticipated and when additional ground investigation does not clarify the situation.

For axially loaded piles, trial pile load tests are taken to the calculated ultimate capacity of the ground (the resistance at which the pile design method predicts geotechnical failure of the pile) or beyond to allow for variations in mobilised resistances in pile shaft and base compared to that assumed in the design. Conversely for transversely loaded piles, trial load tests are not normally taken to geotechnical failure; structural failure would normally be expected to precede geotechnical failure.

The number of trial piles necessary to verify a design depends on the ground conditions, the geotechnical category, comparable experience and the range of type of piles in the design.

For large piles it may be impractical to test a full size trial pile; in such instances the diameter of the trial pile should be no smaller than 0.5 the diameter of the working pile. All other details of construction should be the same.

7.6.1.3 Working piles

Working pile load tests are usually taken to 1.5 times the representative load. They are used to confirm that the pile installation method is in keeping with the design assumptions and to demonstrate acceptable settlement at working load. Working pile load tests are not taken to geotechnical failure.

The number and location of working test piles are selected on the basis of the number of piles installed (1% of piles is an appropriate starting point) and on the basis of pile installation records. It is usual to choose the working piles for testing after installation assuming that this is possible.

7.6.2 Dynamic tests

Dynamic load testing can be used to estimate compressive resistance provided there is adequate site investigation data and the method of testing has been calibrated against static load testing on the same type of pile (with similar dimensions) and in comparable ground conditions. Dynamic load testing can also be used as a quality control measure to check for consistency across a group of piles. Dynamic pile load testing is common for driven piles; it is also occasionally used for smaller cast-in-place piles.

7.6.3 Reporting

A report is required for all pile load tests and is to be specified accordingly (requirement to EC7 Part 1 Clause 7.5.4[1]). Consultation with a geotechnical specialist should be made for further development or requirements.

7.7 Preliminary design

7.7.1 Introduction

This section presents initial thoughts on the choice of pile type that would be appropriate at initial development of a foundation solution. The section stops short of design which is addressed in Section 7.8.

The comment in Section 7.2 suggesting that the reason for using piled foundations is primarily that spread footings are inappropriate is restated. Hence if shallow ground strata (say within 2 to 3m of ground surface) are inadequate for support of the structure with reference to the allowable bearing pressures in Section 6.6 then consideration of piled foundations, or ground improvement (see Section 9.4 for introductory comments), would be appropriate.

7.7.2 Pile types and sizes

The choice of type of piled foundations relies on multiple considerations including the ground conditions, the groundwater conditions, the location of the site relative to sensitive receptors and structures (a sensitive receptor being human, ecological or animal which may be impacted on by the works, e.g. a residential area will be sensitive to excess traffic, noise, vibration and dust), the type and magnitude of loading, and site constraints. Within this *Manual* there is insufficient space to present a detailed discussion on the choice of piles, however general pile types are identified along with principal advantages and disadvantages in Table 7.4. The table can be used as a basis for discussion with a piling contractor.

Table 7.4 Summary of common pile types – advantages and disadvantages

Pile type	Advantages	Disadvantages	Typical uses
Pre-cast concrete piles	– Should have high quality assurance in fabrication process – Installed through water and into ground with flowing water – With rock shoe allowing installation onto rock/sloping rock – Can be installed raked	– Causes vibration – Noisy unless installed with 'silent' hammer or piles jacked in – Limited section sizes (up to 0.45m square)	– Industrial sites – River and marine locations (small unsupported lengths) – Bridge abutments, piers – Working load up to 1.5MN
Driven steel H piles	– High quality assurance in fabrication process – Installed through water and into ground with flowing water – With rock shoe allowing installation onto rock/sloping rock – Can be installed raked	– Causes vibration when driven – Noisy unless installed with 'silent' hammer or piles jacked in – Can be expensive	– Industrial sites – River and marine locations (large unsupported lengths) – Bridge abutments, piers – Working load 1.5MN and greater
Driven steel pipe piles	– High quality assurance in construction process – Installed through water – Can be installed raked – Sizes up to 6m diameter	– Noisy unless installed with 'silent' hammer or piles jacked in – Causes vibration when driven – Can be expensive	– Marine uses – Working load, can be high for large diameter piles
Driven steel mini-piles	– High quality assurance in construction process – Can be delivered in short lengths for low head-room access	– Noisy unless installed with 'silent' hammer or piles jacked in – Causes vibration when driven	– Low head room locations, underpinning locations with low load bearing requirements
Auger bored cast-in-place piles	– A much used method – Sizes from mini to large diameter (0.15 to 2.1m) – Installed to large depths (70m+) – Can be installed in all materials with appropriate support of bore (bentonite, casing etc.) – Can be base grouted in sands or under-reamed in clays or shaft grouted	– Require assessment of integrity (especially for piles cast under bentonite) – Produce spoil which needs to be disposed of (important if ground is contaminated)	– All types of piles installed on land into soils and soft rocks – Working load up to 20MN

Table 7.4 Continued

Pile type	Advantages	Disadvantages	Typical uses
Continuous auger (includes continuous flight auger and segmental auger piles)	– Common product – Sizes from mini to large diameter (0.3 to 1.2m)	– Require assessment of integrity – Maximum length 27m – Produce spoil which needs to be disposed of (important if ground is contaminated) – Construction risk in very soft soils	– All types of piles installed on land into soils and soft rocks – Working load up to 6MN
Percussive rotary piles	– Can be installed in low head room and though existing foundations – Can be raked	– Noisy unless installed with 'silent' hammer or piles jacked in – Causes vibration when driven	– Underpinning (e.g. pali radice)
Rotary displacement piles	– Efficient means of mobilising ground resistance – Reduced production of spoil	– Proprietary design methods (specialist) – Limited data or experience in a wide range of soil types (albeit this is increasing all the time)	– Small diameter piles (e.g. up to 0.5m diameter)
Screw piles	– Factory produced elements (screws) – Can be installed raked – No excavated soil – Can be recovered and reused	– Refusal on obstructions – Developing design methodology	– Light weight structures – Temporary structures

When using Table 7.4 the reader must consider the following in identifying appropriate pile types:

– *Load carrying capacity* – Efficient structural pile design usually involves optimising concrete usage whilst incorporating a construction methodology which is appropriate to the ground conditions being encountered. A pile section should be chosen that optimises use of concrete strength. As a first pass for compression, the pile section size (area of pile) can be based on the representative pile load divided by 30% of the concrete cylinder strength. Design for bending moment etc. will need to be carried out using EC2 procedures[5] which include additional material factors on concrete strength for cast-in-place piles and reduced effective pile diameters. Tension pile design will be based on steel reinforcement alone. Consideration of downdrag is also necessary for assessing structural actions and pile displacements.

– *Installation methodology* – Support of the pile bore is critical in order to prevent ground movements outside the pile bore and to provide a pile which is capable of carrying load. In clays, open bores can be used below a length of temporary casing required to support surface layers and to cut-off shallow water thus preventing it from entering the pile bore (the casing is also necessary to provide a safe working area at piling platform level). In contrast piles in sand and gravel need full support as would be provided by continuous flight auger piles (also good for clays), piles constructed under bentonite or preformed piles.

– *Effects on adjacent/nearby third parties* – Noise and vibration preclude common types of driven piles being used at sensitive sites resulting in these types of piles only being suggested for industrial locations (jacked or pressed in piles may be permissible). Where it is considered that the use of driven piles is possible then it will be necessary to assess noise and vibration 'pollution' to ensure that it is within acceptable bounds; discussion with the local environmental health officer may be necessary. Consideration of adjacent properties and their uses as well as of protected animal species is necessary. (Also see comment in Section 5.4 above on noise and vibration.)
– *Consideration of aquifer protection* – Needs to be provided such that the choice of pile prevents contamination of aquifers both during construction and in the long term. Discussion with the Environmental Agency or other authority may be necessary.

Table 7.5 presents a preliminary indication of working loads that could be achieved with different pile types in different ground conditions; the abbreviations used are presented in the table notes. The table should be used for initial purposes and is in no way exhaustive. Refinement of pile types would usefully be carried out through discussions with a piling contractor or geotechnical engineer.

For detailed design (see Section 7.8) further consideration of pile type and pile dimension is necessary based on the following criteria:
– Geotechnical load carrying resistance, a ULS consideration.
– Settlement of pile and pile group, an SLS consideration.
– Structural performance in axial and bending modes.
– Construction limitations in terms of noise, vibration and risk of causing ground movements.

When each of these criteria is satisfied then a design may be considered to be complete.

Notes to Table 7.5

a It is assumed that sand and gravel are medium dense and clay stiff.

b Key to pile types:

AB	Auger bored piles, with (in sand/gravel) or without (clays) support fluid. Lengths up to 40m.
BBG	Bored base-grouted pile. Lengths up to 50m.
BUR	Bored, cast-in-place pile with under-ream. Lengths up to 30m.
CFA	Continuous flight auger. Lengths up to 27m.
DP	Driven pile.
PCD	Precast concrete driven pile. Lengths up to 25m.
PCD-R	Precast concrete driven pile with rock shoe. Lengths up to 25m.
RS	Drilled piled with rock socket. Lengths up to 20m.
SA	Segmental auger. Lengths up to 20m.

Table 7.5 Example pile sizes and types

Location	Working load	Bearing stratum[a]	Possible pile types, numbers and diameters Φ/square section		
Residential area/ sensitive receptors	Small (<1MN)	Sand/gravel	1 to 3 SA, 0.45m	1 CFA, 0.6m	
		Stiff clay/soft rock	1 to 3 AB 0.45m	1 CFA, 0.6m	
		Hard rock	1 RS, 0.4		
	Medium (3MN)	Sand/gravel	2 to 3 SA, 0.6m	1 or 2 CFA, 0.9m	
		Stiff clay/soft rock	2 to 3 BA, 0.6m	1 or 2 CFA, 0.9m	
		Hard rock	1 RS, 0.6m		
City centre	Medium (3MN)	Sand/gravel	2 to 3 CFA, 0.6m	1 or 2 CFA, 0.9m	1 AB with bentonite, 0.6 to 0.9m
		Stiff clay/soft rock	2 to 3 CFA, 0.6m	1 or 2 CFA, 0.9m	1 AB open hole 0.6 to 0.9m
		Hard rock	1 RS, 0.6m		
	Large (10MN)	Sand/gravel	3 to 5 CFA, 0.9m	1 BBG, 1.2m (dense sand)	
		Stiff clay/soft rock	4 to 5 CFA, 0.9m	1 BUR, 5.0m under-ream (in stiff/ very stiff clay)	
		Hard rock	1 RS, 1.2m		
Industrial/remote	Small (<1MN)	Sand/gravel	1 to 3 SA, 0.45m	1 CFA, 0.6m	1 to 2 PCD, 0.35m
		Stiff clay/soft rock	1 to 3 AB, 0.45m	1 CFA, 0.6m	1 to 2 PCD, 0.35m
		Hard rock	1 DP, 0.4m		1 PCD-R, 0.35m
	Medium (3MN)	Sand/gravel	2 to 3 CFA, 0.6m	1 or 2 CFA, 0.9m	2 to 4 PCD, 0.45m
		Stiff clay/soft rock	2 to 3 CFA, 0.6m	1 or 2 CFA, 0.9m	2 to 4 PCD, 0.45m
		Hard rock	1 RS, 0.6m		2 PCD-R, 0.45m
	Large (10MN)	Sand/gravel	3 to 5 CFA, 0.9m	1 BBG, 1.2 to 1.5m (very dense sand)	
		Stiff clay/soft rock	2 to 5 CFA, 0.9m	1 BUR, 5.0m under-ream (in stiff/ very stiff clay)	
		Hard rock	3 to 4 RS, 0.6m	1 RS, 1.2m	4 to 5 PCD-R, 0.45m

7.8 Detailed design

Whilst Table 7.5 is appropriate for initial indication of pile types and diameters it in no way constitutes a design.

In keeping with the methods available for verifying limit states presented in Section 2.4, EC7 identifies four design approaches, one of which is to be used in design:
- Ground test results (design by calculation) – this is the most common approach used for pile design in the UK (see Sections 7.9.3 and 7.15).
- Static load testing – this approach is not common in the UK but may be used where the measured capacity is justified by means of calculation (see Section 7.10).
- Dynamic load testing – this method is most applicable to driven piles, albeit specialist systems can be used in certain circumstances for cast-in-place piles (see Section 7.11).
- Observed performance (prescriptive design).

In contrast to design by calculation methods, design based on comparable experience (prescriptive design) is not advocated unless the design is backed up by appropriate documented case history data. Where such case history data is available its use should be limited to GC1 structures.

Design using ground test results is the most common approach used in the UK for all but driven piles where design by dynamic load testing also is often used once a preliminary assessment has been made by means of calculation or prescriptive means. When using design by ground test results (soil strength data), the calculations need not be complex but they must be validated by comparable experience of similar pile types, in similar ground conditions using similar construction practice.

7.9 Ultimate limit states – design by calculation

7.9.1 Introduction

Design by calculation is the most common way in which piles are designed in the UK. Design by calculation includes design using geotechnical data and includes the use of pile load tests. The basic design inequality relating the design action F_d to the design resistance R_d, for pile design is:

$$F_d \leq R_d$$

where:
F_d is the design action on a pile or a group of piles
R_d is the design resistance of a pile or group of piles

This inequality must be satisfied for all ultimate limit state load cases and load combinations.

Axially loaded piles
This above inequality can be developed for axially loaded compression and tension piles or pile groups as follows:

$F_{c;d} \leq R_{c;d}$ in compression
$F_{t;d} \leq R_{t;d}$ in tension

In all cases the design resistances and actions are factored values.

For compression piles the design considers both shaft and base resistances as below:

Design resistances of shaft and base are obtained independently from

Shaft: $R_{s;d} = R_{s;k}/\gamma_s$
Base: $R_{b;d} = R_{b;k}/\gamma_b$

leading to the design resistance of the pile $R_{c;d}$:

$$R_{c;d} = R_{s;d} + R_{b;d}$$

Alternatively for design by static or dynamic load test, the design resistance of a pile may also be calculated from the total characteristic resistance of the pile though this is less economic:

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

For tension piles the equations should clearly ignore base resistance resulting in the single equation:

$$R_{t;d} = R_{t;k}/\gamma_{s;t}$$

where:
$R_{b;d}$ is the design resistance of pile base (always compression)
$R_{b;k}$ is the characteristic resistance of pile base (always compression)
$R_{c;d}$ is the design resistance of combined base and shaft friction (compression)
$R_{c;k}$ is the characteristic resistance of combined base and shaft friction (compression)
$R_{s;d}$ is the design resistance of pile shaft friction (in compression)
$R_{s;k}$ is the characteristic resistance of pile shaft friction (in compression)
$R_{t;k}$ is the characteristic resistance of a tension pile or group of tension piles
$R_{t;d}$ is the design resistance of a tension pile or group of tension piles
γ_b is the factor on characteristic base resistance
γ_s is the factor on characteristic shaft resistance
$\gamma_{s;t}$ is the factor on characteristic shaft resistance in tension
γ_t is the factor on combined (shaft + base) characteristic resistance

Note that the EC7 distinction between subscripts for tension and combined are not always intuitive.

Transversely loaded piles
For transversely loaded piles or pile groups the following inequality can be developed:

$$F_{\mathrm{tr;d}} \leq R_{\mathrm{tr;d}}$$

where:
$F_{\mathrm{tr;d}}$ is the design value of transverse action
$R_{\mathrm{tr;d}}$ is the design resistance of a transversely loaded pile

EC7 is silent in terms of obtaining design values. Recourse to proprietary software is usually the optimum means for assessing pile bending moment and displacement.

7.9.2 Partial factors

Values for partial factors γ_{s}, γ_{b}, γ_{t} and $\gamma_{\mathrm{s;t}}$ vary for different pile types. Values are given in Tables 7.6 to 7.13 for assessment of the ultimate limit states EQU, STR, GEO and UPL for design in the UK; these tables build on those presented in Chapter 2. The NA to EC7 Part 1[44] allows these partial factors to be varied in particular circumstances, however it is recommended that a geotechnical specialist be contacted to do so.

In keeping with Section 2.11.3.4 the combinations of partial factors for actions, material strengths (see Section 2.11.3.4 for values) and resistances are as follows:

EQU: A(EQU) + M(EQU) + R(EQU)
STR/GEO: A1 + M1 + R1 (Design Approach 1, Comb. 1)
STR/GEO: A2 + (M1 or M2) + R4 (Design Approach 1, Comb. 2)
UPL: A(UPL) + M(UPL) + R4

Partial factors for the components in the above equations are presented in Tables 7.6 to 7.14.

Table 7.6 Partial factors for actions EQU (as per Table 2.6a)

Factors on actions: Set A (EQU)[a]	Permanent γ_G		Variable γ_Q		
	Unfavourable $\gamma_{G,sup}$	Favourable $\gamma_{G,inf}$	Leading (unfavourable)	Accompanying (unfavourable)	Favourable
γ	1.1	0.9	1.5	1.5	0.0

Note
a Terminology is taken from EC0 but is not included in EC7.

Table 7.7 Possible partial resistance factors for pile design for EQU (not in EC7)

γ_R factors: Set R (EQU)[b]	Driven piles[a]	Bored piles[a]	CFA piles[a]
Base (comp) γ_b	1.30/1.25	1.50/1.35	1.50/1.35
Shaft (comp) γ_s	1.25/1.15	1.30/1.20	1.30/1.20
Total (comp) γ_t	1.35/1.25	1.50/1.35	1.50/1.35
Shaft (tension) $\gamma_{s;t}$	1.50/1.35	1.50/1.35	1.50/1.35

Notes

a Two values are given for each partial factor. The first partial factor is to be used where there is no explicit verification of settlement (SLS consideration), e.g. no contract pile load tests. The second value would be used when contract pile load testing is carried out on at least 1% of piles to 1.5 times the representative load for which they are designed (SLS load) or where settlement (SLS) is not a concern for the pile performance, or if the settlement can be predicted in a manner which is at least as reliable as a pile load test (which is not usually the case).

b Partial factors are not given to all pile types. Where partial factors are not given the designer should choose the nearest comparable pile type and agree with the overseeing authority.

c General note: there are no partial factors for EQU pile design in the UK NA to EC7 or EC7 itself. These values are therefore not compliant with the UK NA to EC7. If used they should be agreed with the client or overseeing authority.

For the Design Approach 1 Combination 2 design of piles there are two options for the partial factor on Materials:

– Where the ground material strength provides stabilising resistance to the applied actions then value M1 is used. The resistance calculated is then factored down using appropriate Set R4 factors to obtain the design resistance.
– In contrast to this the Set M2 factors are used where the ground material strength is detrimental to pile performance (destabilising), as in the case of negative skin friction. In this instance Set M2 factors are used to increase the ground strength and thus increase the calculated design destabilising action. This action is then added to the externally imposed design action. See Section 7.11.1 on negative skin friction.

Table 7.8 Partial factors for actions STR/GEO DA1 Comb 1 (as per Table 2.7a)

Factors on actions: Set A1	Permanent γ_G		Variable γ_Q		
	Unfavourable	Favourable	Leading (unfavourable)	Accompanying (unfavourable)	Favourable
For use with Eqn 2.1	1.35	1.0	1.5	1.5	0.0

Note

Value of partial factors for Eqns 2.1a and 2.1b have not been included here for simplicity as they are rarely used. Values are in Table 2.7a.

Table 7.9 Partial resistance factors for pile design for Set R1 (STR/GEO)

γ_R factors: Set R1[a]	Driven piles	Bored piles	CFA piles
Base (comp) γ_b	1.0	1.0	1.0
Shaft (comp) γ_s	1.0	1.0	1.0
Total (comp) γ_t	1.0	1.0	1.0
Shaft (tension) $\gamma_{s;t}$	1.0	1.0	1.0

Notes

a Partial factors are not given to all pile types. Where partial factors are not given the designer should choose the nearest comparable pile type and agree with the overseeing authority.

b Factors from NA to EC7 Part 1[44], Tables A.NA.6, A.NA.7 and A.NA.8.

c General note: for geotechnical design, design to DA1 C1 is unlikely to dominate due to the relative magnitude of partial factors applied to resistances and actions when compared to DA1 C2 design.

Table 7.10 Partial factors for actions STR/GEO DA1 Comb 2 (as per Table 2.7a)

Factors on actions: Set A2	Permanent γ_G		Variable γ_Q		
	Unfavourable	Favourable	Leading (unfavourable)	Accompanying (unfavourable)	Favourable
γ	1.0	1.0	1.3	1.3 ψ_0	0.0

Table 7.11 Partial resistance factors for pile design Set R4 (STR/GEO)

γ_R factors: Set R4[a]	Driven piles[b]	Bored piles[b]	CFA piles[b]
Base (comp) γ_b	1.7/1.5	2.0/1.7	2.0/1.7
Shaft (comp) γ_s	1.5/1.3	1.6/1.4	1.6/1.4
Total (comp) γ_t	1.7/1.5	2.0/1.7	2.0/1.7
Shaft (tension) $\gamma_{s;t}$	2.0/1.7	2.0/1.7	2.0/1.7

Notes

a Partial factors are not given to all pile types. Where partial factors are not given the designer should choose the nearest comparable pile type and agree with the overseeing authority.

b Two values are given for each partial factor. The first partial factor is to be used where there is no explicit verification of settlement (SLS consideration), e.g. no contract pile load tests. The second value would be used when contract pile load testing is carried out on at least 1% of piles to 1.5 times the representative load for which they are designed (SLS load) or where settlement (SLS) is not a concern for the pile performance, or if the settlement can be predicted in a manner which is at least as reliable as a pile load test (which is not usually the case).

c Factors from NA to EC7 Part 1[44], Tables A.NA.6, A.NA.7 and A.NA.8.

Table 7.12 Partial factors for actions UPL (as per Table 2.8a)

Factors on actions: Set A (UPL)[a]	Permanent γ_G		Variable γ_Q		
	Unfavourable	Favourable	Leading (unfavourable)	Accompanying (unfavourable)	Favourable
γ	1.1	0.9	1.5	1.5	0.0

Note
a Terminology is taken from EC0 but is not included in EC7.

It is noted that the partial factors on resistances for UPL (Table 7.13) are the same as those used for GEO for tension piles (Table 7.11). This is as per the UK National Annex but considered not in keeping with the original values in EC7 where the partial factors for pile resistances in UPL are 20% less than those in GEO. The choice of partial factors in the UK National Annex reflects a more cautious approach to UPL compared to the other limit states than would be taken from the 'boxed values' of partial factors presented in EC7 Part 1[1].

7.9.3 Design using ground test results

7.9.3.1 Introduction

Pile design using ground test results (geotechnical data) is the most common way in which to calculate pile resistance. Within EC7 this approach to pile design allows two different methods to be used to calculate the characteristic resistance of a pile:

– The use of characteristic soil parameters (e.g. $c_{u;k}$) – this is conventional design in the UK.
– The use of continuous profiles of geotechnical data (e.g. CPT data) – this method of design is less common in the UK but is equally valid.

Table 7.13 Partial resistance factors for pile design for UPL

γ_R factors: from Set R4[a]	Driven piles[b]	Bored piles[b]	CFA piles[b]
Shaft (tension) $\gamma_{s;t}$	2.0/1.7	2.0/1.7	2.0/1.7

Notes
a Partial factors are not given to all pile types. Where partial factors are not given the designer should choose the nearest comparable pile type and agree with the overseeing authority.
b Two values are given for each partial factor. The first partial factor is to be used where there is no explicit verification of settlement (SLS consideration), e.g. no contract pile load tests. The second value would be used when contract pile load testing is carried out on at least 1% of piles to 1.5 times the representative load for which they are designed (SLS load) or where settlement (SLS) is not a concern for the pile performance, or if the settlement can be predicted in a manner which is at least as reliable as a pile load test (which is not usually the case).
c Factors from NA to EC7 Part 1[44], Tables A.NA.6, A.NA.7 and A.NA.8.

Table 7.14 Partial factors for material strength – ground properties

Set	Partial factors applied to material strengths			
	γ'_{φ}	γ'_{c}	γ_{cu}	γ_{qu}
M(EQU)[a]	1.1[b]	1.1	1.2	1.2
M1	1.0[b]	1.0	1.0	1.0
M2	1.25[b]	1.25	1.4	1.4
M(UPL)[a]	1.25[b]	1.25	1.4	1.4

Notes

a Terminology is taken from EC0 but is not included in EC7.
b Applied to tan φ'.
c Factors from NA to EC7 Part 1[44], Tables A.NA.2, A.NA.4 and A.NA.16.

If no relevant historical data and back-analysis exist for a similar pile, then it will be necessary to carry out new pile load test(s) to the calculated unfactored pile resistance. The testing will validate the design method. See comments in Section 7.10 on pile testing requirements.

The following items should be considered when assessing the validity of a calculation model based on either method:
– soil type and its characteristics
– pile installation method
– pile dimensions and shape
– method of ground testing.

The systematic and random variability of the ground should or must be recognised in the interpretation of the ground tests and calculated resistances.

7.9.3.2 Design using characteristic soil parameters

Characteristic values of soil parameters are used to derive characteristic pile resistances by means of established pile design equations which may be empirical or theoretically based.

When using this method (e.g. using value of $c_{u;k}$ or values of φ'_k) to design piles the UK National Annex to EC7 Part 1[44] requires an added level of factoring to be included in the form of 'model factors'. These are given in Table 7.15. The characteristic resistance is equal to the calculated resistance factored down by the model factor. These factors are used to provide a pile design that will result in pile performance that is in keeping with existing UK experience in terms of reliability and also settlement at working load.

Pile design is based on the following basic equations:

$$R_{s:k} = \int q_{s;k} P_s \, dL / \gamma_{R;d}$$
$$R_{b:k} = A_b \, q_{b;k} / \gamma_{R;d}$$

where:
A_b is the base area of the pile
L is the shaft length of the pile
P_s is the perimeter of the pile
$q_{s;k}$ is the unit shaft resistance calculated from characteristic values of soils data
$q_{b;k}$ is the unit base resistance calculated from characteristic values of soils data
$\gamma_{R;d}$ is the model factor

The design compressive resistance $R_{c;d}$ of the pile will then be:

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

For tension piles there is no base resistance. The partial factor on shaft resistance in tension is $\gamma_{s;t}$ and the design resistance of the pile is $R_{t;d}$. In similar manner to the above:

$$R_{t:k} = \int q_{t;k} P_s \, dL/\gamma_{R;d}$$
$$R_{t;d} = R_{t:k}/\gamma_{s;t}$$

Values of γ_s, γ_b and $\gamma_{s;t}$ can be found in Tables 7.7, 7.9, 7.11 and 7.13 for the various ultimate limit states. It should be noted that use of total capacity (combined shaft and base capacity) is not used in this instance.

A further check on wedge pull-out for single or groups of piles must be carried out to check limiting value of $R_{t;d}$.

In compression, hollow or H-section piles the shaft capacity may include internal friction until such time as the pile ‘plugs’.

Pile design is usually based on empirical equations and factors which link a measureable geotechnical parameter (e.g. c_u) to an observed pile resistance. When using empirical equations to derive pile resistances there is a need to consider the process by which the characteristic value of pile resistance is

Table 7.15 Model factors for pile design by characteristic soil parameters

Situation	Model factor $\gamma_{R;d}$
General requirement	1.4
Where a representative maintained load test taken to the calculated unfactored resistance is performed on the site	1.2
Note Factors from NA to EC7 Part 1[44], Section A.3.3.2.	

obtained. Such a characteristic resistance is a function of the geotechnical parameters of the ground and the empirical factor to modify the soil strength to the pile resistance (e.g. a shaft adhesion factor such as α where $q_s = \alpha c_u$). In order that pile design does not fall out of step with other designs carried out to EC7 it is necessary that the value of the soil parameter brought into this calculation process is the characteristic value unless the design is deemed to be prescriptive, in which case the choice of ground parameters would follow the prescriptive method. To maintain a link to pre-EC7 design experience it is recommended that the value of any empirical factor (e.g. the adhesion factor α used to correlate undrained shear strength of clay soils to shaft friction) should not be changed from pre-EC7 practice. As additional experience is gained in designing piles to EC7 it is quite possible that further guidance will become available allowing designs to be optimised.

Further discussion of UK design practice by this method is presented in Section 7.15.

7.9.3.3 Design using persistent profiles of geotechnical data

The second method of calculating pile resistance is based on profiles of geotechnical data such as would be provided by CPT, or possibly by Ménard pressuremeter tests or similar (see Annex D of EC7 Part 2[2]). Unlike the method above in which the designer must choose the characteristic value for soil parameters, in this method the designer calculates the capacity of the pile directly from the test results (e.g. the profile of cone resistance with depth from CPT).

For the calculation of compressive resistance, the calculated pile (most probable) resistance $R_{c;cal}$ for each profile is used to calculate the characteristic compressive resistance of a pile $R_{c;k}$ using correlation factors ξ as follows.

$$R_{c;k} = \min\left\{\frac{(R_{c;cal})_{mean}}{\xi_3};\frac{(R_{c;cal})_{min}}{\xi_4}\right\}$$

For the calculation of tensile resistance, the same equation is used albeit with a change in nomenclature as follows: $R_{c;k}$ is replaced by $R_{t;k}$ and $R_{c;cal}$ is replaced by $R_{s;cal}$.

Values of ξ_3 and ξ_4 are provided in Table 7.16 and are a function of the number of profiles of geotechnical data available to calculate $R_{c;cal}$ values with.

By making the value of the factor on mean resistance ξ_3 greater than the factor on minimum resistance ξ_4 it is intended that both the typical case and the variability of the results are considered. When $R_{c;k}$ based on the minimum value $R_{c;cal}/\xi_4$ is significantly less than the value of $R_{c;k}$ based on the mean value $R_{c;cal}/\xi_3$ then the designer should appraise the reason why the profiles of ground data vary significantly and consider if the grouping of ground profiles into a single pile design is correct. Careful consideration of the data

Table 7.16 Correlation factors for pile design with geotechnical profiles

ξ for $n =$	1	2	3	4	5	7	10
ξ_3 (on mean value)	1.55	1.47	1.42	1.38	1.36	1.33	1.30
ξ_4 (on minimum value)	1.55	1.39	1.33	1.29	1.26	1.20	1.15

Notes

a For structures with adequate stiffness and strength to transfer load from one pile to another (from weak or soft piles to strong or stiff piles) then the values of ξ above may be reduced by a factor of 1.1.

b Factors from NA to EC7 Part 1[44], Table A.NA.10.

(geotechnical, geological and historical) may suggest that the profiles of ground data be grouped together into smaller batches or that there is inherent variability in the ground which has not been incorporated in the ground model, requiring a reassessment of the ground conditions.

For piles in compression, the characteristic resistance $R_{c;k}$ is separated into the characteristic values of base resistance $R_{b;k}$ and shaft resistance $R_{s;k}$ such that:

$$R_{c;k} = R_{b;k} + R_{s;k}$$

For tension piles only the shaft is considered, hence the design resistance can then be calculated as:

$$R_{c;d} = R_{b;k}/\gamma_b + R_{s;k}/\gamma_s \quad \text{(compression)}$$
$$R_{t;d} = R_{t;k}/\gamma_{s;t} \quad \text{(tension)}$$

7.10 Design using static load test results

Whilst not typical in the UK, EC7 allows design of piles using the results of static trial pile load testing as the means of obtaining pile resistance directly and is acceptable if the following conditions are met:

– Trial pile installation and founding stratum is the same as for contract piles.
– If the trial pile diameter differs from the contract pile diameter then the resulting resistance of shaft and base shall be independently corrected. The trial pile diameter should be no less than half that of the contract pile diameter. A correction (reduction) in measured pile skin friction if the trial pile diameter is less than the contract pile diameter may be appropriate for very coarse grained dense soils and for rock (consult a geotechnical engineer). The trial pile should also be instrumented to allow separate derivation of the shaft and base resistance. Caution should be exercised with respect to open ended driven piles due to the effect of scale on base plugging.
– For piles installed in soils which may result in downdrag (negative skin friction), then the measured or upper bound shaft resistance in these soils

shall be subtracted from the measured capacity to arrive at the measured resistance (see comment on design for negative skin friction in Section 7.12.1).
– Allowance shall be made for variability in ground conditions (systematic and random components) and pile installation when arriving at the characteristic resistance from one or a series of pile load tests.

All of the above requirements lead the designer to the position of having to have a design methodology against which pile behaviour can be assessed.

The assessment of compression pile characteristic resistance is as follows:

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

For the calculation of tensile resistance, the same equation is used albeit with a change in nomenclature as follows: $R_{c;k}$ is replaced by $R_{t;k}$ and $R_{c;cal}$ is replaced by $R_{s;cal}$.

Values of ξ_1 and ξ_2 taken from the UK NA[44] are provided in Table 7.17 and are a function of the number of pile test results available.

For piles in compression, the characteristic resistance $R_{c;k}$ can be separated into the characteristic values of base resistance $R_{b;k}$ and shaft resistance $R_{s;k}$ such that:

$$R_{c;k} = R_{b;k} + R_{s;k}$$

The design resistance can then be calculated as:

$$R_{c;d} = R_{b;k}/\gamma_b + R_{s;k}/\gamma_s \quad \text{(compression)}$$
$$R_{t;d} = R_{t;k}/\gamma_{s;t} \quad \text{(tension)}$$

Alternatively, for piles in compression where there is no means of splitting shaft from base resistance as would be possible in an instrumented pile or as may be possible by means of calculation (relative stiffness of shaft and

Table 7.17 Correlation factors for pile design with static load testing

ξ for $n=$	1	2	3	4	≥ 5
ξ_1 (on mean value)	1.55	1.47	1.42	1.38	1.36
ξ_2 (on minimum value)	1.55	1.35	1.23	1.15	1.08

Notes
a For structures with adequate stiffness and strength to transfer load from one pile to another (from weak or soft piles to strong or stiff piles) then the values of ξ above may be reduced by a factor of 1.1 such that ξ_2 is not less than 1.0.
b Factors from NA to EC7 Part 1[44], Table A.NA.9.

base[87]), the design resistance can be calculated from characteristic resistance as:

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

7.11 Design using dynamic load tests

Design of piles (acting in compression) using the results of dynamic pile load testing is allowed by EC7 when there is a robust correlation between static load tests and dynamic load tests appropriate to the site location, geology, pile type, dimensions and installation technique. The method for assessment of characteristic pile resistance is similar to that for static pile load tests in Section 7.10 albeit with larger correlation factors due to the increased uncertainty. It is recommended that specialist geotechnical engineering input is included if such an approach is taken to pile design.

For further reading about the technique of dynamic load testing and the risks involved, the reader is directed towards specialist piling text books such as *Piling Engineering*[87] or *Pile design and construction practice*[88].

If such an approach is taken to pile design then it is recommended that wave equation analysis is used to analyse the results of pile driving in order to assess the pile resistance; it is recommended that pile driving formula such as Hiley are not used. In all cases the ground conditions shall be investigated by means of ground investigations that are appropriate to the structure being designed, see Chapter 3.

7.12 Further considerations for pile design

7.12.1 Negative skin friction (downdrag)

Negative skin friction (Clause 7.3.2.2 in EC7 Part 1[1]) occurs when the soil in which a pile is constructed settles during the design life of the pile due to influences that are not caused by loading of the pile. Typical situations where negative skin friction may occur include:
- soft soil which is naturally settling under self-weight (usually recent deposits)
- soft soil which is surcharged causing settlement to occur
- soft soil which is dewatered or suffers a reduction in pore-water pressure causing settlement.

In all situations the negative skin friction is brought about as a result of the soil, usually at shallow depth, settling relative to the pile which is founded into

a deeper stable stratum. The magnitude of negative skin friction is limited to the lesser of the following:
– the fully mobilised skin friction along the length of pile in settling ground and above
– the overburden force which is generating the negative skin friction.

In the first limiting condition, negative skin friction is calculated using mobilised skin friction. The process for calculating the characteristic shaft resistance given in Section 7.18 below can be used along with a few notable exceptions and key points:
– Negative skin friction is calculated for the layer that is settling and all the layers above it (e.g. a fill layer causing a soft clay layer to settle).
– Care must be exercised when assessing the characteristic value of soil parameters as, in this instance, a low value of c_u or φ' would tend to underestimate the potential negative skin friction. As it is a requirement that negative skin friction be the maximum which could reasonably be generated, it is suggested that values of $c_{u;k}$ and φ'_k etc. are based on upper characteristic values. This is shown illustratively in Figure 7.1 for a soft alluvial clay site with a surface crust. For negative skin friction design the upper characteristic line should be used for assessment of characteristic negative skin friction.

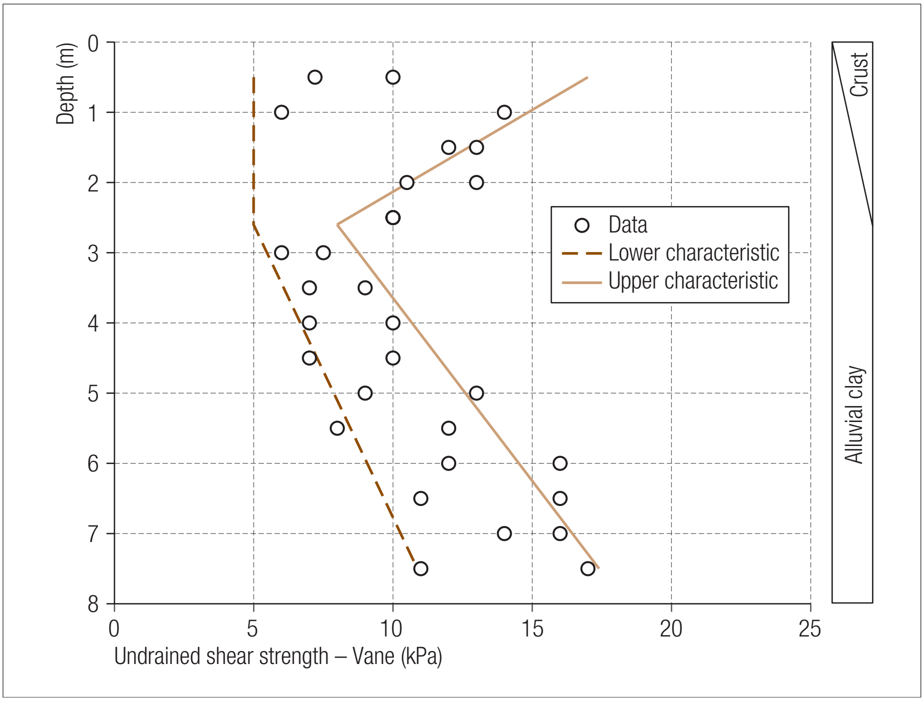


Fig 7.1 Lower and upper characteristic lines

In the second condition the limiting value of negative skin friction is the magnitude of load which generates the negative skin friction; in effect the assumption is that the surcharge load is carried directly by the pile. This assumption is inherently safe when considering the placement of fill on a soft layer through which piles have or will be installed. The characteristic value of the negative skin friction will be a function of the thickness of the fill layer (upper characteristic thickness), the density of the fill layer (upper characteristic density) and the area around each pile which causes settlement of the soft layer (piles central in a group will attract less load than piles at the edges or corners of the group). When taking this approach the calculated negative skin friction can be viewed as a permanent action.

In equation $F_d \leq R_d$ presented in Section 7.9.1 it is usual for the F_d term to be a function of externally applied loads to the pile and R_d term to be derived from the pile's interaction with the ground. However when negative skin friction is included then these general conditions are no longer valid. Hence the build-up of actions and resistance for design of a pile for negative skin friction will be:

$$F_d \leq R_d$$

where:

F_d is the applied design action at pile head + design negative skin friction

R_d is the design resistance of the pile below the level where negative skin friction is assessed.

The design value of negative skin friction $Q_{nsf;d}$ can be obtained from the calculated characteristic value of negative skin friction $Q_{nsf;k}$ factored up by a partial factor γ as given in Table 7.18. The value of γ accommodates both factors on Actions A and on Material strengths M as appropriate for Design Approach 1 Combinations 1 and 2.

Table 7.18 Suggested partial factors for calculation of design negative skin friction

Factors for assessment of design negative skin friction	Case 1: NSF calculated from mobilised skin fiction		Case 2: NSF calculated from magnitude of overburden force	
	DA1:C1	DA1: C2	DA1: C1	DA1: C2
Partial factor set	A1	M2	A1	A1
γ	1.35	1.4 for clays c_u 1.25 for sand/gravel/silt (factors applied to increase downdrag)	1.35 (treating the surcharge as a permanent load)	1.00 (treating the surcharge as a permanent load)
Comment:	Or take $\gamma = 1.0$ and calculate $Q_{nsf;d}$ directly using an upper bound value of negative skin friction (not the characteristic value)		$Q_{nsf;k}$ is calculated as being the action causing the downdrag which will not be exceeded	

The action generated on the pile by the negative skin friction should be included in the structural design of the pile.

7.12.2 Compression loading

7.12.2.1 General

The weight of a pile and overburden pressure at a pile base may be neglected if they cancel out approximately. This may not be the case if downdrag is significant, if the soil is very light or if the pile extends above the ground surface.

7.12.2.2 Pile groups

When designing pile groups, the design resistance is to be considered in terms of both individual compressive resistance (piles alone) and group compressive resistance (piles and soil together as a block). The design resistance is the lower of the two mechanisms.

The stiffness and strength of the structure connecting piles in a group is to be considered when deriving the design resistance. A stiff structure may be able to redistribute load between piles whereas in a flexible structure the resistance of the weakest pile governs. Where the structure can redistribute load then it is permissible to reduce factors ξ in Tables 7.16 and 7.17 by 10% to account for this.

Attention should be given to edge piles where loads from the supported structure are inclined or eccentric as the structure is less able to redistribute loads at the edge of the group.

7.12.2.3 Pile base

The strength of the zone of ground in the vicinity of the pile base (above, below and adjacent) is to be considered when calculating the base resistance.

Where the bearing stratum overlies a weaker stratum, the effect of the weaker stratum is to be considered. Punching failure should be considered if the weaker stratum is less than four base diameters below the pile base.

When the base diameter is larger than the shaft diameter any possible adverse effects are to be considered.

Base plugging in open ended driven piles should be considered in terms of the lesser of

- the base resistance derived using the gross cross-sectional area; or
- the shaft resistance between the plug and inside face of the pile, accounting for the plug length.

Settlement in all strata below the pile base should be considered in terms of individual and group settlement.

7.12.3 Tension loading

The geotechnical design of tension piles must consider two failure mechanisms:
– Pullout of tension pile(s) from a mass of ground (shaft friction failure).
– Pile and ground mass around pile failing in uplift together (block failure).

The geotechnical design of tension pile block/wedge failure is generally verified under the UPL and GEO ultimate limit states. Detailed consideration is, however, beyond the scope of this *Manual* and recourse to specialist geotechnical advice is advised.

In determination of design actions and design resistances of tension piles it is necessary to consider the correct set of limit states as per Table 7.19.

The magnitude of the characteristic action for uplift and due to eccentric loading will be based on the geometric conditions being considered (design water level and design eccentricity).

The magnitude of the characteristic action for heave pressure generated below a piled raft is not so straight forward due to the play off between flexure of the structure which relieves heave pressure and the stiffness of the soil which generates the pressure. In the absence of detailed assessment of heave pressures the upper bound limiting value can be taken to be the full pressure of the effect causing heave (e.g. in clay soils the thickness of excavated soil times the density of the soil). For more economic assessment the heave pressure can be assessed by considering the stiffness of the structure as it reacts to heave and the stiffness of the soil as it relaxes during heave movement.

Within basement construction combined soil and uplift water pressures can occur at the same time requiring assessment of the combined effects on tension pile design.

Table 7.19 Limit states for tension pile design

Limit states	Tension generated by water pressure beneath a slab (uplift)	Tension from eccentric loading	Tension from heave pressures[a]
EQU	Yes	Yes	No
STR/GEO	Yes	Yes	Yes
UPL	Yes	Yes	Yes

Note

a Heave can be generated by uplift below excavations, frost action and changes in groundwater level (from, for example, the release of suction due to the removal of trees, cessation of abstraction from aquifers, etc.) Heave forces are generally treated as actions and in a similar manner to that of downdrag in Section 7.12.1.

Heave may also occur during construction before structural loads are applied. This can lead to uplift or structural failure of the piles if they are not designed for this transient design situation.

7.12.4 Transverse loading

Transverse loading (i.e. lateral loading) (Clause 7.3.2.4 of EC7 Part 1[1]) of a pile can be a result of load applied to the pile head above ground level or the load applied to the pile shaft by movement of the ground relative to the pile.

Design situations can include:
- Piles designed to carry a horizontal load due to inclined columns (whose load is not taken out by the structure).
- Piles designed to carry a horizontal load from uneven boundary conditions (e.g. one side of building supports an earth embankment or landscaping whilst the other does not).
- Raking piles installed to resist horizontal load but which also attract transverse loading due to settling ground (more common in bridge abutments).
- Vertical piles installed in ground which is moving laterally due to uneven surcharge loading (more common in bridge abutments).
- Piles adjacent to a cutting or in a slope.

The design methods, design considerations and load testing requirements for axially loaded piles also apply to transversely loaded piles.

Design of piles for transverse loading for ultimate limit states is to satisfy the following condition:

$$F_{\mathrm{tr;d}} \leq R_{\mathrm{tr;d}}$$

where:
$F_{\mathrm{tr;d}}$ is the transverse design loads on a pile or pile foundation
$R_{\mathrm{tr;d}}$ is the design resistance of the pile or pile foundation to transverse loading.

Design should consider load cases STR and GEO in the first instance as these are likely to be the controlling design cases. Methods for assessment of transverse loading and transverse resistance of piles are provided in publications such as:
- *Piling engineering*[87]
- *Pile design and construction practice*[88].

Alternative use of proprietary software is useful.

Aspects to be aware of include:
- type of failure mechanism, i.e. short vs. long piles
- group effects

– variability of the ground at the top of a pile
– pile head fixity.

In design, consideration of soil strength and stiffness is required in addition to consideration of pile stiffness. This will allow pile movement assessment and calculation of structural actions for use in structural design of the pile. Where ground stiffness is more important than strength a cautious estimate of ground stiffness should be used in ULS calculations.

7.13 Serviceability limit states

7.13.1 Single pile movement

Vertical displacements are to be assessed and checked against the serviceability criteria discussed in Chapter 2. The occurrence of a serviceability limit state in a supported structure is also to be checked. It should be recognised and taken into account that calculations for vertical pile displacement are estimates.

The settlement of a single pile designed with ULS Design Approach 1 Combination 2 partial factors will usually be relatively small, typically 0.5% to 1% of the pile diameter plus any additional settlement resulting from pile shaft shortening above or within the bearing stratum level (especially important for small diameter or slender piles). Such movements are not likely to be critical for structural design. Pile group settlement on the other hand (Section 7.13.2) should be checked.

It is considered advisable to include pile load testing to demonstrate that pile performance on a particular site is as anticipated and can be accommodated by the building. However, if load tests are not available then pile stiffness should be based on safe empirical assumptions, for example, those given in Table 7.20.

The applied actions to piles for assessment of displacement should include downdrag forces for compression piles as well as heave forces for tension piles.

7.13.2 Pile group movement

In addition to the requirements for single piles, the vertical displacement of pile groups requires assessment with a distinction between individual and group action.

Pile group movements for typical buildings as included in this *Manual* are considered to be relatively small for piles bearing in competent ground (stiff

Table 7.20 Single pile settlement/heave in soils

	Settlement when design (or representative) load is applied	Pile settlement at geotechnical failure	Comment
Shaft alone	–	0.5–1% pile diameter	Load – displacement response stiff all the way to failure
Base alone	–	10–15% pile diameter	Load – displacement response is usually ductile. Definition of failure is in general terms 'large displacement'
Compression design (SLS) action carried on shaft alone	0.5% pile diameter plus pile compression	–	If design is based on a stiff pile then consider a check to demonstrate that all representative load can be carried by the characteristic shaft friction divided by 1.2
Compression design (SLS) action carried on combined shaft and base	1% pile diameter plus pile compression	–	Possibly larger if base resistance is significant at representative load as used in base grouted piles or under-ream piles
Tension	0.5% pile diameter plus structural strains	–	It is necessary to consider pile structure elongation from pile head to the location at which the mobilised tensile force in the pile drops to zero. For reinforced concrete piles the structural elongation can be considerable compared to geotechnical movements

clay, medium dense to dense sand etc.) In order to check differential movement between columns and total settlement, the use of proprietary computer software is recommended wherein pile and ground properties can be modelled to assess pile group settlement. Ground stiffness parameters must be characteristic values for use with SLS design loads when calculating movements.

Movement of piles constructed within basements need to consider the effect of pile settlement due to the application of building load and heave due to the effects of the excavation unloading; this is particularly important for excavations in clay soils where heave movements may take years to occur.

7.13.3 Further considerations for transversely loaded piles

Assessment of transverse displacement needs to take account of ground stiffness, pile stiffness, moment fixity, group effects and load reversals.

7.14 Interaction of piles with other structures

When designing piles or structures adjacent to existing piles, consideration needs to be made of the interaction between the piles and structures which surround them. Such interactions are increasingly common in urban development where new basement construction is adjacent to piled structures without basements. In these situations the design, specification and construction of retaining walls (usually embedded retaining walls) needs to consider how the wall installation and subsequent basement excavation will affect adjacent piles. Possible effects on adjacent piles include downdrag actions, loss of pile bearing resistance and pile movement (both bending and settlement) in addition to the general considerations of ground and building damage. Modifications to retaining wall location may be appropriate to reduce the effects of retaining wall installation on adjacent piles as may the adoption of a basement excavation sequence which reduces ground movements to an acceptable level.

Corresponding consideration of how pile installation affects retaining walls is also relevant in the case where pile installation is carried out from within an excavation or behind a wall retaining water (e.g. a dock wall); this is especially important in soft clays/silts or loose sands/gravels neither of which are self-supporting. In situations where the piles are located in the passive ground supporting the retaining wall, pile installation may result in retaining wall movement which in turn may result in unacceptable ground movements beyond the excavation or loss of water tightness of the retaining wall. Changes to construction sequence may be necessary (e.g. construction of piles from a higher level prior to basement excavation) or changes to piling technique may similarly be appropriate to ensure that the ground is supported during the pile excavation process (e.g. use of appropriately long temporary casings). Consideration of how the retaining wall and walings will accommodate the effects of pile installation and uneven distributions of passive resistance may also help in justification of piling close to the toe of the retaining wall.

Many urban areas have infrastructure tunnels below ground level. These tunnels include utility tunnels (telecoms, electricity, water and drainage), transportation tunnels (metro and mainline train as well as shafts and inclined tunnels for access) as well as 'secret tunnels' which include those owned by defence agencies and sometimes banks. Deep foundation design and construction must consider the locations of these tunnels and any constraints which these tunnels place on development. Once the presence of tunnels is identified consultation with the tunnel owner is necessary to understand the constraints which the tunnels place on development and the steps that must be taken to allow construction to commence. Early meetings are often necessary due to the possible need to monitor the tunnels in advance (in some circumstances for a year) of the commencement of construction. Depending on the location of the tunnel, the sensitivity of the tunnel and the changes that the development will place on the tunnel, significant modifications to the substructure may be necessary (piles spanning over tunnel exclusion zones, sleeved piles to prevent the tunnel being loaded etc.)

In nearly all cases of soil structure interaction, ground and structural movements will occur. These need to be quantified, the effects assessed and accepted prior to commencement of construction.

Where complex interactions between ground structures are foreseen then consultation with a geotechnical specialist may be appropriate.

7.15 Interaction with structural design

Structural design of pile caps is covered in Section 5.10.9 of the Institution's *Manual for the design of concrete building structures to Eurocode 2*[85].

Over and beyond the general statements of design interaction between structural and geotechnical engineering made in Chapters 1 and 4 the following comments are specific to the design of piled foundations:
- Checks must be undertaken to ensure that movement of piled foundations (geotechnical Eurocode) does not lead to an ultimate limit state in the supported structure (structural Eurocode).
- Piled foundations and a supported structure must be designed to accommodate or resist negative skin friction in the ground that has the potential to settle relative to the pile as well as uplift pressures that have a tendency to cause heave in the pile (likely only critical for tension piles).
- Foundation solutions with mixed foundation types (piles and spread foundations or foundation solutions with piles of varying sizes/lengths) must be assessed to identify resulting load distributions and settlement and how these will impact on foundation and structural design.
- Ground-structure interaction may be generally undertaken using spring or continuum models. However, when ground-structure interaction is important more sophisticated models may be necessary (numerical analysis). Table 7.21 illustrates the general hierarchy of ground-structure interaction models and when they may be used appropriately.

When considering pile settlement the contribution of, and impact on, the structure it is important to note that:
- Differential settlement calculations have a tendency to be overly conservative when soil-structure interaction is ignored.

Table 7.21 Options for design complexity – piles

Ground-structure interaction model	Appropriate use
Rule of thumb design/design charts for piled foundations[85]	Pile supporting pile caps with regular geometry and with piles at typical pile to pile centres (e.g. at 3 diameters centre to centre spacing)
Rigid pile cap allowing load distribution between columns and piles	Design of raft foundation reinforcement (shear and bending moment) and pile load distribution for situations with uniform pile type (diameter, length) in horizontally uniform ground
Iterative solutions using proprietary software incorporating ground (modelled with elastic constants) and structure accounting for structural form (note that where the models are fully elastic edge/corner effects are likely to be over predicted)	Provision of pile loads and pile cap actions (shear and bending moment) allowing for non-uniform loading and variable structure/ground stiffness
Numerical analysis of ground and structure in 2D and 3D	Category 3 structures, with high degree of structural complexity and risk

– Differential ground and pile stiffnesses will act to increase bending and shear in the structure (pile cap), alter mobilised pile reactions and increase deformation in the structure.

The structural design of a piled foundation is to accommodate the actions which it will experience during its design life. It should take into account construction tolerances and the required pile performance. The design should be carried out to the requirements of the appropriate Eurocode (EC2 for concrete, EC3 for steel in the main part). For preformed piles this will also include handling and installation actions. For precast concrete piles which include joints, these joints should be suitable for the actions being transferred across them (axial loads and bending moment in the main part). A check for buckling may be necessary for piles that are slender and in weak ground (for example, soil with $c_{u,k} \leq 10\text{kN/m}^2$).

7.16 Underpinning

Underpinning covers the remedial works and modification carried out to foundations (usually spread pad or strip footings) for a variety of reasons:
– To increase the capacity of foundations due to under design or increased requirements for capacity (e.g. due to super-structure modifications).
– To mitigate against existing and on-going differential movements as may be caused by desiccation due to tree roots, local variations in foundation stiffness or collapse of shallow underground workings.
– To protect structures from proposed adjacent construction works which may cause unacceptable damage to the structure.

– To effect an increase in foundation depth to allow for basement construction beneath or adjacent to existing structures.

Underpinning may be achieved by the addition of piles (usually small diameter) either through or adjacent to the foundation requiring underpinning along with structural connections, or by means of increasing the depth of the spread footing by means of local excavation and deepening of the spread footing.

In all situations listed above underpinning is applied to an existing structure and results in modifications to the load path from structure through the foundation to the ground. This modification to the load path will usually result in settlement that must be accommodated by the structure above the underpinning level. If the structure cannot accommodate this movement then underpinning (or the proposed underpinning method) is not viable. Underpinning should therefore only be undertaken, be it for remediation or development purposes, following investigation of the structure, of the existing foundations and of the ground conditions on which they are bearing. When underpinning is being proposed for remedial purposes careful investigation of the cause of movement must be carried out to identify with reasonable certainty the cause(s) of settlement and the effect that any proposed underpinning solution will have on these movements. Particular attention to the underpinning solution is required where only part of a structure is to be underpinned (partial underpinning) to ensure that the underpinning works do not cause differential movement in the future as would be caused by the underpinned section being less susceptible to seasonal movements than the shallow founded section. Clearly in this consideration the extent of the structure is not limited by property ownership constraints (e.g. a terrace of houses of a semi-detached house).

Whilst included in this *Manual*, there is no explicit mention of underpinning in EC7. The lack of mention in EC7 is, in itself, not surprising given that the process of underpinning incorporates elements of structural and geotechnical design which are covered by wall and foundation sections of the code.

For details on underpinning techniques and design the reader is referred to such documents as *Geotechnics for Building Professionals*[89], BRE Digest 352 *Underpinning*[90] and for the situation of protection of buildings against adjacent development works the Institution's publication *Design and construction of deep basements including cut-and-cover structures*[91].

7.17 Execution standards and construction tolerances

The Eurocodes are complemented by execution standards. With respect to pile installation and specification the following execution documents apply:
– BS EN 1536:2010 *Execution of special geotechnical works – bored piles*[8]
– BS EN 12063:1999 *Execution of special geotechnical works – sheet pile*

walls[11]
– BS EN 12699:2001 *Execution of special geotechnical works – displacement piles*[12]
– BS EN 14199:2005 *Execution of special geotechnical works – micro piles*[15]

Reference may also be made to:
– BS EN 12794:2005 *Precast concrete products – foundation piles*[92]
– BS EN 1538:2010 *Execution of special geotechnical works – diaphragm walls*[10]

For construction in the UK use of the specification *ICE Specification for piling and embedded retaining walls*[38] is also recommended as the base specification. Similar documents also exist in the Highways Agency Manual of Contract Documents for Highway Works, Specification for Highway Works, Series 1600 *Piling and Embedded Retaining Walls*[39].

The information in the execution standards should form part of the design process to allow for acceptable construction tolerances and resulting pile design actions (variation in axial, shear and bending actions). Table 7.22 presents the construction tolerances that should be allowed in design and achieved in construction (specification limits).

These tolerances need to be considered in the structural design of piles in terms of bending moment at the top of the pile (based on pile eccentricity) and shear at the top of the pile (based on pile inclination). It is clear that the

Table 7.22 Construction tolerances for piled foundations

Consideration	Range or type	Bored piles (BS EN 1536:2010)	Displacement piles (BS EN 12699:2001)
Plan eccentricity e (resultant)	Diameter <1.0m	$e \leq 0.1$m	–
	Diameter 1.0 to 1.5m	$e \leq 0.1D$ where D is pile diameter	–
	Diameter >1.5m	$e \leq 0.15$m	–
	All sizes	–	$e \leq 0.1$m
	UK practice[a]	$e \leq 0.075$m	$e \leq 0.075$m
Inclination tolerance i	Vertical piles	$i \leq 0.02$ radians (1 in 50)	$i \leq 0.04$ radians (1 in 25)
	Raked piles	$i \leq 0.04$ radians (1 in 25) Maximum rake 1 in 4	$i \leq 0.04$ radians (1 in 25) Maximum rake 1 in 4
	UK practice – vertical piles[a]	$i \leq 0.013$ radians (1 in 75)	$i \leq 0.013$ radians (1 in 75)

Note
a These are from the ICE Specification for Piling and Embedded Retaining Walls[38]. This document is often used as a means of specifying piling works along with a particular specification.

need to consider these eccentricities and inclinations will be a function of the robustness of the pile or pile group and as a function of the loading being applied. For a single slender pile supporting a column the impact of eccentricity of 0.1m may have a significant effect on the pile's structural design; the same is probably less true for a 1.5m diameter pile. Similarly, where piles are in groups of three or more (assuming not in a line), the geometry of the pile group tends to prevent a moment being mobilised due to eccentricity but does not reduce the effect of shear being mobilised due to lack of inclination control. Further comments on structural design of pile caps are presented in Sections 5.10.9 and 5.10.10 of the Institution's *Manual* on EC2[85].

7.18 UK practice for design using ground test results

Conventional UK design adopts the following approach for assessment of pile resistance with modifications in nomenclature to account for EC7 modifications:

$R = R_s + R_b$

$R_{s;k} = \pi D L\, q_{s;k} = (\pi D L)\, \alpha\, c_{u;k}$	clay
$R_{s;k} = \pi D L\, q_{s;k} = (\pi D L)\, K_s\, \sigma'_v \tan(\varphi'_{cv;k})$	sand/gravel
$R_{s;k} = \pi D L\, q_{s;k} = (\pi D L)\, a(\sigma_{ci;k})^b$	rock
$R_{b;k} = \pi D^2/4 \times 9\, c_{u;k\_base}$	clay
$R_{b;k} = \pi D^2/4 \times N_q\, \sigma'_{v\_base}$	sand/gravel
$R_{b;k} = \pi D^2/4 \times \text{fn}(\sigma_{ci;k})$	rock

where:
α is the factor linking undrained shear strength to shaft friction
K_s is the factor linking horizontal to vertical effective stresses
N_q is the bearing capacity factor based on characteristic value of φ'
$\sigma_{ci;k}$ is the characteristic unconfined compression strength of rock
a and *b* represent constants

General pile design equations in the following ground conditions are given in Tables 7.23 to 7.26:
– Pile design in clay soils.
– Pile design in sands and gravels.
– Pile design in chalk.
– Pile design in rock.

Tables 7.23, 7.24, 7.25 and 7.26 are provided as an outline guide to pile design; the reader is advised to read the references cited and other appropriate references to understand the limitations of a specific design method being used. Where ground strata are highly variable over the length of a pile (e.g. inter-bedded clay and sand layers) or where ground conditions

Table 7.23 Pile design equations: clay soils

Pile design in clay		
$q_{s;k}$	$\alpha\ c_{u:k}$	α is shaft adhesion factor $\alpha = 0.5$ for firm and stiff clays, limit of $q_{s;k\_ave} = 110 kN/m^2$ (see Figure 7.2) $\alpha = 1.0$ to 0.5 for very soft to soft clays depending on strength (see Figure 7.2)
References: – *Piling engineering*[87] – *Shaft capacity of driven piles in clay*[93]		
$q_{b;k}$	$N_c\ c_{u:k}$	N_c is the bearing capacity factor = 9

deteriorate below the base level of a pile (risk of block failure) then specialist advice should be obtained.

Figure 7.2 (Fleming[87]) shows how the shaft adhesion factor varies with the ratio of undrained shear strength to vertical effective stress. The undrained shear strength to vertical effective stress ratio (c_u/σ'_v) is a measure of the soil over-consolidation. For values of c_u/σ'_v less than 0.2 to 0.3 (depending on soil plasticity) the clay can be considered to be unconsolidated which implies that it is settling and as such no beneficial effect can be obtained from it, while for values of c_u/σ'_v in excess of 0.8 the ground can be considered to be

Table 7.24 Pile design equations: sand and gravel soils

Pile design in sands and gravels (silica soils only)		
$q_{s;k}$	$\sigma'_v\ K_s \tan \delta_k$	K_s is the earth factor to convert vertical to horizontal effective stresses. This is usually taken as 0.7 for bored piles and between 0.5 and 0.9 for CFA piles (lower value for silty sands). For driven piles a value of 1.0 may be assumed. δ_k is the characteristic angle of friction on the shaft of the pile. This is usually taken to be between $\varphi_{in\text{-}situ;k}$ and $\varphi_{cv;k}$ for conventionally bored and CFA piles and to be $\varphi_{cv;k}$ for preformed driven piles. σ'_v is the vertical effective stress.
Reference: – *Piling Engineering*[87]		
$q_{b;k}$	$N_q\ \sigma'_v$	N_q is the bearing capacity based on $\varphi'_{p;k}$ as per Figure 7.3[87]. Base resistance is usually limited to a maximum of $11 MN/m^2$ in very dense sands, less in looser deposits. It should be noted that that the bearing capacity factor is also seen to reduce with depth (others have looked at reducing bearing capacity factor with increasing effective stresses).
References: – *Load bearing capacity and deformation of piled foundations*[94] – *Piling engineering*[87] – *Pile design and construction practice*[88]		

Table 7.25 Pile design equations: chalk

Pile design in chalk		
$q_{s;k}$	$a + b\sigma'_v$	a and b are constants (values below) σ'_v is the vertical effective stress (the increment below top of chalk is approximately equal to $10z$ (kN/m^2) where z is distance below top of chalk) Driven cast-in-place and (non-CFA) bored piles: $a = 0$, $b = 0.8$; $q_{s;k} \leq 300$kN/m^2 CFA piles: $a = 0$, $b = 0.45$; $q_{s;k} \leq 100$kN/m^2 Low displacement driven piles (steel): $a = 20$kN/m^2, $b = 0$: Low and medium density chalk Low displacement driven piles (steel): $a = 120$kN/m^2, $b = 0$: High density chalk Large displacement driven piles: $a = 20$kN/m^2, $b = 0$
$q_{b;k}$	cN_k (kN/m^2)	c is a constant N is the SPT blow count per 300mm penetration. All bored piles: $c = 200$ Driven cast-in-place piles: $c = 250$ Driven piles: $c = 400$
Reference: – *Engineering in Chalk*[73]		

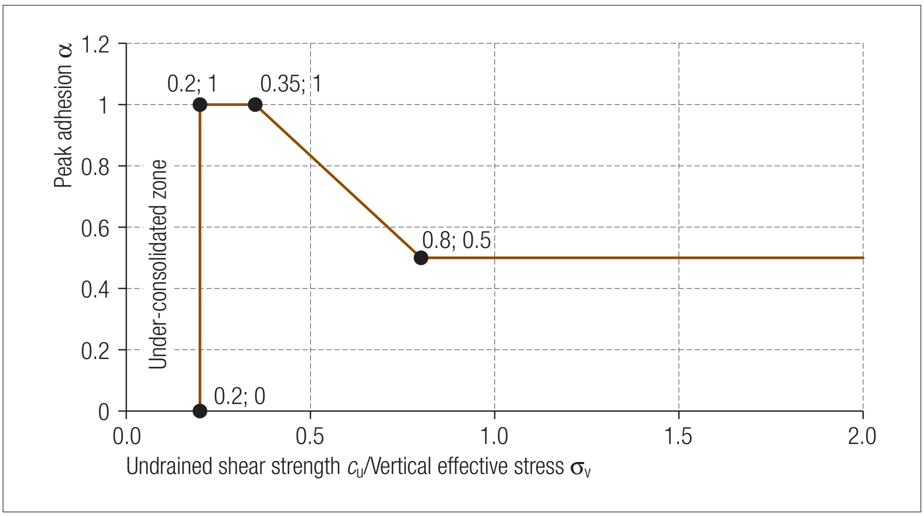


Fig 7.2 Shaft adhesion factor with undrained shear strength c_u

Table 7.26 Pile design equations: rock (generic)

Pile design in 'rock'		
$q_{s;k}$	$a\sigma_{ci;k}^{\ b}$	*a* and *b* are constants (values below). *a* varies between 0.45 and 0.6 for small and medium diameter piles depending on the roughness of the rock socket (all sockets assumed clean rock to concrete interface) with 0.6 being for clean rough sockets. Where large rock sockets are constructed, greater than 0.6m in diameter, then it is necessary to investigate the effect of socket diameter on shaft resistance. The *a* value can reduce to less than 50% of the values for smaller diameter piles. *b* is conventionally taken as 0.5. It should be noted that shaft friction is usually limited to 5% of the concrete characteristic cube strength (6.25% of the cylinder strength) based on unfactored loads (working stress). It should be noted that σ_{ci} is always measured in MN/m^2 in this equation with the resulting shaft friction also in MN/m^2.
$q_{b;k}$	$a\sigma_{ci;k}^{\ b}$	*a* is typically 4.8 albeit the lower bound value of 3.0 is recommended unless there is ample case history data available to fully justify a higher value. *b* is found to be 0.5. It should be noted that σ_{ci} is always measured in MN/m^2 in this equation with the resulting shaft friction also in MN/m^2.
Notes: – Due to the potential brittle nature of rock sockets it is recommended that design be based on either shaft or base alone in order to control settlement and prevent the risk of brittle behaviour. – For combined shaft and base behaviour attention to *A design method for drilled piers in soft rock*[95] is recommended. Propriety software based on *A new rock socket roughness factor for prediction of rock socket shaft resistance*[96] also exists which models full pile performance. – Further information particularly in relation to 'softer' rocks can be found in *Piled foundations in weak rock*[97].		
References: – *A design method for drilled piers in soft rock*[95] – *A new rock socket roughness factor for prediction of rock socket shaft resistance*[96] – *End bearing capacity of drilled shafts in rock*[98].		

moderately to highly over-consolidated. Further comments on how to design for under-consolidated soil are provided in Section 7.11.1 for downdrag effect on piles.

Alternative designs based on *in situ* testing exist. These are most often used in mainland Europe and may become of value in the UK as site investigation techniques change. Such design rules are presented in the appendices to EC7 as follows:
– Annex D of EC7 Part 2[2]: Pile design using the CPT method.
– Annex E of EC7 Part 2[2]: Pile design using the Ménard Pressuremeter method.

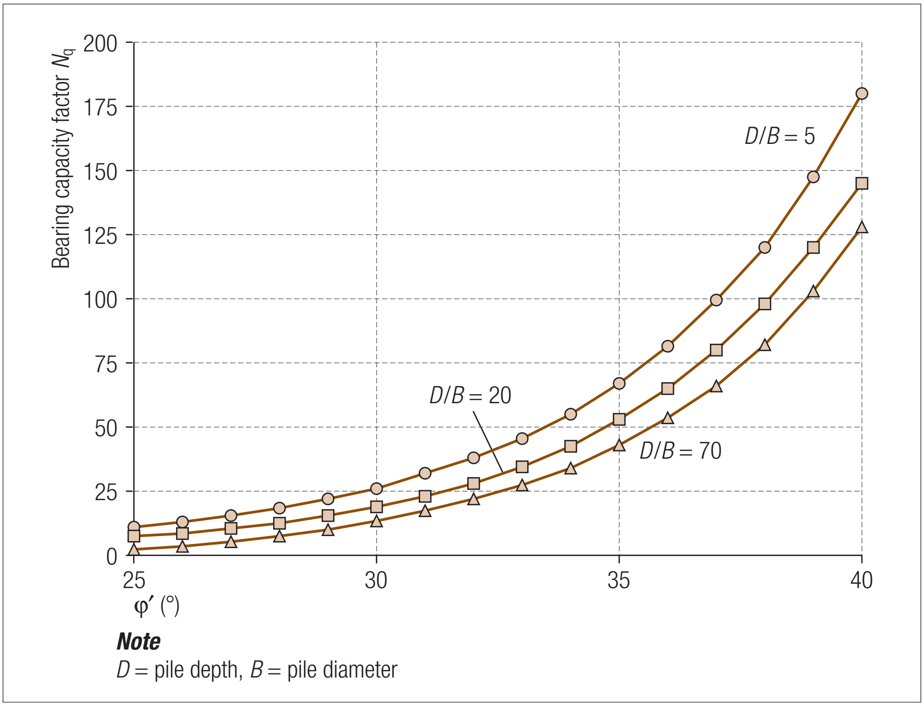


Fig 7.3 Variation of N_q with φ' (Fleming)

7.19 Illustration of design process

Refer to Appendix D.

7.20 Summary

- Prior to adoption of a particular pile type for support of building loads it is necessary to confirm the following:
  - Use of piles are appropriate when compared to spread foundations with or without ground improvement
  - The process of pile installation is compatible with the ground conditions (installation method, impact on stability of pile bores for bored piles, contamination of aquifers below contaminated ground etc.) at the site and to the locality of the site (noise and vibration, site access for piling plant etc.)

– Pile design (ULS) is typically carried out using geotechnical parameters obtained from ground investigations and equations for shaft and base resistance. Pile design using continuous profiles of ground data (e.g. a CPT profile) to calculate a 'location specific' pile capacity is also possible but not common in UK practice. Pile design methods are based on the results of pile load tests (project specific or archive).

– The use of preliminary and construction pile load tests is relatively common place, allows validation of the design assumptions, provides confidence in the construction method, and allows for reduction in design partial and model factors.

– Design of piles using the results of pile load tests directly (justified by experience or calculation) is permitted but again not usually carried out in the UK.

– Design of piles differs from most other geotechnical designs in that partial factors are applied to calculated resistances (shaft or base – these resistance factors are applied after the geotechnical calculation) rather than to the geotechnical parameter (e.g. to c_u or φ' – these partial factors are applied prior to the geotechnical calculation).

– ULS design of piles using geotechnical parameters (the typical approach in the UK) has the sets of factors listed in Table 7.27.

Table 7.27 Partial factors used in ULS pile design

Partial factors used	Comments
Model factors: – Table 7.15	Used to account for the presence, or not, of relevant pile load test data to calculated failure load. The model factor reduces the calculated resistance to the characteristic resistance
Resistance factors (R): – Table 7.7 for EQU – Table 7.9 for STR/GEO (DA1 C1) – Table 7.11 for STR/GEO (DA1 C2) – Table 7.13 for UPL	Used to reduce the characteristic resistance to the appropriate design resistance. Different resistance factors are provided for different pile types (bored, driven, CFA), for shaft and base components, for compression or tension loading, to account for the use of contract pile load tests, to verify performance in the working load range and finally as a function of the importance of reliable settlement at working load
Factor on actions (A): – Table 7.6 for EQU – Table 7.8 for STR/GEO (DA1 C1) – Table 7.10 for STR/GEO (DA1 C2) – Table 7.12 for UPL	Load factors are applied appropriate to the calculation being carried out (EQU, STR or GEO and UPL)
Factor on materials: – Table 7.14	Factors are applied as appropriate to EQU, STR, GEO and UPL
Negative skin friction: – Table 7.18	Factors used in assessing the design value of negative skin friction for STR/GEO load cases

– Pile design must consider both pile settlement due to loading, which includes both actions applied by the structure and actions which are generated by ground movements. In particular negative skin friction (downdrag) should be included in pile design calculations for both ULS and SLS considerations.

8 Retaining structures and basement stability

8.1 Introduction

Geotechnical materials (i.e. soil, rock and water) are considered to be retained by a structure if they are kept at a slope steeper than that which they would normally assume if the structure was absent. Correspondingly, a structure is considered to be a retaining structure when the retained material imposes forces on its structural elements.

A distinction is made between three main types of retaining structure (as illustrated in Figure 8.1) (Section 9.1.2 EC7 Part 1[1]):
– gravity walls (the weight of the wall and any stabilising backfill contributes significantly to supporting the retained material)
– embedded walls (the bending capacity of the wall contributes significantly to supporting the retained material)
– composite retaining structures (a wall combining elements of the previous two types – generally not covered by the scope of this *Manual*).

Section 2.11.2.2 introduced NCCI document PD 6694-1[34] which presents the design of retaining walls subjected to traffic loading. A key comment relating to PD 6694-1 is the need, at an early stage of design development, to

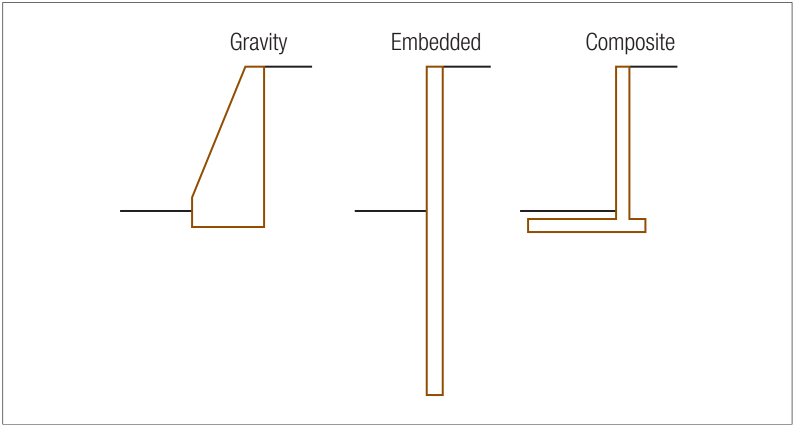


Fig 8.1 Types of retaining walls

agree with the highway overseeing authority the design basis for any wall supporting a highway. The following are of prime importance for a conventional retaining wall design:
– the use of either ‘bridges’ or ‘building’ partial factors for actions
– the magnitude of highway (vehicular) loading.

8.2 Geotechnical category

Retaining structures in GC1 should be small, simple and be employed only in competent ground. Prescriptive design for retaining structures is relatively limited though it may be adopted for accompanying aspects of design such as the assessment of durability.

The design of retaining structures in a qualitative manner could include the use of design charts presented by manufacturers of products such as modular masonry blocks or in other guidance documents[99]. These charts relate to specific conditions so the assumptions would need to be carefully checked. Design charts could also be used for GC1 or to inform preliminary design for GC2 retaining structures. For gravity or composite walls especially (as in Figure 8.1), design charts typically only address the retention of fill above the toe level of the wall; consideration of the stability of the bearing layer supporting the wall needs to be included in the design (see Section 8.16 for geotechnical failure modes).

It is considered that the majority of retaining structures within the scope of this *Manual* are more likely to fall into the GC2 category and therefore require quantitative analysis. However, this does not imply that design must be complex as many retaining structures can be designed using relatively simple calculations.

8.3 Limit states

A list of limit states to be considered should be created. Table 8.1 presents a basic list of limit states that must be considered for any retaining structure. Combinations of limit states are to be considered where relevant.

Table 8.1 Limit states to be considered as a minimum – retaining structures

Gravity walls	Composite structures	Embedded walls
– Bearing resistance failure of the soil below the base (see Chapter 6) – Failure by sliding at the base – Failure by toppling		– Failure by rotation or translation of parts thereof – Failure by lack of vertical equilibrium
– Loss of overall stability – Failure of a structural element or the connection between elements – Combined failure in the ground and structure – Failure by hydraulic heave and piping – Movement leading to collapse, adverse appearance or efficient use (including nearby structures or services) – Unacceptable leakage through or beneath the wall – Unacceptable transport of soil through or beneath the wall – Unacceptable change in the groundwater regime		

8.4 Design situations

Design situations were introduced in Chapter 2. Additional considerations for retaining structures include:
– Variations in ground conditions (soil properties, water levels, pore-water pressures) in space and anticipated variations in time (e.g. clay behaviour moving from ‘undrained’ to ‘drained’ strength with time).
– Variation in actions and how they are combined.
– Removal/loss of material from the front of a retaining structure (e.g. service trench or accidental overdig).
– Effects of compaction on fill material behind a retaining structure.
– Effects of anticipated future structures, surcharges and unloading behind a retaining structure.
– Anticipated ground movements and how these impact on adjacent structures.
– Effects of chemicals and corrosion on structural materials (it should be noted that corrosion of sheet piles is addressed in Section 4.4.3. Chemical (sulphate) attack on concrete is addressed in Section 4.4.2).

8.5 Design considerations

Considerations for design were introduced in Chapter 2. Additional considerations for retaining structures include:
– Demonstrating that vertical equilibrium can be achieved (i.e. a retaining wall must be stable in all directions; this is specifically important for anchored walls and walls carrying vertical loads).

- Designing for ductility so that the approach of ultimate limit states is clearly visible.
- Recognising that acceptable movements for a retaining structure may not be acceptable for a nearby or supported structure.
- Eurocode methods and partial factors are usually sufficient to prevent the occurrence of ultimate limit states in nearby structures (given competent ground and appropriate construction) but may not prevent the occurrence of serviceability limit states.
- Special care should be taken with regard to potential movements in some highly overconsolidated clays.
- The Observational Method may be appropriate where complex ground-structure interaction makes detailed design difficult. This approach requires ductile behaviour which can be controlled by means of modified construction sequences. There must also be no fear of sudden load change (e.g. change in water pressure due to a rain storm).
- Effects of wall construction (e.g. temporary support, stress changes, ground movements, installation disturbance, construction access).
- Water tightness requirements to provide the desired quality of (basement) space; the wall may be part of the water proofing system when combined with other protection measures.
- Water cut-off: feasibility (in terms of depth required) and its effects on existing groundwater conditions.
- Feasibility of excavating between props.
- Access for maintenance.
- Appearance and durability.
- Driveability of sheet piles.
- Borehole and trench stability.
- Materials available for use as backfill and the means used to compact them.
- Where the safety and serviceability of a retaining structure depend on successful performance of the drainage system (as in Section 9.4.2 EC7 Part 1[1]):
  - Consequences of drainage system failure (including safety and cost of repair).
  - Drainage systems must be designed with access for a specified maintenance programme or it must be shown that they will operate adequately without maintenance (if it cannot be shown that a drainage system will remain functional then the design must be completed without reliance on it).
  - Discharge water volume, pressure and chemical content.

8.6 Retaining structure selection

The selection of a particular type of retaining structure should be based upon:
- location
- size
- cost

– space available for construction plant
– ground and groundwater conditions
– durability and maintenance requirements
– movement criteria
– aesthetics
– construction duration
– method of construction.

Some of the advantages and disadvantages of selected types of retaining structure are presented in Table 8.2.

An important aspect of retaining wall selection is sizing. Table 8.3 presents typical dimensions and ratios for preliminary sizing.

Table 8.2 Some advantages and disadvantages of various retaining structures

Type	Advantages	Disadvantages
Gravity		
Mass (concrete, masonry)	– Simple – Easy to shape	– When used for retention of existing ground, and depending on space available, temporary cutting may need support – Large volume of materials used – Need good foundation – Need movement joints
Gabion (rock, basket)	– Simple – Efficient use of man-made materials – Can deal with poor foundation	– When used for retention of existing ground, and depending on space available, temporary cutting may need support – Large volume of natural materials used – Quality rock needed for infill – Corrosion issues
Modular (concrete)	– Simple – Efficient use of materials – Prefabricated	– When used for retention of existing ground, and depending on space available, temporary cutting may need support
Crib (concrete, timber)	– Simple – Efficient use of materials – Prefabricated	– When used for retention of existing ground, and depending on space available, temporary cutting may need support – Self-draining material needed – Need good detailing
Embedded		
Contiguous pile (concrete)	– Efficient use of space – Bearing capacity not important – Efficient when incorporated in final structure such as basement – Cheapest concrete pile wall	– May need significant embedment depth – Higher cost than gravity/composite

Table 8.2 Continued

Type	Advantages	Disadvantages
Embedded (*cont.*)		
Secant pile (concrete, hard-soft)	– Efficient use of space – Efficient when incorporated in final structure such as basement	– May need significant embedment depth – Higher cost than gravity/composite – May affect groundwater flows – Hard-soft may require on site batching – Less durable than hard-firm (temporary water retention only) – Can declutch
Secant pile (concrete, hard-firm, hard-hard)	– Efficient use of space – Efficient when incorporated in final structure such as basement – Hard-firm and hard-hard can retain water permanently	– May need significant embedment depth – Higher cost than gravity/composite – May affect groundwater flows – Hard-hard requires high torque plant – Can declutch
King post and lagging (concrete, steel)	– Efficient use of space – Can avoid obstructions	– May need significant embedment depth – Higher cost than gravity/composite – Not a permanent solution – Soils need to be self-supporting while lagging is placed
Diaphragm (concrete)	– Efficient use of space – Bearing capacity available – Efficient when incorporated in final structure such as basement – Can retain water permanently and fewer joints than piled walls – Good verticality	– May need significant embedment depth – Higher cost than gravity/composite – May affect groundwater flows – Construction plant occupies significant site space – Not suited to variable wall alignments
Sheet pile/combi wall (mainly steel, new polymer products becoming available)	– Efficient use of space – Efficient when incorporated in final structure such as basement – No spoil for disposal – Can retain water – Economic – Re-useable	– May need significant embedment depth – Higher cost than gravity/composite – May affect groundwater flows – Installation sensitive to obstructions or hard ground – Maximum length 30m – Corrosion issues
Composite		
Cantilever (T, L) (reinforced concrete)	– Conventional reinforced concrete construction – Can be prefabricated	– Formwork needed

Table 8.3 Preliminary sizing of retaining structures

<table>
<tr><th>Type</th><th colspan="2">Typical retained height</th><th colspan="2">Typical dimensional ratios[a]</th><th>Typical structural dimensions</th></tr>
<tr><td colspan="6">Gravity[b]</td></tr>
<tr><td>Mass</td><td colspan="2">Up to 3m
(can be much greater)</td><td colspan="2">Width to retained height ratio = 0.4 to 0.6 depending on founding material</td><td>–</td></tr>
<tr><td>Gabion</td><td colspan="2">Up to 10m
(often tilted at 10/20° to vertical)</td><td colspan="2">Trapezoidal, base width to retained height = 0.4 to 0.75 depending on retained height</td><td>Multiple of $2 \times 1 \times 1$m</td></tr>
<tr><td>Modular</td><td colspan="2">3m (can be larger)</td><td colspan="2">–</td><td>Stem from 0.2m</td></tr>
<tr><td>Crib</td><td colspan="2">Up to 7m</td><td colspan="2">–</td><td>Width from 1.2m</td></tr>
<tr><td colspan="6">Embedded</td></tr>
<tr><td></td><td>Cantilever</td><td>Propped[c]</td><td>Cantilever</td><td>Propped[c]</td><td></td></tr>
<tr><td>Contiguous</td><td>Up to 5m</td><td>Up to 20m</td><td rowspan="5">Typical ratio of embedment to retained height = 1 to 2</td><td rowspan="5">Typical ratio of embedment to retained height = 0.5 to 1
Prop spacing = 4m (vertical)</td><td>Diameter from 0.3 to 1.2m</td></tr>
<tr><td>Secant</td><td>Up to 6m</td><td>Up to 25m</td><td>Diameter from 0.45m (soft), 0.6m (firm), 0.75m (hard) to 1.2m</td></tr>
<tr><td>King post</td><td>Up to 4m</td><td>Up to 20m</td><td>Spacing 1 to 3m</td></tr>
<tr><td>Diaphragm</td><td>Up to 8m</td><td>Up to 90m</td><td>Thickness from 0.6m, width 2.8m</td></tr>
<tr><td>Sheet/ Combi[d]</td><td>Up to 5m</td><td>Up to 20m</td><td>Section depth from 0.15m</td></tr>
<tr><td colspan="6">Composite</td></tr>
<tr><td>Cantilever</td><td colspan="2">Up to 8m (higher with counterforts)</td><td colspan="2">Base width to retained height ratio = 0.45
Stem one third width back from toe</td><td>Width from 0.2m</td></tr>
</table>

Notes

a Ratios need to be increased if adversely sloping ground, water pressures, surcharges, compaction pressures or poor foundations are included.

b Crib, gabion and modular walls are typically laid back at 1:4 to 1:10 though they can be near vertical for retained heights of say up to 3m. Faces should be laid back to at least 1:50 to avoid the illusion of the wall leaning forward. A tilted base, shear key or small back batter will help to reduce the size of the structure. On poor ground a structural footing may be required.

c Propping (or anchoring) reduces structural forces and deflection. Additional propping has little effect on the required embedment.

d Sheet piles are typically U- or Z-shaped. Z-shaped piles have better water retention properties and higher effective section modulus. The effective section modulus of U-shaped sections can be improved by welding.

8.7 Lateral wall movement

The consideration of lateral wall movement involves consideration of:
– mobilisation of earth pressures (causing lateral and rotational movements)
– mobilisation of structural strength (wall flexure as a function of its stiffness).

Gravity and composite walls are constructed in a prepared excavation/cutting and then backfill is placed behind them. Embedded walls are either driven into the ground or cast-in-situ in excavated bores after which the ground in front of them is excavated.

As a consequence of the construction method that the different categories of wall experience they are subjected to different lateral movements or deflections. Hence, gravity walls are typically only subjected to movements due to mobilisation of earth pressures while the other two forms of wall experience movement due to mobilisation of earth pressures and flexural stiffness.

EC7 provides guidance on lateral wall movement due to the mobilisation of earth pressure behind vertical walls retaining horizontal ground in drained soils with an initial stress state of $K_0 < 1$, as set out in Table 8.4.

Linear and parabolic interpolation for intermediate values of earth pressure can be adopted for active and passive pressures respectively.

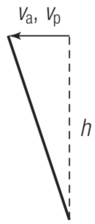
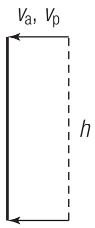
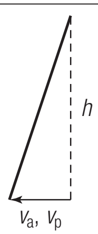
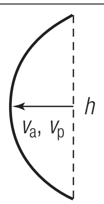
Mobilisation of structural stiffness is not addressed by EC7 and the reader is directed to the accompanying structural Eurocodes.

Guidance on lateral wall movement for embedded walls in stiff clay/competent soil is provided in CIRIA Report C580[35].

8.8 Backfill

Where backfill is used, such as for gravity retaining structures, it should:
– be free draining
– be durable
– have a high shear strength
– have a high stiffness
– contain no deleterious material
– not be clay or uniformly graded silt (though there is a trend for their use in relation to sustainability)
– be compacted.

Table 8.4 Wall movement and mobilisation of earth pressure (vertical wall, horizontal ground surface, drained and non-cohesive soil, initial stress state $K_0 < 1$)

Kind of movement	Loose soil		Dense soil	
	v_a/h (%)	$v_p/h^{a,b}$ (%)	v_a/h (%)	$v_p/h^{a,b}$ (%)
 Rotation about toe	0.4 – 0.5	7 (1.5) – 25 (4)	0.1 – 0.2	5 (1.1) – 10 (2.0)
 Lateral (sliding)	0.2	5 (0.9) – 10 (1.5)	0.05 – 0.1	3 (0.5) – 6 (1.0)
 Rotation about prop	0.8 – 1.0	6 (1.0) – 15 (1.5)	0.2 – 0.5	5 (0.5) – 6 (1.3)
 Bending deflection	0.4 – 0.5	–	0.1 – 0.2	–

Notes
a Values in brackets are those to mobilise half the passive earth pressure.
b Values should be increased by a factor of 1.5 to 2.0 for ground below the water table.
c v_a is the wall movement to mobilise active earth pressure, h is the wall height.
d v_p is the wall movement to mobilise passive earth pressure.

8.9 Temporary support

Temporary support can generally be divided into that required for embedded retaining structures and that required for gravity/composite retaining structures.

For embedded retaining structures, support can take the form of steel or concrete props, anchors or soil berms whereas for gravity and composite structures, temporary support is typically achieved by a temporary cut slope or construction in bays.

The benefit of a propped system in terms of providing support and reducing deflections will depend upon spacing and structural stiffness. The structural design of props will fall under other Eurocodes (notably EC2 and EC3).

The benefit of berm support will depend upon the bench width, height and slope. Berms will allow deeper excavation at the centre of a site which may then be used to install reaction (thrust blocks or part built slab) to raking props. The retaining wall movement needed to mobilise berm resistance will be more than that of a prop.

The benefit of an anchored system (which is specialised works and not considered in detail in this *Manual*, see Section 9.6 for a suggested design methodology for ground anchors) is that it provides for an open excavation and can further reduce wall deflection. Anchors are installed beyond the perimeter of the excavation and thereby often require permissions from third parties in terms of construction beyond property ownership (removable systems can be used to lessen the impact) and design life (typically short which may become an issue if construction is paused for an extended period). Ultimate capacity (failure load) is generally limited to around 0.2 to 3MN[33] depending on ground conditions and anchor length.

Design of temporary stability should be of a standard equivalent to that of the permanent works albeit based on different design situations. Reduced partial factors may be adopted in certain circumstances (see Chapter 2) for temporary works depending on the consequences of failure. Responsibility for temporary support design should be clearly identified. Selection of a support system should consider construction sequencing and the effect of the temporary support on the retaining structure.

8.10 Retaining structures in basements

For retaining structures in basements, preliminary design issues that typically arise relate to restrictions on space, quality of basement space (the basement environment) and adjacent structures. For example:

– Making adequate allowance for internal walls and drainage cavities when considering architectural layout.
– Considering the construction tolerances of piles horizontally and vertically.
– Allowing for long-term movements due to dissipation of excess pore-water pressures in the soil and relaxation of concrete walls.
– Agreeing the grade of the basement space in terms of waterproofing.
– Choosing a wall type that is compatible with the grade of basement desired and whether groundwater control is needed.
– Providing adequate support to adjacent buildings.

Contiguous piles, combi walls, king post walls and potentially hard/soft secant walls do not provide permanent groundwater control.

Achievable tolerances are typically $\pm$75mm ($\pm$25mm with a guide wall) or better for horizontal positioning at piling mat level and 1:75 to 1:150 vertically below piling mat level depending on the structural stiffness of the piling equipment used.

8.11 Execution standards

The following Execution Standards are applicable to the design of retaining walls within the scope of this *Manual*:
– BS EN 1536:2010 – bored piles[8]
– BS EN 1538:2010 – diaphragm walls[10]
– BS EN 12063:1999 – sheet pile walls[11].

For construction in the UK, use of the publication *ICE Specification for piling and embedded retaining walls*[38] is recommended.

8.12 Actions

Actions were introduced in Chapter 2. Additional requirements for backfill, surcharges, water, impact and temperature for retaining structures are addressed in Table 8.5.

8.13 Ground level

Issues relating to geometry were introduced in Chapter 2. Additional considerations for retaining structures in terms of ground level include:

Table 8.5 Retaining wall actions – additional requirements

Action	Requirements
Unit weight of backfill	– Design values to be estimated from knowledge of available material.
Surcharges	– Design values to take account of, for example, nearby buildings, vehicle loading, construction plant, stockpiles and containers. – Consider the effect of repeated loading.
Unit weight of water	– Design values to take account of water salinity and chemical or contaminant content, if significant.
Seepage forces	– Consider seepage forces resulting from different groundwater levels on either side of a retaining structure.
Temperature effects	– Consider effects of temperature changes (e.g. for temporary prop loads and displacements subjected to daily and annual temperature variations). – Special precautions are to be taken to prevent the formation of ice lenses in the ground behind retaining structures (e.g. suitable backfill, drainage, insulation).

- Design values are to consider variation in field values, anticipated excavation and possible scour.
- For ULS calculations where stability is dependent upon the ground in front of a wall (passive side ground), an appropriate value for over excavation Δa (see Chapter 2) for a normal level of site control is the lesser of:
  - 10% of the wall height above excavation level, to a maximum of 0.5m, for cantilevered walls.
  - 10% of the distance between the lowest support and excavation level, to a maximum of 0.5m for supported walls.
- Δa may be decreased below these suggested limits when tight construction control is put in place to prevent overdig (albeit it is not recommended that this is done without careful consideration); similarly, Δa may need to be increased where significant uncertainty in ground level is possible.
- The use of Δa is related to site control and cannot be used to offset soil degradation or softening which should be allowed for in the design parameters independent of Δa.

8.14 Water

Issues relating to water were introduced in Chapters 2 and 4 (in particular Section 4.2.4 provides recommendations for choice of groundwater level for retaining wall design). Additional considerations for retaining structures in terms of water level and pressure include:

- Design or characteristic values for free water and phreatic surfaces are to be selected on the basis of hydraulic and hydrogeological conditions at the site (desk study and ground investigation data).

- Effects of variation in permeability on groundwater regime and how this will influence the pore-water pressure regime around a retaining structure.
- Possibility of unfavourable water pressure due to perched or artesian water tables.
- Design or characteristic values for water pressures are to consider water levels above and below ground level.
- Effects of non-steady state (when water levels and pore-water pressures are changing with time) and steady state (when water level and pore-water pressures are constant with time) conditions where there is a possibility of a sudden change in free water level.
- That upward seepage and heave on the passive side of a retaining structure reduce effective stresses and therefore the ability of the soil to provide passive resistance.
- The effect of ground anisotropy on water flow.
- Account for water pressures in combinations of actions.
- In retained ground of low/medium permeability (silts and clays), water pressures should be assumed to act behind a wall with values corresponding to a hydrostatic water table at the surface of the retained ground unless a drainage system is employed (see Section 8.5) or infiltration prevented. See also comment in Section 4.2.4 relating to UK practice.
- Possible effects of water filled tension/shrinkage cracks where no special measures are taken to prevent the build-up of water.
- Adopting the maximum possible water level as the design value if drainage system adequacy cannot be demonstrated and maintenance ensured.
- Effects of drainage, favourable and unfavourable, natural and artificial, and in light of future maintenance.
- Surface water and groundwater from rain, flood, burst water mains or other source.
- Change in pore-water pressure due to vegetation growth or removal.
- Unfavourable water levels resulting from catchment area increase, drainage blockages, freezing or other causes.
- Exercise caution when choosing to adopt sub-hydrostatic destabilising water pressures.

8.15 Earth pressures

8.15.1 General

The determination of earth pressure involves consideration of both ground and groundwater with the method of interpretation varying according to whether undrained or drained conditions are being considered. It also includes consideration of appropriate limit modes (see Section 8.16), and associated movement and strain. The following information on earth pressures is based on soils; however, it can also be applied to soft and weathered or strong and highly fractured rocks which can be modelled in terms of soil parameters (c' and φ' parameters).

Calculating the magnitude of earth pressures and the directions of resulting forces includes consideration of:
– surcharges and ground surface slope
– wall inclination to the vertical
– water tables and groundwater seepage forces
– wall movement relative to the ground (mobilisation of friction between wall and supported/supporting ground)
– horizontal and vertical equilibrium of the wall
– ground properties (shear strength, unit weight, swelling potential and rock fabric/discontinuities)
– passive side softening and disturbance
– wall and support stiffness
– wall roughness
– shape of failure surface
– selected limit state (i.e. SLS or ULS)
– initial ground conditions and structural stiffness
– allowable wall and supported ground deformation at SLS.

The degree to which wall friction, $\sigma_n \tan \delta$, and/or adhesion a is mobilised should be considered in terms of:
– ground strength
– friction properties of the wall-ground interface
– wall movement relative to the ground
– capacity of wall to support vertical forces.

The mobilised shear strength τ can be calculated as follows:

$$\tau = \sigma_n \tan \delta + a$$

where:
σ_n is the normal stress on wall
a is the adhesion (a function of c' or c_u; $a = 0$ may be appropriate when using φ_{cv})
δ is the angle of shearing resistance at wall-ground interface (δ_d = design value)

The following values for $\delta_d/\varphi_{cv;d}$ are given in EC7 Part 1[1]:

0.667 (concrete panel or steel in sand or gravel)
1.0 (concrete cast against soil)
0.0 (steel sheet pile in clay under undrained conditions immediately after driving).

φ_{cv} is the critical state angle of shearing resistance ($\varphi_{cv;d}$ = design value)

Whilst the prescribed use of φ_{cv} may be logical given its relationship to ϕ' and ultimate limit states, obtaining its value from standard laboratory testing is not easy. However, some guidance is given in Section 4.8 of PD 6694[34] and in Section 2.2 of BS 8002[30] along with example values.

8.15.2 At-rest earth pressure – K_0

The at-rest earth pressure, taking account of stress history, is used when there is no movement of a wall relative to the ground. For normally consolidated soil, at-rest conditions are normally assumed for movement of less than 0.0005 times the height of the wall.

There are several similar equations for calculation of K_0, the suggested equation in EC7 is as follows:

$$K_0 = (1 - \sin \varphi')(1 + \sin \beta)\sqrt{OCR}$$

where:

φ' is the angle of shearing resistance
β is the upwards slope angle of ground behind the wall ($0 \leq \beta \leq \varphi'$)
OCR is the over-consolidation ratio (it should be noted that the equation is not appropriate for very high OCR values).

The resulting force is parallel to the ground surface.

CIRIA C580[35] presents details of assessment of K_0 for stiff over-consolidated clays along with suggestions of how wall installation modifies K_0 prior to the excavation phase of construction.

8.15.3 Intermediate earth pressure

If there is insufficient movement to mobilise the active or passive limits, intermediate values of earth pressure will occur. This should be considered to occur when, for example, struts, anchorages or similar elements restrain the movement of a retaining structure. The magnitude of movement required to mobilise the active and passive limits is presented in Section 8.7. Intermediate values of earth pressure may be determined using these movements and linear interpolation for active earth pressure or parabolic interpolation for passive earth pressure. Alternatively empirical rules, spring constant methods or finite element methods could be used to calculate appropriate active and passive side earth pressures based on wall movement and structural stiffnesses (wall and propping stiffnesses).

8.15.4 Limit earth pressures – K_a and K_p

The active and passive limit values of earth pressure on a vertical wall can be calculated as follows:

$$\sigma_a(z) = K_a(\gamma z + q - u) + u - c K_{ac}$$
$$\sigma_p(z) = K_p(\gamma z + q - u) + u + c K_{pc}$$

where:

$\sigma_a(z)$ is the stress normal to the wall at depth z (active earth pressure)
$\sigma_p(z)$ is the stress normal to the wall at depth z (passive earth pressure)

K_a is the coefficient of active earth pressure
K_p is the coefficient of passive earth pressure
γ is the unit weight of ground
z is the depth
q is the surface surcharge
u is the pore-water pressure
c is the cohesion of ground (c' – drained, c_u – undrained)
K_{ac} is the is the active earth pressure coefficient to model soil cohesion
K_{pc} is the passive earth pressure coefficient to model soil cohesion

$$K_{ac} = \min\left\{2\sqrt{K_a\left(1+\frac{a}{c}\right)}, 2.56\sqrt{K_a}\right\}$$

$$K_{pc} = \min\left\{2\sqrt{K_p\left(1+\frac{a}{c}\right)}, 2.56\sqrt{K_p}\right\}$$

For retaining walls the adhesion a is usually taken to be equal to 0kN/m^2 for drained analysis. For undrained analysis (temporary conditions in clays) the value of adhesion a is usually taken to be $0.5c_{u;d}$ to allow for disturbance during wall installation[35].

For drained soil K_a and K_p are functions of φ', δ, β and θ (angle of wall back from vertical) whereas for undrained conditions, $K_a = K_p = 1$. Figures 8.2a–8.2d[1] provide the horizontal component of K_a and K_p for two common situations:

$\beta = 0$, $\theta = 0$, $0 \le \delta/\varphi' \le 1.0$

$\beta > 0$, $\delta/\varphi' = 0.66$, $\theta = 0$

Relatively highly fractured or soft rock can potentially be modelled in terms of φ' and c' using soil models. In relatively massive/hard rock where joint/discontinuity orientation and spacing dominate, soil models are often not appropriate for use in retaining wall design; it is, however, possible to derive equivalent K_a and K_p parameters from rock mass parameters when rock mass structure is not aligned to active and passive failure surfaces. Specialist advice should be sought where retention of rock masses is being considered.

In soft/normally consolidated soils it should be recognised that the active pressure on the back of a retaining wall may exceed the passive pressure on the front of a retaining wall for a significant depth below excavation level. This can lead to structural resistance and deformation issues which may require the installation of cross panels or jet grouting to limit wall depth and ground movements. Specialist advice should be sought in this situation. See also Brand and Brenner[100].

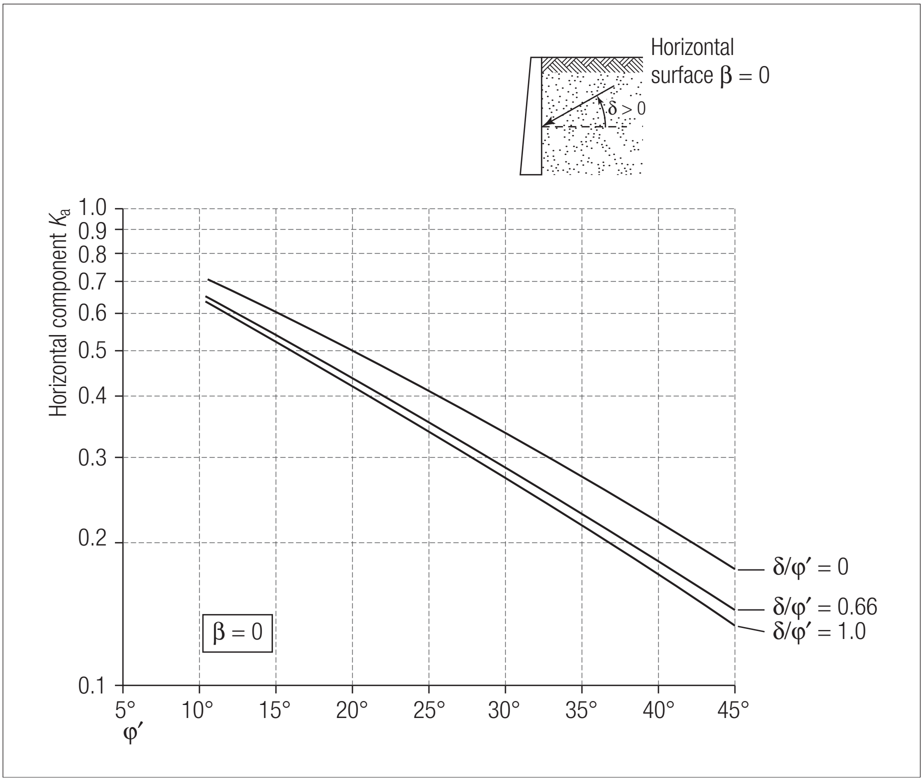


Fig 8.2a K_a chart – horizontal surface, δ/φ' variable (after Annex C EC7 Part 1)

8.15.5 Compaction pressures

Earth pressures behind a wall are to include pressures resulting from the placing and method of compaction of backfill. Appropriate compaction procedures and plant need to be specified by the designer with the aim of avoiding excessive additional earth pressures.

As a part of design it is necessary to state the compaction plant that has been allowed for in the design. This information must be included in construction information as it is related to safe construction of the wall (CDM issue, see Section 10.2).

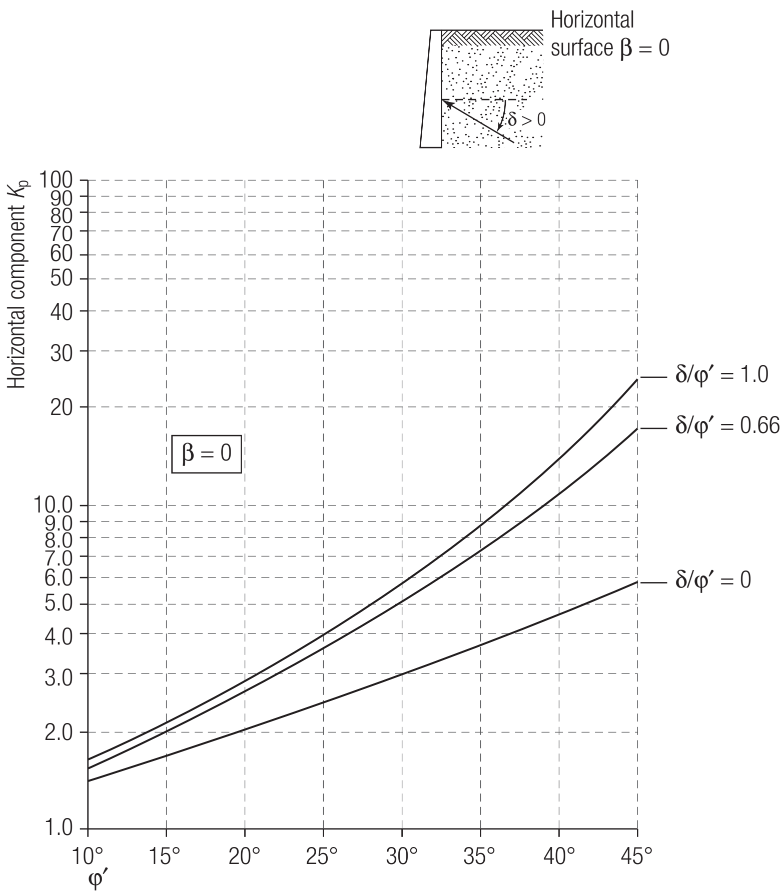


Fig 8.2b K_p chart – horizontal surface, δ/φ' variable

8.16 ULS design – GEO and STR

8.16.1 Geotechnical

Retaining structures and supporting structural elements are checked at the ultimate limit state for appropriate design situations and relevant limit modes using design values. Geotechnical limit modes of failure (other than those for overall stability which are addressed in Chapter 5) to be considered as a minimum are illustrated in Figures 8.3 and 8.4[1].

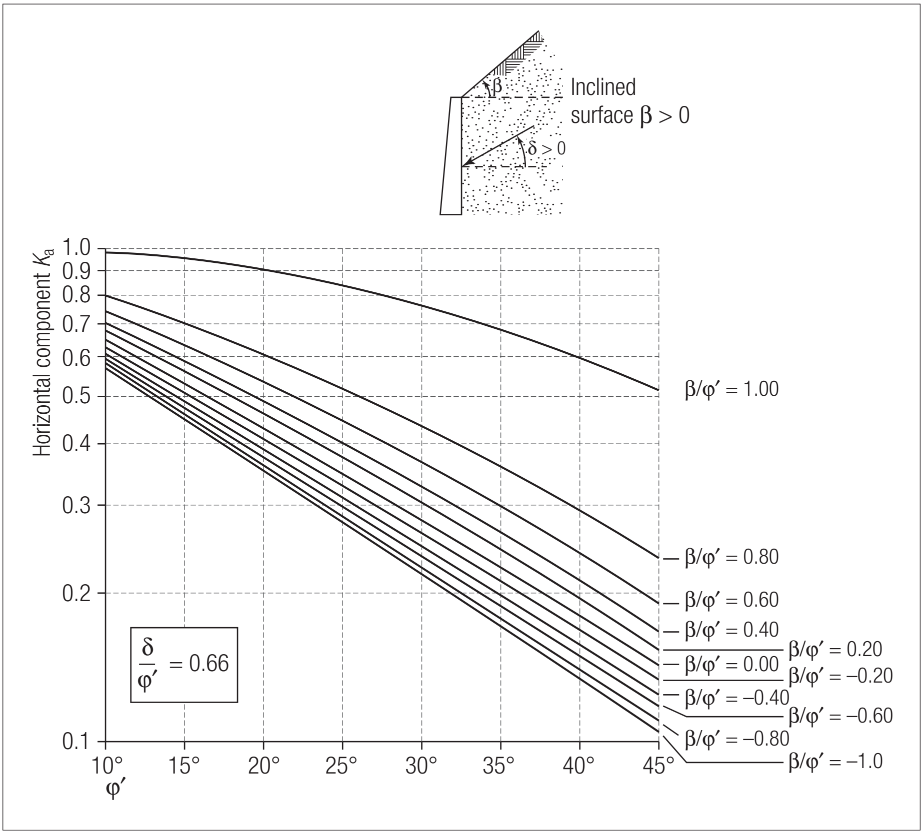


Fig 8.2c K_a chart – inclined surface, $\delta/\varphi' = 0.66$

Furthermore, design includes:

- Showing by calculation that equilibrium can be achieved using design values of actions and material properties.
- Considering the compatibility of deformations when assessing design strengths/resistances.
- Using the more adverse of upper and lower design values for design strengths/resistances.
- Considering short and long-term behaviour.
- Checking safety against failure due to hydraulic heave and piping.
- Using the content of Chapter 6 as appropriate to demonstrate that failure is sufficiently remote and deformations are acceptable.
- Showing that rotational failure of embedded walls is prevented by sufficient wall penetration into the ground.
- Ensuring that shear stress vectors between the soil and wall are consistent with the relative vertical displacement that would occur for the design

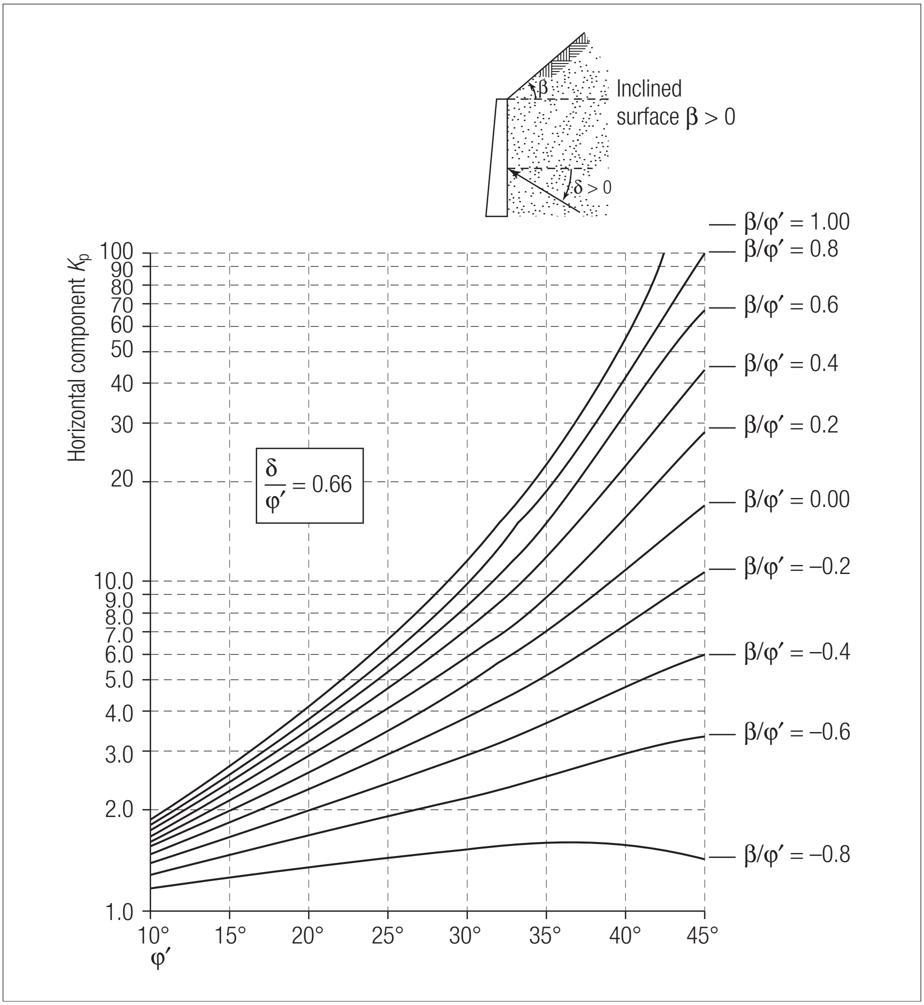


Fig 8.2d K_p chart – inclined surface, $\delta/\varphi' = 0.66$

situation being considered (e.g. if the wall is required to carry vertical load this should be accommodated in the calculation of active and passive pressure coefficients).

– Ensuring that shear stress vectors between the soil and wall are consistent with checks for vertical and rotational equilibrium.
– Using the content of Chapter 7 to check vertical equilibrium where a wall acts as a foundation.

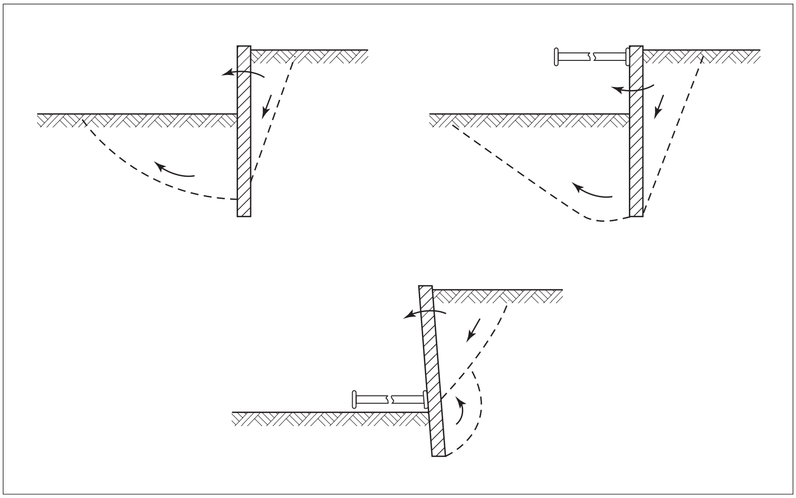


Fig 8.3 Examples of limit modes for rotational failure of embedded walls (after Section 9.7.4 EC7 Part 1)

8.16.2 Interaction with structural design

Structural limit modes of failure to be considered as a minimum are illustrated in Figure 8.5[1].

Design includes:
– Showing that the required structural strength can be mobilised with compatible deformations in the ground and structure.
– Using EC2 to EC6 as appropriate, including verification of structural strength and consideration of deformation induced reductions in strength.

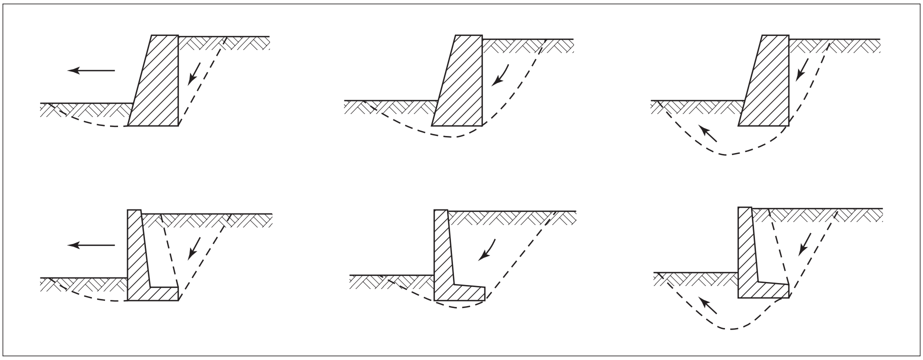


Fig 8.4 Examples of limit modes for foundation failure of gravity walls (after Section 9.7.3 EC7 Part 1)

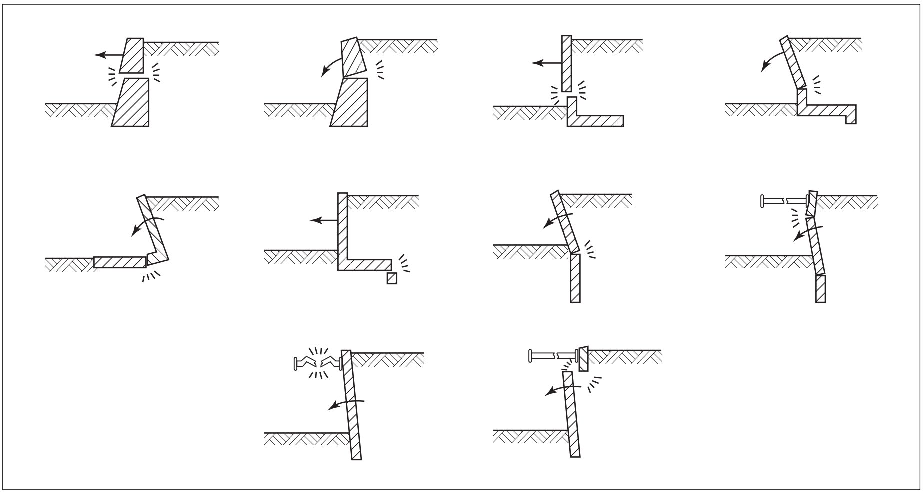


Fig 8.5 Examples of limit modes for structural failure (after Section 9.7.6 EC7 Part 1)

The structural design of a wall element should consider the actions which it will experience during its design life. For preformed piles or wall elements this will also include handling and installation actions.

EC3 Part 5[6] deals specifically with sheet piles. Of particular importance to the design of sheet pile retaining walls is the need to correctly assess bending stiffness for computation of deflection and combined section modulus for assessment of bending capacity. For U-shaped sheet pile panels, the location of the interlock is unfortunately coincident with the neutral axis of the wall thereby reducing shear capacity at the location of maximum shear requirement. Reduction factors β_B and β_D for the section modulus z and second moment of area I are provided in Table 8.6[61].

The effect of corrosion on steel sheet piles must also be included in assessment of the design section modulus and the design second moment of inertia used in design.

Similarly, the design flexural stiffness of retaining walls made out of reinforced concrete should be considered. This is generally taken to be:

EI (short term) $= 0.7 \times EI$

EI (long term) $= 0.5 \times EI$

where:

E is the short term Young's modulus of concrete

I is the second moment of area of the full section[35].

Table 8.6 Reduction for U-piles for section modulus and second moment of area (after UK NA to EC3 Part 5)

Type of U-pile unit	Number of structural levels of support[a]	Reduction factors β_B on section modulus and β_D on second moment of area[b,c]					
		Highly unfavourable conditions[d]		Unfavourable conditions[e]		Favourable conditions[f]	
		β_B	β_D	β_B	β_D	β_B	β_D
Singles or uncrimped doubles	0	0.40	0.30	0.50	0.35	0.60	0.40
	1	0.55	0.35	0.60	0.40	0.70	0.45
	>1	0.65	0.45	0.70	0.50	0.80	0.55
Crimped or welded doubles	0	0.70	0.60	0.75	0.65	0.80	0.70
	1	0.80	0.70	0.85	0.75	0.95	0.80
	>1	0.90	0.80	0.95	0.85	1.00	0.90

Notes

a Restraint is any support element which results in shear force changing between positive and negative values. Wall toe embedment is not included in this assessment.

b Values may be increased by 0.05 if joint is not treated with a lubricant or sealant (max value 1.00).

c Structural requirements for crimping, see EC3 Part 5[6] and its UK NA[61].

d Highly unfavourable conditions include:
- Retaining substantial depth of water
- Significant presence of very low strength or very loose soils
- Soil disturbed to ease installation by pre-augering in fine soil and water jetting at a rate greater then 240litres/min fine soil and 480litres/min in coarse soil.

e Unfavourable conditions include:
- Significant presence of low strength or loose soils
- Soil disturbed to ease installation by pre-augering in coarse soil and water jetting at a rate between 60litres/min and 240litres/min in fine soil and between 240litres/min and 480litres/min in coarse soil.

f Favourable conditions may be assumed if none of the highly unfavourable or unfavourable conditions apply.

g This Table is derived from NA to EC3 Part 5[61].

8.17 ULS design – UPL and HYD

8.17.1 General

Design for hydraulic failure is beyond the scope of prescriptive design. However, an understanding of the issue associated with hydraulic failure is important at preliminary design stage such that it can be included in the ground investigation and detailed design stages which follow. In the context of this *Manual*, hydraulic failure is considered to be limited to excavations and basements, hence its inclusion in this Chapter.

Within EC7 hydraulic failure is identified in terms of four ultimate limit state ground failure modes caused by water:

UPL:
– Uplift (water pressure under a structure or low permeability stratum > structural load plus overburden)
HYD:
– Hydraulic heave (upward seepage reduces σ'_v to zero)
– Internal erosion (transport of soil particles)
– Piping (a sub-class of internal erosion comprising regressive pipe shaped erosion)

Only failure by uplift needs to be checked if the pore-water pressure is hydrostatic (i.e. negligible hydraulic gradient). The hydraulic gradient is the change in piezometric level with depth or plan position.

The following factors are to be considered when determining hydraulic gradients, pore-water pressures or seepage forces:
– soil permeability variation in time and space
– water levels and pore-water pressures and their variation in time
– changes to boundary conditions.

High hydraulic gradients (i.e. those that pose a significant danger to the proposed construction, for example water flows around a retained excavation in sand below the water table) are required to be reduced. In general, hydraulic gradients can be reduced by measures such as:
– increasing the seepage path with physical barriers (e.g. a grout curtain or a deeper retaining wall toe level)
– providing more resistance (e.g. increasing structural weight)
– controlling seepage (e.g. drains etc.)
– using filters
– providing slope protection
– installing relief wells.

8.17.2 UPL – uplift

The assessment of failure by uplift requires a comparison of the permanent and variable destabilising actions (e.g. pore-water pressure beneath a structure or stratum) with the permanent stabilising actions (e.g. weight, side friction) and resistances (e.g. anchors, tension piles).

UPL is to be checked for situations such as:
– a buried hollow structure with a water table above its base
– an excavation into low permeability ground which overlies a confined aquifer.

The design inequality for UPL that must be satisfied is given in Section 2.11.3.5 and is repeated below:

$$G_{dst;d} + Q_{dst;d} \leq G_{stb;d} + R_d$$

where:

$G_{dst;d}$ is the design value of the destabilising permanent action
$Q_{dst;d}$ is the design value of the destabilising variable action
$G_{stb;d}$ is the design value of the stabilising permanent action
R_d is the design value of the resistance to an action (e.g. side shear on a basement box)

Commonly adopted measures for avoiding uplift include:
– increasing permanent stabilising actions (e.g. weight)
– decreasing destabilising actions (e.g. dewatering)
– adding resistance (e.g. tension piles or anchors).

The design resistances for tension piles and side friction are to conform to the GEO/STR requirements elsewhere in this *Manual*. Anchor design is beyond the scope of this *Manual*.

For UPL, the characteristic uplift pressure is based on worst credible water level and is considered to be a permanent rather than a variable action. To obtain the design destabilising action, the characteristic uplift pressure is factored by 1.1 (i.e. the uplift from water pressure is factored by 1.1 when the worst credible level is used).

When using this approach to uplift water pressures it is likely that variable destabilising actions will be equal to zero for typical UK buildings.

For preliminary assessment of potential for uplift on a building basement, the inequality for UPL can be simplified by assuming destabilising variable actions are zero and no shear resistance R_d is available on the sides of the structure. Hence the inequality becomes (with partial factors incorporated):

0.9 $\times$ characteristic self-weight (permanent) $\geq$ 1.1 $\times$ force from uplift pressures acting on structure

The inequality must be checked for the entire weight of the structure as well as individual areas of the structure if the structure is not capable of redistributing stabilising actions to locations where they are required so as to prevent local failure or distortion. For this inequality the groundwater level or pore-water pressure should be based on the worst credible water level; if a lower level is used then the partial factors would need to change. If in doubt the groundwater level should be the highest ground level adjacent to the site taking account of the potential for artesian water pressures (e.g. if the site is located adjacent to higher ground levels with permeable layers overlain by layers of lower permeability).

In situations where earth heave pressure is being generated against the base of a buried structure then these heave pressures may work in combination with uplift water pressures. Such pressures will reduce reasonably rapidly with movement but may be important in the design of pile tie down forces or slab pressures. Calculation of such pressures should be carried out by a geotechnical specialist.

8.17.3 HYD – hydraulic heave

While failure by uplift considers water pressures acting directly on the structure, failure by hydraulic heave considers the effect of water moving upwards within the soil mass resulting in heave and softening, possibly to quick conditions (loss of strength), of the ground.

The assessment of failure by hydraulic heave requires a comparison of the destabilising action (pore-water pressure or seepage force) with the stabilising action (total vertical stress or weight of the soil column).

HYD is to be checked for a situation such as seepage under an embedded wall after excavation to one side.

The design inequalities for HYD that must be satisfied are given in Section 2.11.3.6 and are repeated below:

$u_{dst;d} \leq \sigma_{stb;d}$ (a total stress comparison)

or

$S_{dst;d} \leq G'_{stb;d}$ (an effective stress comparison)

where:

$u_{dst;d}$ is the design value of the destabilising water pressure
$\sigma_{stb;d}$ is the design value of the stabilising total stress in the ground
$S_{dst;d}$ is the design value of the destabilising seepage force in the ground
$G'_{stb;d}$ is the design value of the stabilising permanent vertical actions for heave verification (submerged weight)

It should be noted that where the soil has significantly low permeability, the mode of failure changes to UPL.

HYD is illustrated in Figure 8.6, where water is seen to flow from the perched water table on the active side of the wall to the formation level on the passive side of the wall. In the sand strata the water table the pore-water pressure gradient is hydrostatic. In the clay below the sand the pore-water pressure is below hydrostatic on the active side, height h_{a2}, and above hydrostatic on the passive side, h_p.

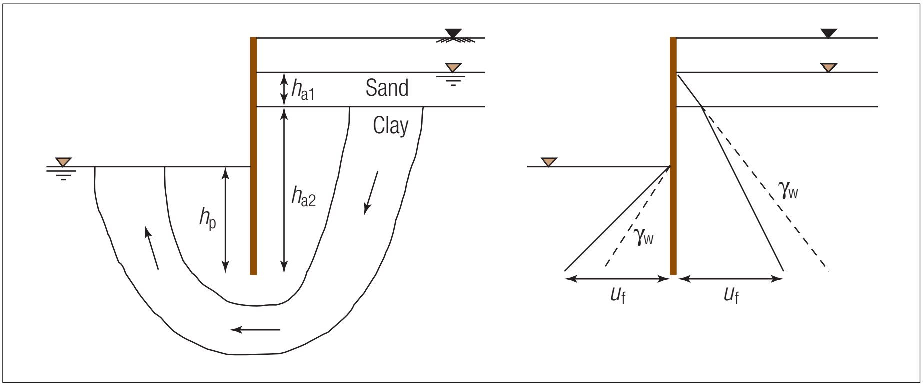


Fig 8.6 Seepage around an embedded retaining wall

The value of the pore-water pressure at the toe of the retaining wall u_f is given by the following equation which assumes that the permeability in the clay strata is constant with depth:

$$u_f = \gamma_w h_p + \gamma_w h_p \frac{(h_{a1} + h_{a2} - h_p)}{(h_{a2} + h_p)}$$

The first term $\gamma_w h_p$ is the equivalent hydrostatic head on the passive side assuming no head loss in the sand layer; the second term accounts for the change in water pressure from a hydrostatic stress distribution in the clay layer. The second term, $(h_{a1} + h_{a2} - h_p)/(h_{a2} + h_p)$, includes an assessment of the hydraulic gradient; when this approaches 0.8 (assuming a clay density of 18kN/m^3) then heave of the passive ground will occur as the value of u_f approaches the vertical total stress on the passive side.

Whilst not directly related to hydraulic heave, the process of calculating u_f also provides the designer an appropriate pore-water pressure profile around the retaining wall for equilibrium conditions for use in the GEO/STR limit states. In GEO/STR calculations the value of the second term will need to be much less than 0.8 for wall design as a result of the need to mobilise ground resistance on the passive side of the wall.

The characteristic value of the pore-water pressure is to take account of all possible unfavourable conditions (e.g. thin layers of low permeability soil and spatial effects).

Commonly adopted measures for avoiding hydraulic heave include:
– dewatering on the active or passive side
– increasing the total stress on the passive side
– increasing the embedment of the wall.

8.17.4 HYD – internal erosion/piping

Failure by internal erosion and piping relates to water movement through a soil body resulting in migration of soil particles within the soil matrix or filter layers (internal erosion). The development of such particle migration may result in preferential drainage paths in the soil where all soil has been eroded away (piping). Such situations are mainly associated with hydraulic structures such as dams, where onset of internal erosion and piping can have catastrophic consequences. However, they are also pertinent to building works which result in temporary or permanent changes in the groundwater regime and groundwater. Where permanent changes in the groundwater regime are planned it is recommended that consultation with an appropriate specialist is undertaken. These types of failure should be checked separately from heave.

8.18 SLS design

Retaining structures are checked at the serviceability limit state for appropriate design situations.

Limiting values (C_{d} – see Chapter 2) for displacements/distortions of walls and adjacent ground are to be agreed during design. The tolerance of supported structures and services to movement is to be considered when identifying the value of C_{d}.

The design value of the effect of actions E_{d} (i.e. displacement/distortion) required to evaluate the serviceability inequality $E_{\mathrm{d}} \leq C_{\mathrm{d}}$ is initially assessed as a cautious estimate based upon comparable experience and including for the effects of construction of a wall.

However, if:
– the above inequality is not satisfied, or
– nearby structures and services are unusually sensitive to displacement, or
– comparable experience is not well established,
then more detailed investigation and calculations are required.

When considering the potential likelihood of SLS failures then the nature of the ground will be an important consideration. The following generic ground conditions have been identified as being susceptible to SLS failures requiring calculation of E_{d}:
– walls retaining more than 6m of low plasticity cohesive soil
– walls retaining more than 3m of high plasticity cohesive soil
– walls supported by soft clay.

Calculations include consideration of ground/structural element stiffness and construction sequence.

The effect of variable actions behind a wall in contributing to displacement is to be considered.

Calculation models of material behaviour should be calibrated by comparable experience. For linear models, adopted stiffnesses should be compatible with computed deformations.

Calculation of wall movement can be carried out using proprietary software used for wall design and rules of thumb providing indications of ground movement behind the retaining wall (see CIRIA C580[35]) or by means of finite element or finite difference analysis. Assessment of E_d should also include wall installation movements, examples of wall installation movements are provided in CIRIA C580. Such movements will be more important for adjacent structures than for the structure itself, see comments in Chapter 5 on damage to adjacent structures.

8.19 Illustration of design process

Refer to Appendix E.

8.20 Summary

- Retaining structures are gravity walls, embedded walls or composite structures.
- Various modes of failure and limit states need to be considered.
- The majority of retaining structures within the scope of this *Manual* will be Geotechnical Category 2 and therefore are required to be designed by calculation rather than by prescriptive methods.
- Groundwater pressures should be carefully considered. They are often more important than soil pressures.
- Basement stability in terms of uplift failure and hydraulic failure require careful consideration.
- The selection of a retaining structure is not an arbitrary decision.
- Methods for calculating earth pressures and the movements needed to reach the active/passive limits are defined.
- Ground movements require careful consideration.

- The way in which traffic surcharges at retaining walls are calculated has changed.

- Basements, in terms of restrictions on space and environmental conditions, should be carefully considered early in the design process.

- The implications of choosing one type/size of sheet pile over another should be understood (e.g. lower section modulus for U-shape).

9 Special geotechnical works

9.1 Introduction

This section is presented in order to give the reader an appreciation of specialist geotechnical works. The following are addressed: earthworks fill, ground improvements, dewatering, reinforced ground and ground anchorages. It is assumed that a suitably qualified geotechnical engineer would be engaged in relation to the design and specification of these activities. When planning for these special geotechnical works consideration of testing should be made early in the design process to allow programme time for testing and to optimise the design.

9.2 Earthworks – filling

Filling in terms of earthworks (e.g. as may be encountered on a residential estate development to create a construction platform) generally comprises of the placement of natural soil, suitable waste products or crushed rock to raise an existing ground surface to a new level. Fill material can include materials such as selected colliery waste, selected demolition waste, pulverised fuel ash, manufactured lightweight aggregate as well as natural materials encountered on or near to the site.

Fill material is usually selected on the basis of its handling properties, its local availability (reduced transportation costs), and design properties such as strength, stiffness, durability and permeability once placed.

In nearly all cases, fill must be compacted to achieve the required performance. Compaction is usually achieved using rolling compaction plant; vibrating plate plant is also used in small awkwardly shaped areas. Plant and the method of compaction must be selected according to the material being compacted and the location of the work. Compaction of material takes place in layers, typically ranging from 0.1 to 0.3m in thickness (proprietary systems exist for deeper compaction), in order to achieve an adequate and uniform build-up of the ground level. In the UK, compaction is typically undertaken to either a method specification or performance criterion. Method specification requires a set number of passes per layer by a specified piece of plant (this is a presumptive rule based on experience and usually justified on site by assessment). The fill being compacted must have a moisture content that allows it to be densely compacted (not too wet, not too dry). Performance based compaction involves testing in which the density of the compacted fill is assessed and shown to be within acceptable limits (e.g. have a dry density greater than specified percentage of the maximum dry density).

Modification of material for compaction may be required when the material is in a state not suitable for compaction (too wet, too dry, inadequate grading etc.) Modification includes adjusting the water content by drying, wetting or adding lime; for material with poor grading, crushing, sieving or washing may be used.

The suitability of material for compaction should be assessed during ground investigation by means of suitable testing. The following minimum tests are typically carried out:
- Natural moisture content (care to ensure that this is not changed during handling on site).
- Particle size distribution (coarse and fine fractions).
- Atterberg limits for clays/silts.
- Compaction testing (dry density vs. moisture content relationship).
- Sulphate and pH testing.

In general, fill must be inspected or tested to ensure compliance with job specification requirements. In certain circumstances testing may not be required if field trials prove the suitability of a compaction procedure or if comparable experience is available.

The Highway Agency publications *Design Manual for Roads and Bridges*[36] and *Manual of Contract Documents for Highway Works*[39] contain commonly adopted guidance on earthworks, classification of fill material and compaction. Charles' guide to building on fill[101] is a further useful reference.

Failure to achieve adequate compaction may lead to large or uneven settlement under building foundation loads and possibly collapse settlement when the fill material is wetted.

Compaction of fill behind retaining walls is discussed in Chapter 8. Overall stability of earthwork fill in terms of ULS design is considered in Chapter 5.

9.3 Dewatering

Dewatering is often required during construction of temporary works and permanent works. It can also be used in the permanent condition to prevent unacceptable uplift or lateral loading.

Dewatering for the construction of spread foundations below water level in coarse and medium grained soils and for the construction of basements below the water table where there is no obvious cut-off stratum at acceptable depth are common examples of when dewatering is used. Dewatering can also be used to enhance stability during temporary works by means of increasing effective stresses (and thereby shear strength and factor of safety for a given slope geometry).

The design of dewatering systems is best left to specialist consultants/contractors; an awareness, however, of potential dewatering systems and of the pitfalls of dewatering is necessary at scheme design stage.

A dewatering scheme should to be based on site specific data from ground investigation which may include permeability testing in boreholes (and on laboratory samples) and trial well pumping tests. Anisotropy of the ground (layering) is very important in the assessment of how a dewatering system will work; such anisotropy is relevant to discrete strata (clay over sand) and to variations within a single stratum (clay layers/lenses within a predominantly sand stratum).

When considering the use of dewatering it is necessary to assess the potential effects as presented in Table 9.1.

Table 9.1 Dewatering risks and mitigations

Dewatering issue	Example/mitigation
Strata of variable permeability leading to excessive lateral drawdown.	– If the lateral extent of dewatering is likely to be excessive and result in ground settlement outside the immediate area then dewatering may not be appropriate. This is especially the case where structures penetrate the dewatered zone resulting in downdrag forces on piles etc. – If lateral extent of dewatering is shown to be acceptable then careful consideration is necessary to ensure an adequate depth of dewatering.
Adverse settlement of adjacent structures (SLS and ULS assessments to be carried out on adjacent structures).	– Dewatering of high permeability strata (e.g. gravels or fractured rock) which are overlain by soft soils (soft clays or peat) will result in large settlement in the overlying strata. This is easily predictable and likely not acceptable. Use of recharge wells beyond the immediate area of dewatering to limit the extent of the dewatered zone may be considered. Alternatives to recharge include grouted cut-offs and freezing.
Extraction of fines from the soil matrix resulting in ground loss.	– If the soil matrix is eroded during dewatering, flows will likely increase and ground movements will occur; in short, control will be lost. Design of appropriate filters is necessary.
Extracted water requires appropriate disposal.	– Permission to dispose of water at a suitable distance, especially in urban areas where there are limited natural water courses. Consult utility companies or bodies controlling adjacent water bodies.

Table 9.1 Continued

Dewatering issue	Example/mitigation
Provision of a system which can maintain design water levels without excessive variation and with redundancy in case of breakdown.	– Dewatering systems are usually required to maintain a design water level for a fixed period; failure to do so may have safety implications. The dewatering system must be able to accommodate foreseeable water level fluctuations as may exist in tidal areas or areas adjacent to rivers which flood, and be adequately robust to allow periods of maintenance and breakdown. – The effectiveness of a system needs to be checked by monitoring of groundwater levels or pore-water pressures. For long term dewatering, corrosion and clogging (mineral or biological) may be relevant to system design.
Heave or collapse movements of loose soils can occur when dewatering is turned off.	– Correct assessment of recharge is needed prior to dewatering commencement.
Interference with drinking water supply (or with other licensees' rights).	– Extraction licenses are needed, with testing carried out to third party extraction systems being a possible constraint to demonstrate no impact. Compensation payments may be necessary.
Dewatering resulting in contaminant movement.	– Dewatering systems should not cause the movement of contamination.
Deterioration or loss of durability of adjacent structures.	– Dewatering and drawdown should not cause any change to the ground that would cause deterioration of adjacent structures. An example of this is historic timber foundations situated below the water table being exposed to air (oxygen) resulting in deterioration in the timber and building settlement.

An initial assessment of the feasibility of dewatering as a means of groundwater control can be carried out by means of a useful design chart (CIRIA Report C515[102]) as in Figure 9.1.

In using this chart it is necessary to bear in mind the losses that will occur at well points may limit the extent to which a stratum type can be dewatered. For example, a well in gravel overlying clay would not be able to dewater the gravel to the top of the clay.

When using Figure 9.1[102], the representative permeability values in Table 9.2 can be assumed for initial assessment. However, pumping tests should be undertaken at the ground investigation stage to obtain site specific data prior to well design being carried out. Wells should always be tested to

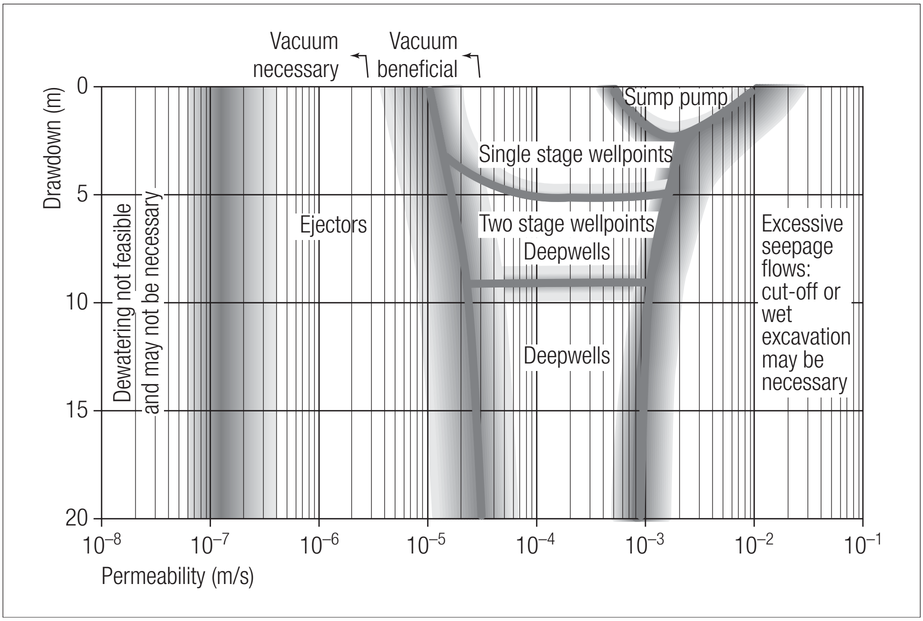


Fig 9.1 Dewatering system and drawdown (after CIRIA C515)

demonstrate the required drawdown in advance of construction works being carried out which rely on well dewatering performance.

Further guidance on dewatering can be found in CIRIA Report C515 *Groundwater control – design and practice*[102].

The design of the effects of dewatering (downdrag on piled foundations, the stability of slopes etc.) should be carried out in accordance with the relevant section of EC7 Part 1[1].

Table 9.2 Example permeability values – preliminary assessment

Soil type	Degree of permeability	Permeability (m/s)
Clean gravel	High	$>1 \times 10^{-3}$
Sand and gravel mixtures	Medium	1×10^{-3} to 1×10^{-5}
Very fine sands/silty sands	Low	1×10^{-4} to 1×10^{-7}
Silt and interlaminated silt/clay/sand	Very low	1×10^{-6} to 1×10^{-9}
Intact clay	Almost impermeable	$<1 \times 10^{-9}$

9.4 Ground improvement

9.4.1 General

Ground to be improved can be natural or man made and improvement can be temporary or permanent. While there are many ground improvement options available, those within the scope of common building types are limited. The following options are typical, other types exist and continue to be developed:
– vibro-stone columns
– stabilised soil columns
– jet grouting
– vegetation.

However, prior to choosing a ground improvement method (if one is required at all) aspects including but not limited to the following require consideration:
– understanding the limitations of a particular method
– data from site specific ground investigation
– durability of chosen technique
– potential adverse impact on the ground or groundwater (e.g. cross contamination between aquifers)
– nuisance during execution of the works
– design responsibility
– engineering and construction risks
– cost.

Guidance on ground improvement can be found in CIRIA Reports C573 *A guide to ground treatment*[103] and C514 *Grouting for ground engineering*[104]. Further discussion of the options introduced above is provided in the following sections.

9.4.2 Vibro-stone columns

Vibro-stone columns are compacted cylindrical columns of gravel. The objective is to form a composite material (stone columns within existing ground) which is stiffer than the existing ground alone. It is not the intention of stone columns to improve the properties of the existing ground, though this may be a side effect in some soils, e.g. sand. The purpose of the existing ground is to provide lateral support to the stone columns. While stone columns have been used in a wide range of soils, they are more suited to finer soils or loose coarse grained soils (e.g. loose made ground). They can be used in intermediate soils (i.e. those between finer and loose coarse grained), however these are better dealt with by vibrocompaction.

Vibro-stone columns are constructed by forming a hole with a crane-suspended vibrating poker (typically 300-450mm diameter) to the desired depth, introducing gravel to the poker base via a hollow stem or side feed tube and compacting it in stages as the poker is withdrawn. Treatment

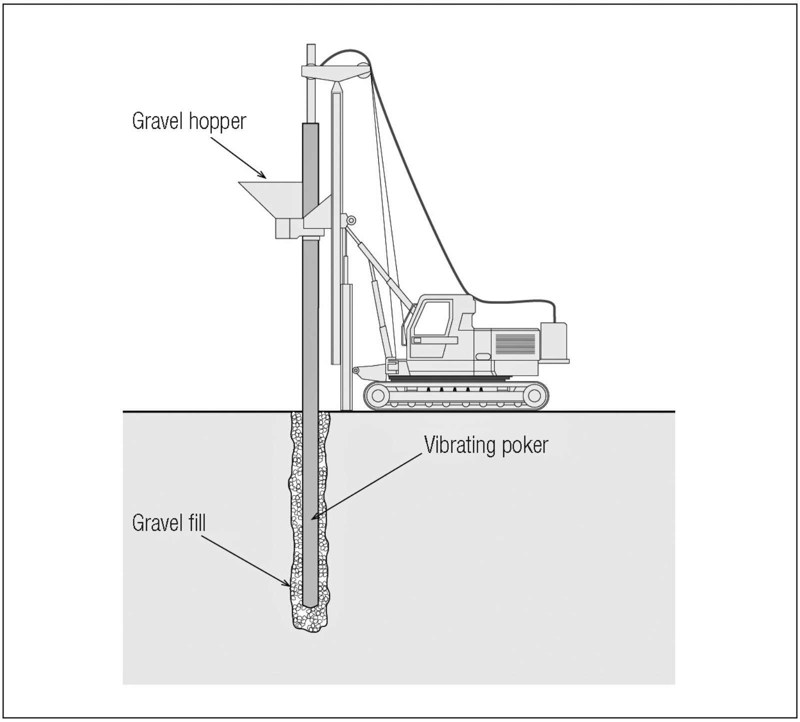


Fig 9.2 Vibro-stone columns

depths are usually less than 10m. See Figure 9.2 for a simple illustration of the process.

A typical approach to assessing the increase in stiffness is to plot the settlement ratio (settlement with columns/settlement without columns) against the area ratio (plan area of columns/plan area of site treated). At a practical area ratio of say 0.25 to 0.35, a settlement ratio of 0.5 is possible in clay.

Stone columns are not suitable in soils that provide little lateral restraint (e.g. very soft soils) or in soils where the initial lateral restraint may degrade over time (e.g. organic soils). It should also be noted that stone columns will form a pathway for water and contamination which may have undesirable consequences in some soils/locations. Plant access needs to be considered.

9.4.3 Stabilised soil columns

Stabilised soil columns are cylindrical columns of lime and/or cement mixed with *in situ* soil. The objective is to form a composite material (lime/cement/soil columns and existing ground) which is stiffer than the existing ground

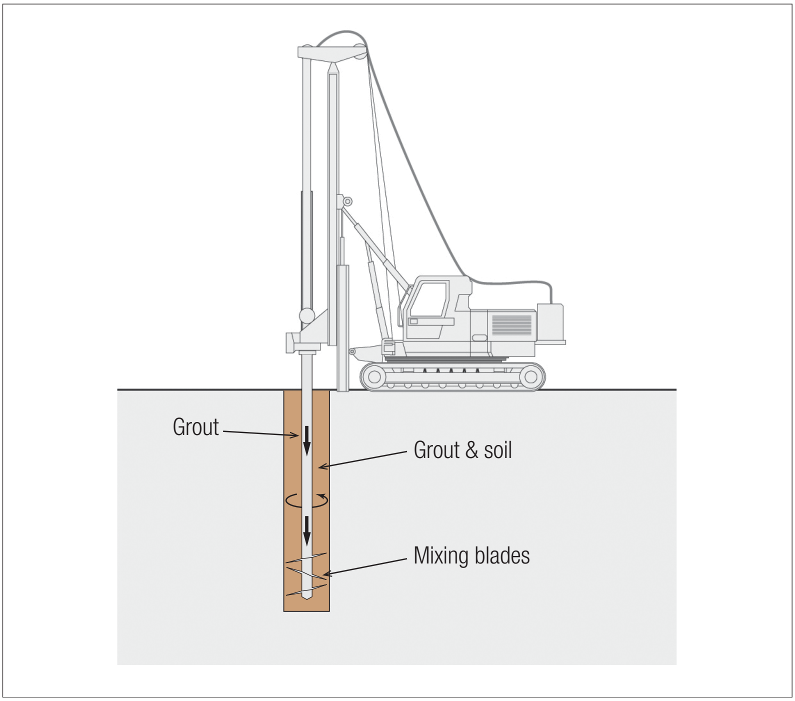


Fig 9.3 Stabilised soil columns

alone. The lime or cement reacts chemically with the soil to change the engineering properties of the soil.

Stabilised soil columns are constructed by drilling a mixing tool to the desired column depth, introducing lime and/or cement (dry or wet) to the end of the mixing tool via a hollow stem which is then mixed with the soil as the mixing tool is rotated and withdrawn. Treatment depths can be up to 50m. Single mixing tools can typically range from 0.3 to 1.5m in diameter; multiple tools can be used to form blocks. In weaker soils, larger diameters are used at shallow depths. See Figure 9.3 for a simple illustration of the process.

Stabilised soil should achieve a similar increase in stiffness to that of vibro-stone columns, albeit at a lower area ratio (see Section 9.4.2).

Stabilised columns can be installed in softer ground than stone columns. Lime alone is generally only effective in clays whereas cement can be used in sands. Lime columns are relatively permeable compared with cement columns. The potential for obstructions in the ground should be considered.

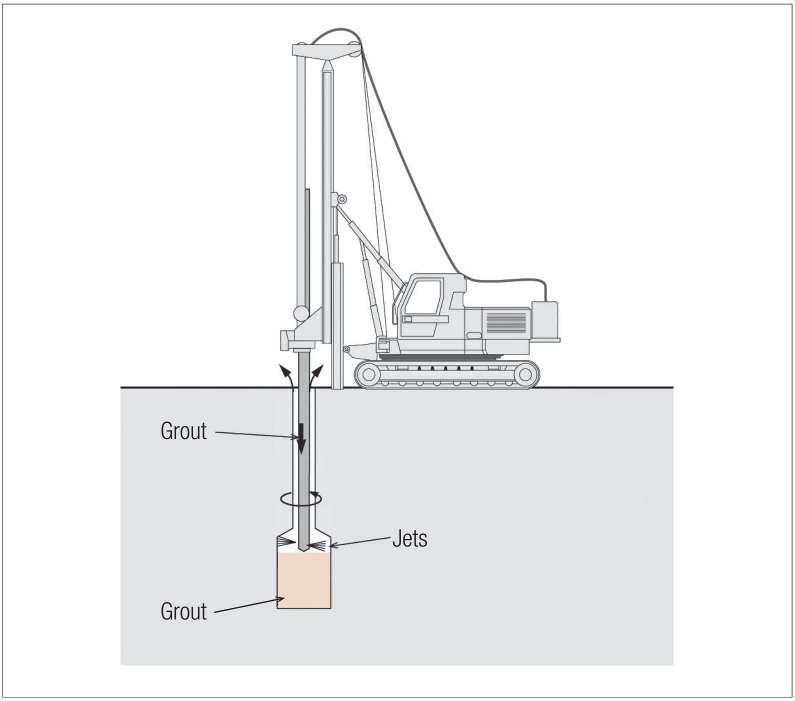


Fig 9.4 Jet grouting plant

It is noted that soil treatment with lime and cement can also be applied to pavements.

As this process adds chemicals to the ground the acceptability of the chemicals on the environment must be checked (lime is a strong alkali). Similarly the chemical composition of the ground must be assessed for its impact on the added materials (sulphates in soils may cause large heaves when stabilised with lime).

9.4.4 Jet grouting

Jet grouting comprises the construction of grout columns in the ground. The objective is to form one or more (potentially overlapping) grout columns which are generally used to provide support or control groundwater.

A rotating probe with jet holes at its tip is introduced into a narrow bore and a larger bore is excavated by using grout, air and/or water jets, to liquefy the soil and wash it to the surface as the probe is lifted. See Figure 9.4 for a simple illustration of the process.

Jet grouting is quite flexible in application to a range of soil types. However, issues to be aware of include: high grouting pressures and the associated potential for ground heave, and large amounts of spoil. Jet grouting can be carried out to depths of approximately 35m.

Quality control of the grouting operation and the end product is an important consideration.

9.4.5 Vegetation

Planting can be used in ground engineering to perform a number of functions:
– surface protection against water erosion
– surface protection against wind erosion
– surface permeability improvements in terms of reducing run-off volumes and rates
– improvement in shallow soil strength by means of root fibres.

An insight into the role of vegetation in ground engineering can be found in CIRIA Guide C708 *Use of vegetation in civil engineering*[105]. Specification of the appropriate plant types to perform specific functions is a specialist's field.

9.5 Reinforced soil structures

9.5.1 General

The image of a reinforced soil structure that typically comes to mind is of a reinforced soil wall, although reinforced soil can cover other structures such as soil nailed slopes. The common thread linking all reinforced soil structures is that there is a non-soil material which is used to support soil thereby allowing the soil to reside in a manner (load state or geometry) where it would be unstable without the reinforcement. A key issue in the design of these structures is the durability of the reinforcement providing confinement and strength addition to the soil.

Two examples of reinforced soil structures are presented below.

9.5.2 Reinforced soil walls

Reinforced soil walls typically comprise alternating layers of horizontal reinforcement such as polymer geogrid or steel straps, and compacted soil. The facing or cladding can be hard (e.g. concrete panels which may require a strip footing) or soft (e.g. geosynthetic). However, the disadvantage of geosynthetic faced walls is that they can melt/burn if the vegetation is set alight.

Together these elements form a composite retaining structure. The length of the reinforcement (i.e. distance from face to back) generally needs to be around 50% ($\pm$) of the height of the wall, though for sloping backfill or surcharges this may need to increase.

The geotechnical advantage of using reinforced soil as a retaining structure is that it can tolerate more movement than a conventional retaining structure.

BS 8006-1:2010 *Code of practice for strengthened/reinforced soils and other fills*[37] provides direction on the design and testing of such structures. In BS 8006-1 it states that EC7 does not cover the design and execution of reinforced soil structures and that the value of partial factors in EC7 (and the UK NA to EC7) have not been calibrated for reinforced soil structures. The result of this is that in the UK, EC7 should not be used to design reinforced soil structures and that BS 8006-1:2010 and BS EN 14475:2006 *Execution of special geotechnical works – Reinforced fill*[16] should be used instead.

9.5.3 Soil nailed slopes

Soil nailed slopes are typically steep soil cuttings stabilised by steel reinforcement bars. A bearing plate is attached to the bar and sometimes a facing (e.g. netting or shotcrete) is also placed (to stabilise the surface soil). Soil nailing is a passive system (i.e. the nails are not prestressed after placement) and load is only mobilised in the nails once the soil mass starts to move. Nail lengths are generally 70 to 100% of the height of the wall and placed at 1 to 2m centres.

Soil nailed slopes are constructed in a top-down manner comprising alternating excavation of a cut face and installation of nails. Nails are grouted into sub-horizontal drilled bores.

The geotechnical advantage of using soil nails is that significantly steeper soil cuttings can be formed than would be possible without the nails.

Soil nailing is addressed in BSI publication BS 8006 Part 2 *Code of practice for reinforced and strengthened earth Part 2: Soil nailing*[106] and BS EN 14490:2006 *Execution of special geotechnical works – Soil nailing*[17].

9.6 Ground anchorages

The chapter on ground anchorages in EC7 Part 1[1] is under discussion and review. Until such time as this review is completed there has been a reluctance to address the issue of ground anchorages within a Eurocode framework. Whilst it is tempting to present a non Eurocode compliant design approach to ground anchors this has been resisted. The reader may of course consider BS 8081:1989 *Code of practice for ground anchorages*[33] (non-contradictory complementary information, i.e. it remains current but EC7 takes precedence where conflicts occur) for design information. It is considered to be a useful document for the design of ground anchorages and it is recommended that the recommendations within it should be followed until superseded.

BS EN 1537:2000 *Execution of special geotechnical work – ground anchors*[9] currently provides guidance on construction and testing of ground anchorages (the document is being revised and testing requirements likely to be moved to EN 22477-5[26]). The reader is advised to monitor the publication of new and revised documents.

Detailed design of ground anchorages can be beneficially carried out by a geotechnical specialist who should, when available, use the rules and advice included in an updated version of EC7 Part 1[1].

9.7 Summary

- This chapter introduces selected specialist geotechnical activities considered relevant to typical buildings in the UK.
- Filling in terms of earthworks is a controlled engineered process.
- The advantages and disadvantages of dewatering need to be understood.
- If there is a need for ground improvement, a technique should be selected based upon its suitability.
- The status of ground anchor guidance within EC7 is currently being reviewed; an updated chapter in EC7 Part 1[1] will be issued in due course.

10 Construction, monitoring and maintenance

10.1 General

Within the EC7 approach, design and construction are linked by more than the contract drawings and specifications. It is a requirement that the ground conditions encountered during construction are assessed against those which have been documented during the design (Sections 3.7 and 3.8).

The process of tying design and construction together involves an addendum (Clause 2.8(4)P[1]) to the Geotechnical Design Report which is used to document the conditions experienced on site relative to the conditions assumed in design. Where there is a detrimental divergence then the designer must reappraise the design and update it as necessary, such that at the end of construction there is uniformity between design assumptions and construction experience. This approach can be summed up in the following requirements:
– construction processes and workmanship are supervised
– ground and groundwater conditions are checked during construction
– execution complies with relevant specifications
– performance of structures are monitored (visually and also by instrument) during construction
– people involved are competent.

A geotechnical close-out report may also be produced (not an EC7 requirement) which would augment the information in the addendum to the GDR with construction records (material test results, non-conformance report and as built drawings).

Whilst EC7 is predominantly a design code, its requirements also apply to the in-service phase of a building's life. In order that the long-term performance of a building can be provided it may be necessary to monitor parts of a building's performance as part of maintenance regime. For a typical UK building monitoring could include groundwater level behind a retaining wall with a drainage system. In more complicated designs monitoring could include settlement monitoring, e.g. of a bridge abutment (settlement monitoring during the operational life of a typical building is less likely). Hence the requirements for the in-service phase of a building are:
– performance of structures are monitored after construction against predetermined conditions
– structures are adequately maintained.

Clearly the level and quality of supervision and monitoring should be commensurate with complexity of the design/structure and level of performance required of the structure during its design life.

Accepting the approach of linking design, construction and in-service monitoring, the long-term safety and quality of a structure will depend upon:
- identification and mitigation of hazards during design
- specification of execution
- checking of design and execution documentation
- construction supervision
- monitoring of building once completed
- maintenance of the completed building.

10.2 Construction (Design and Management) Regulations, 2007

10.2.1 General

The Construction (Design and Management) Regulations (CDM) 2007[41] are not part of the Eurocode process; they are, however, part of design carried out for UK buildings. The CDM Regulations require that designers:
- are competent
- eliminate hazards and reduce risks during design
- provide information about residual risks
- check the client is aware of their duties and that a CDM co-ordinator has been appointed (notifiable projects must be registered)
- provide information needed for the health and safety file.

To accord with the CDM Regulations[41] the designer must consider and document the decision making process, as it applies to the health and safety of those involved in the construction of the building and of those involved in the maintenance of the building. The term designer is a broad term and relates to the function performed rather than the profession or job title; design functions include preparation of design drawings, specifications, bills of quantities and the specification of articles and substances. Designers include architects, engineers and quantity surveyors.

10.2.2 Duty holders

Six duty holders are identified by the CDM Regulations 2007[41]: clients, CDM co-ordinators, designers, principal contractors, contractors and finally workers. Each duty holder has different roles and responsibilities.

A summary of the requirements for designers was presented in Section 10.2.1. However, designers should be careful as they may fall under one of the other duty holder categories. For example, a designer may become a contractor if they instruct work on a site.

10.2.3 Discharge of responsibilities

Designers can effectively fulfil much of their responsibility under the regulations by maintaining and communicating a risk register and by implementing the risk register mitigation measures in their design.

Further information on the CDM Regulations 2007[41] can be obtained from the Health and Safety Executive[107].

10.3 Specification

10.3.1 Construction

Contract documentation for construction works is beyond the scope of this *Manual*. Hence the comments provided below do not give a detailed description of specification for construction works. However, references to sources of information that may be used are presented.

Comments on execution standards and technical specifications for construction have been made in previous chapters for key geotechnical structures.

For piles and retaining walls a primary reference document is the *ICE Specification for Piling and Embedded Retaining Walls*[38].

For earthworks the *Specification for Highway Works*[108] provides a useful source of information. The following sections are of particular importance to geotechnical works:
- Series 200 Site Clearance.
- Series 500 Drainage and Service Ducts.
- Series 600 Earthworks.
- Series 1600 Piling and Embedded Retaining Walls (closely related to the ICE Specification referenced above).

Reference to the National Building Specification documents can also be made for generic specifications. For example, Work Section D[109] deals with groundwork.

10.3.2 Construction supervision, maintenance and monitoring

The specification of construction supervision, maintenance and monitoring forms part of the Geotechnical Design Report (GDR) (see Section 3.8).

10.4 Construction supervision

10.4.1 General

Supervision of construction (process and workmanship) comprises:
– Checking that design assumptions are valid.
– Checking that construction conforms to design.

Observations and measurements of behaviour (see Section 10.6 below) are undertaken during construction to identify the need for changes or remedial works. Where appropriate, observations and measurements should continue beyond the construction phase in order to evaluate long-term performance.

10.4.2 Inspection and quality control

Inspection (a component of supervision) of the works (temporary and permanent) should be carried out. This can best be done by the designer or their representative as they are most aware of the design and assumptions made therein. Inspection should be on a 'continuous' basis. The requirement 'continuous' does not imply that full time inspection is required but that the required level of inspection is maintained throughout the construction works. Whilst inspection is development specific, an initial scope can be based on the Geotechnical Category (see Chapter 2):
– GC1: basic inspection, basic quality control, qualitative assessment.
– GC2: more comprehensive inspection, quality control and quantitative assessment.

The results of inspections are to be recorded, maintained and contain, as appropriate:
– Ground and groundwater conditions (see Section 10.5).
– Environmental conditions (e.g. contamination and archaeology).
– Monitoring/measurement/observation data.
– Material test results and occurrences of non-compliance.
– Modifications to the design to accommodate variations in site conditions from that assumed in the design.
– Modifications to the design resulting from reaction to monitoring data.
– As-built data and drawings.
– Progress and sequence of works.
– Unforeseen events.
– Indirect evidence of geotechnical properties of the ground.

Observed differences between design assumptions and site operations are to be reported to the designer without delay.

The requirement to inspect works has a clear benefit in terms of quality of construction. It also has legal implications in the case of accidents (not addressed in EC7). Designers have an obligation to specify the required inspection level that is appropriate to the construction being carried out (the design being either permanent or temporary works as appropriate). The designer should advise the client of the requirement for appropriate professional inspection of construction works; it is then the client's responsibility to see that appropriate inspection is catered for. Whilst it is beneficial for the designer to continue in an inspection role it is not necessary in the context of the client discharging their duties.

Liability for inadequate inspection typically falls under contract to the client, in tort to third parties or under the Health and Safety at Work Act[110] (and CDM Regulations[41]). When planning inspection, designers should consider issues such as, how a court might view the situation should a case arise, the method of procurement for the works and the agreement they have with their client.

10.4.3 Review

Reviewing supervision records is necessary on an on-going basis and comprises:
- Assessment of the suitability of construction procedures and sequencing in light of actual ground and groundwater conditions.
- Comparison of predicted and observed behaviour of structures.

Where modifications to the design presented in the GDR are required (in light of review of supervision and monitoring records), these are to be agreed with the designer prior to implementation and clearly documented. The designer is to explicitly and rationally consider any changes.

Once all the required construction checks have been completed they are recorded in an addendum to the GDR.

10.4.4 Particular considerations for piles

Piling works are to be based on a pile installation plan including at least the following information: pile number, type, location, inclination, tolerances, cross-section, reinforcement, length, load carrying capacity (compression, tension and horizontal), toe level, installation sequencing if applicable, obstructions on site, and other constraints. See also Section 7.17 for references.

Monitoring and recording of the installation of all piles is required and is to be specified accordingly. Pile records should include those aspects of construction dealt with in the execution standards (see Section 7.17). If uncertainties are revealed with respect to pile quality, appropriate investigation is to be undertaken and the need for remedial works identified.

Integrity testing of an appropriate nature is to be undertaken when quality is sensitive to installation procedures which cannot be reliably monitored during pile installation.

10.4.5 Particular considerations for spread foundations

The subsoil (i.e. formation) is to be prepared with care so as to minimise disturbance at and below the formation level. Roots, deleterious matter or weak ground should be carefully removed and the disturbed ground or voids reinstated to a stiffness equivalent to or better than the undisturbed ground.

10.5 Control investigation

This section addresses, in most cases, the final stage of ground investigation for a project. Earlier stages of ground investigation, namely preliminary and design ground investigation, are discussed in Chapter 3. Control ground investigations generally form part of the scope of construction supervision.

The ground and groundwater conditions are to be checked/investigated during construction and compared with the results of the preliminary and design ground investigations and the design assumptions. Deviations from design assumptions are to be reported (normally to the designer) without delay. The design shall then be checked and modified as needed and the GDR updated to reflect the changes.

The scope of control investigations is to be appropriate to the complexity and scale of the project. In this respect, the Geotechnical Category is helpful in guiding the necessary level of checking/investigation to mitigate geotechnical risks.

For GC1:
- inspecting the site at the beginning and during the works
- identifying the nature of the ground influenced by the structure
- recording the nature of the ground and groundwater in excavations
- checking groundwater conditions.

For GC2 (in addition to those for GC1):
- additional intrusive investigation and testing may be required (e.g. if full ground investigation was not possible prior to demolition of existing structures and there is a need for confirmatory works above those that can be carried out a part of general construction)
- verification testing should be carried out on improved ground (e.g. fill, grouted ground, ground improved with stone columns, etc.)
- direct observation of groundwater conditions if important (e.g. excavations below water level or the design for structure which may be buoyant until super-structure loading is in place).

In all cases checks, both visual and olfactory, should be made for unexpected contamination. Where unexpected conditions are encountered samples of soil and groundwater should be taken for testing to check the implications for material durability (sulphate, pH, chloride etc.), safety of site operatives/third parties and disposal of material off-site (waste acceptance criteria).

Working in confined spaces should include the appropriate level of gas monitoring, ventilation and safety equipment as well as safe access and egress.

10.6 Monitoring

It is required that monitoring be carried out in accordance with the assumptions and requirements set out in the GDR to:
– check that prediction of behaviour (e.g. movement or changes in groundwater level) made during design are appropriate
– ensure that the construction will continue to perform as required after completion
– control construction processes.

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

Monitoring includes measurement of:
– movements (lateral and vertical) of ground affected by the construction works
– movement (lateral, vertical and shear movement) of adjacent structures affected by the construction works
– movement (usually lateral and vertical movements) of the construction work
– structural forces and displacements (e.g. thermal effects in temporary steel props)
– pumped discharge from dewatering wells/sumps (checks for quality with respect to any discharge licences and that no fines are being removed)
– pore-water pressure, groundwater level and groundwater flow, including variations around the site
– contact pressure between structure and ground
– values of actions.

As appropriate, monitoring also includes visual inspection.

At the commencement of monitoring baseline conditions need to be recorded. The duration required to achieve a full set of baseline readings may be weeks for structures in a stable environment or up to a year for structures which are influenced by seasonal effects. Assessment of cyclic daily effects should also be considered when forming baseline readings (e.g. the effect of sun on building facades).

The results of monitoring/measurement are to be evaluated and interpreted in a quantitative manner and should be linked to the phase of the construction

works, to qualitative observations of site activities and the appearance (signs of distortion/cracking etc.) of relevant structures. The scope of monitoring can be related to the Geotechnical Category:
– For GC1, monitoring may be qualitative visual inspection.
– For GC2, monitoring may comprise measurement of ground and structural movements with quantitative analysis.

Monitoring programmes will need to take account of structures with the potential to adversely impact on ground and groundwater conditions. Structures in this category could include those that retain water/ground, those that control seepage, slopes and ground improvement. The effect of construction activity on the groundwater regime is also to be checked.

The duration of post-construction monitoring recommended in the GDR should not be considered to be an absolute requirement. The monitoring regime should be reviewed at appropriate stages to assess the correctness of scope, frequency and indeed nature of monitoring. Prior to termination of the monitoring regime, or part thereof, a conscious decision should be made that the equilibrium conditions have been reached beyond which movements are not expected.

10.7 Maintenance

The maintenance requirements for a new structure should be recorded in the GDR and should be brought to the attention of the owner/tenant (and future owners/tenants). Maintenance is related to the safety and serviceability of the structure and also to instances where the structure has an impact on adjacent structures (e.g. where there is a risk of groundwater flow around the new building causing a rise in groundwater level at adjacent buildings).

10.8 Demolition

Where possible consideration of demolition and decommissioning of buildings should be included in the design and transferred to the building's operation manual.

10.9 Summary

– In geotechnical engineering, design drawings and specifications should not be seen as the finished article but as a clear statement of the design intent that needs to be taken to the construction phase of the project. At construction the combination of control ground investigations and monitoring of execution will confirm, or otherwise, that the design as shown in construction information is correct or if modifications are required to enable the design intent to be realised. This approach is in contrast to other parts of building construction where the design is assumed to be the finished article.

– Over and beyond the requirements of design and construction, engineers and construction professionals have obligations under the Construction Design and Management Regulations (2007)[41]. These requirements exist to protect construction workers, third parties and those involved in the maintenance of completed structures. Under the regulations the designer is responsible for ensuring that owners and clients are aware of their obligations of the act. Properly enacted risk registers are seen to be a suitable tool for managing risk and for conveying residual risk to subsequent parties who may be impacted by these risks (e.g. contractors, owners and maintenance staff).

– Monitoring is often a requirement of construction activity. Monitoring is equally applicable to existing structures which may be impacted by the proposed construction and to the construction itself. For structures or conditions which are impacted by the proposed works a period of monitoring is required to establish baseline conditions prior to the commencement of the works. The period of baseline monitoring may be relatively short for relatively insensitive structures but relatively long for sensitive structure or structures whose movement responds to other external influences (e.g. seasonal variations for earthworks).

Notation

Latin upper case letters

A'	effective base area ($A' = B' \times L'$)
A_b	base area under pile
A_c	total base area under compression
$A_{s;i}$	pile shaft surface area in layer i
A_d	design value of an accidental action
B	width of a foundation
B	pile diameter
B'	effective width of a foundation
C_d	limiting design value of the relevant serviceability criterion
D	pile depth
E	Young's modulus
E_d	design value of the effect of actions
$E_{stb;d}$	design value of the effect of stabilising actions
$E_{dst;d}$	design value of the effect of destabilising actions
$E_{v;d}$	design value of vertical Young's modulus
$F_{c;d}$	design axial compression load on a pile or a group of piles
F_d	design value of an action
F_k	characteristic value of an action
F_{rep}	representative value of an action
$F_{t;d}$	design axial tensile load on a tensile pile or a group of tensile piles
$F_{tr;d}$	design value of the transverse load on a pile or a pile foundation
$G_{dst;d}$	design value of the destabilising permanent actions for uplift verification
$G_{stb;d}$	design value of the stabilising permanent vertical actions for uplift verification
$G'_{stb;d}$	design value of the stabilising permanent vertical actions for heave verification (submerged weight)
H	horizontal load, or component of total action acting parallel to the foundation base
H_d	design value of H
I	second moment of area
K_0	coefficient of earth pressure at rest
$K_{0;\beta}$	coefficient of earth pressure at rest for a retained earth surface inclined at angle β to the horizontal
K_a	coefficient of active earth pressure
K_{ac}	active earth pressure coefficient to model soil cohesion
K_p	coefficient of passive earth pressure
K_{pc}	passive earth pressure coefficient to model soil cohesion
K_s	factor linking horizontal to vertical effective stresses
L	foundation length
L'	effective foundation length
N_q	bearing capacity factor based on characteristic value of φ'

P	load on an anchorage
P	representative value of a prestressing action
P_d	design value of P
P_p	proof load in a suitability test of a grouted anchorage
$Q_{dst;d}$	design value of the destabilising variable vertical actions for uplift verification
Q_k	leading value of a variable action
R_a	anchorage pull-out resistance
$R_{a;d}$	design value of R_a
$R_{a;k}$	characteristic value of R_a
$R_{b;cal}$	pile base resistance, calculated from ground test results, at the ultimate limit state
$R_{b;d}$	design value of the base resistance of a pile
$R_{b;k}$	characteristic value of the base resistance of a pile
R_c	compressive resistance of the ground against a pile, at the ultimate limit state
$R_{c;cal}$	calculated value of R_c
$R_{c;d}$	design value of R_c
$R_{c;k}$	characteristic value of R_c
$R_{c;m}$	measured value of R_c in one or several pile load tests
R_d	design value of the resistance to an action
R_k	characteristic value of a material or product property
$R_{p;d}$	design value of the resisting force caused by earth pressure on the side of a foundation
$R_{s;cal}$	ultimate shaft friction, calculated using ground parameters from test results
$R_{s;d}$	design value of the shaft resistance of a pile
$R_{s;k}$	characteristic value of the shaft resistance of a pile
R_t	ultimate tensile resistance of an isolated pile
$R_{t;d}$	design value of the tensile resistance of a pile or of a group of piles, or of the structural tensile resistance of an anchorage
$R_{t;k}$	characteristic value of the tensile resistance of a pile or a pile group
$R_{t;m}$	measured tensile resistance of an isolated pile in one or several pile load tests
R_{tr}	resistance of a pile to transverse loads
$R_{tr;d}$	design resistance of transversally loaded pile
$S_{dst;d}$	design value of the destabilising seepage force in the ground
$S_{dst;k}$	characteristic value of the destabilising seepage force in the ground
T_d	design value of total shearing resistance that develops around a block of ground in which a group of tension piles is placed, or on the part of the structure in contact with the ground
V	vertical load, or component of the total action acting normal to the foundation base
V_d	design value of V
V'_d	design value of the effective vertical action or component of the total action acting normal to the foundation base
$V_{dst;d}$	design value of the destabilising vertical action on a structure
$V_{dst;k}$	characteristic value of the destabilising vertical action on a structure
X_d	design value of a material property
X_k	characteristic value of a material property

Latin lower case letters

a_d	design value of geometrical data
a_k	characteristic value of a geometrical property
a_{nom}	nominal value of geometrical data
b'	effective width of a foundation
c	cohesion intercept
c'	cohesion intercept in terms of effective stress
c_u	undrained shear strength
$c_{u;d}$	design value of undrained shear strength
d	embedment depth
d_s	discontinuity spacing
h	height of a wall
h	water level for hydraulic heave
h'	height of a soil prism for verifying hydraulic heave
$h_{w;k}$	characteristic value of the hydrostatic water head at the bottom of a soil prism
k	ratio $\delta_d/\varphi_{cv;d}$
n	number of, e.g. piles or test profiles
q	surface surcharge
q_a	allowable or presumed bearing resistance
$q_{b;k}$	characteristic value of unit base resistance
$q_{s;i;k}$	characteristic value of unit shaft friction in stratum i
q_u	unconfined compressive strength
s	settlement
s_0	immediate settlement
s_1	settlement caused by consolidation
s_2	settlement caused by creep (secondary settlement)
s_t	total settlement
u	pore-water pressure
$u_{dst;d}$	design value of destabilising total pore-water pressure
u_f	pore-water pressure at toe of retaining wall
z	vertical distance
z	depth

Greek upper case letters

Δa	change made to nominal geometrical data for particular design purposes

Greek lower case letters

α	inclination of a foundation base to the horizontal
α	shaft adhesion factor for pile design in clays
α	factor linking undrained shear strength to shaft friction
β	slope angle of the ground behind a wall (upward positive)
β_B	reduction factor on section modulus
β_D	reduction factor on second moment of area
γ	weight density
γ	unit weight of ground
γ'	effective weight density

γ_b	partial factor for the base resistance of a pile
$\gamma_{c'}$	partial factor for the effective cohesion
γ_{cu}	partial factor for the undrained shear strength
γ_E	partial factor for the effect of an action
γ_f	partial factor for actions, which takes account of the possibility of unfavourable deviations of the action values from the representative values
γ_F	partial factor for an action
γ_G	partial factor for a permanent action
$\gamma_{G;dst}$	partial factor for a permanent destabilising (unfavourable) action
$\gamma_{G;inf}$	partial factor for permanent action in calculating lower (favourable) design value
$\gamma_{G;stb}$	partial factor for a permanent stabilising (favourable) action
$\gamma_{G;sup}$	partial factor for permanent action in calculating upper (unfavourable) design value
γ_{Gset}	partial factor for uneven settlement
γ_m	partial factor for a soil parameter (material property)
$\gamma_{m;i}$	partial factor for a soil parameter in stratum *i*
γ_M	partial factor for a soil parameter (material property), also accounting for model uncertainties
γ_P	partial factor for prestressing actions
γ_Q	partial factor for a variable action
γ_{qu}	partial factor for unconfined strength
γ_R	partial factor for a resistance
$\gamma_{R;d}$	partial factor for uncertainty in a resistance model
$\gamma_{R;e}$	partial factor for passive earth resistance
$\gamma_{R;h}$	partial factor for sliding resistance
$\gamma_{R;v}$	partial factor for bearing resistance
γ_s	partial factor for shaft resistance of a pile
$\gamma_{S;d}$	partial factor for uncertainties in modelling the effects of actions
$\gamma_{Q;dst}$	partial factor for a variable destabilising action
$\gamma_{Q;stb}$	partial factor for a variable stabilising action
$\gamma_{s;t}$	partial factor for tensile resistance of a pile
γ_t	partial factor for total resistance of a pile
γ_w	weight density of water
γ'_φ	partial factor for the angle of shearing resistance (tan φ')
γ_γ	partial factor for weight density
δ	structure-ground interface friction angle
δ_d	design value of δ
θ	direction angle of *H*
ξ	correlation factor depending on the number of piles tested or of profiles of tests
ξ	reduction factor (see Section 2.11.3.2)
ξ_a	correlation factor for anchorages
ξ_1; ξ_2	correlation factors to evaluate the results of static pile load tests
ξ_3; ξ_4	correlation factors to derive the pile resistance from ground investigation results, not being pile load tests
ξ_5; ξ_6	correlation factors to derive the pile resistance from dynamic impact tests
$\sigma_{cl;k}$	characteristic unconfined compression strength of rock
$\sigma_{stb;d}$	design value of stabilising total vertical stress
$\sigma'_{h;0}$	horizontal component of effective earth pressure at rest
$\sigma(z)$	stress normal to a wall at depth *z*

$\tau(z)$	stress tangential to a wall at depth z
φ'	angle of shearing resistance in terms of effective stress
φ_{cv}	critical state angle of shearing resistance
$\varphi_{cv;d}$	design value of φ_{cv}
φ'_d	design value of φ'
ψ	factor for converting the characteristic value to the representative value
ψ_0	characteristic combination factor
ψ_1	frequent combination factor
ψ_2	quasi-permanent combination factor

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Appendix A Selection of characteristic parameters

Introduction

This design example builds on the examples presented in Chapter 4. It presents the approach that may be taken when assessing characteristic ground parameters using engineering judgement to arrive at a cautious estimate of the ground parameter which controls the limit state. The example looks at ground parameters for a preliminary retaining wall design and considers only the short term conditions prior to the permanent structure and propping being built; it focuses on soil parameters and not wall design.

Task

Two boreholes have been obtained as part of the desk study carried out for a site. The information included in these boreholes will be used for assessing the preliminary design of an embedded retaining wall which will support an excavation. The excavation is planned to have a maximum depth of 6.5m. The typical ground level is +82mOD at the location of the proposed excavation; the site is on a gentle slope (the boreholes are uphill and downhill of the planned excavation). Following installation and an initial dig to 1m depth the wall will be propped at the top allowing the dig to be completed to the final level. Thereafter the permanent propping will be installed. On one side of the excavation a highway exists, on the other three sides the area is parkland. Preliminary considerations suggest that a bored pile wall or a sheet pile wall may be appropriate.

The ground conditions are presented in Figures A.1 and A.2 for boreholes 1 and 2 respectively.

GI data

The information relating to the geological succession contained in the borehole logs is summarised in Table A.1.

The borehole logs have SPT (Standard Penetration Tests) data contained within them; no laboratory test data is available alongside the archive borehole logs. The SPT data is plotted against depth in Figure A.3; the SPT *N* values have been extrapolated to a full penetration of 300mm when refusal was met prior to achieving 300mm penetration.

Geotechnical category (see Section 2.9)

The Geotechnical Category is GC2.

Borehole Log	Job No. 210365	Hole ref **Borehole 1**	Page 1 of 1
	Contractor		
Boreholes for EC7 manual	NG co-ordinates	Ground Level (mOD) 80.34	Date

Samples & tests

Depth	Sample Type Ref	Test Result	Water
8.10		4,8,9,8,18,15	
10.10		3,9,9,8,10,9	
13.60		8,9,14,9,8,10	
16.40		25,R,50 for 40	
18.00		10,10,70 for 75	
19.50		7,10,12,70 for 75	
21.00		8,12,11,80 for 75	
22.50		12,19,100 for 75	

Strata log

Red. Level	Legend	Depth (Thickness)	Description	Geology	Instrument/ Backfill
79.8		(0.50) 0.50	TOPSOIL		
79.6		(0.20) 0.70	Firm brown mottled CLAY		
78.6		(1.00) 1.70	Firm greyish brown mottled CLAY with occasional cobbles (Weathered Glacial Till)		
77.0		(1.60) 3.30	Stiff grey boulder CLAY with occasional cobbles (Glacial Till)		
72.3		(4.70) 8.00	Very stiff grey boulder CLAY with occasional cobbles and much mudstone gravel (Glacial Till)		
63.7		(8.60) 16.60	Very stiff grey slightly sandy gravelly CLAY with sub angular to sub rounded sandstone and mudstone cobbles. Gravel is angular to sub rounded fine to coarse of sandstone. Sand is fine to coarse. (Glacial Till)		
59.0		(4.75) 21.35	Very stiff grey and white mottled slightly gravelly sandy CLAY. Gravel is sub angular and sub rounded fine to coarse of sandstone and mudstone. Sand is fine to coarse. (Driller notes boulder)		
56.6		(2.35) 23.70	Boulder CLAY with large boulders		
53.3		(3.30) 27.00	Medium strong thinly laminated to medium bedded light grey fine grained. SANDSTONE .		
			Borehole completed at 27m depth		

Client	Logged by:	Database check:

Fig A.1 Borehole 1

Borehole Log	Job No. 210365	Hole ref **Borehole 2**	Page 1 of 1
	Contractor		
Boreholes for EC7 manual	NG co-ordinates	Ground Level (mOD) 84.16	Date

Samples & tests

Depth	Sample Type Ref	Test Result	Water
3.10		6,8,10,8,12,18	
5.10		4,7,9,9,10,9	
7.10		5,8,10,8,9,13	
9.40		10,12,8,9,10,9	
11.05		20,5R,50 for 70	
13.00		15,13,16,19,23,27	
15.00		17,17,18,21,24,26	
17.00		12,R,50 for 75	
19.00		16,18,18,21,24,24	
21.00		17,19,21,24,23,26	
23.00		19,22,22,24,26,27	
25.00		30,37,40,42 for 75	
27.00		17,19,22,24,26,26	
29.00		17,17,19,19,20,22	

Strata log

Red. Level	Legend	Depth (Thickness)	Description	Geology	Instrument/ Backfill
83.8		(0.40) 0.40	TOPSOIL		
82.1		(1.70) 2.10	Soft brown and orange mottled slightly gravelly (Weathered Glacial Till)		
73.0		(9.10) 11.20	Very stiff grey slightly gravelly sandy CLAY with occasional sub rounded dolerite cobbles. Gravel is sub rounded to sub angular fine to coarse of mudstone and sandstone. Sand is fine to coarse (Glacial Till)		
72.2		(0.80) 12.00	Boulder CLAY		
54.7		(17.50) 29.50	Very stiff dark brown slightly sandy slightly gravelly CLAY. Gravel is sub rounded to sub angular fine to coarse of sandstone and mudstone. Sand is fine to coarse (Glacial Till).		
			Borehole completed at 29.5m depth		

Client	Logged by:	Database check:

Fig A.2 Borehole 2

Table A.1 Summary ground strata and parameters – characteristic parameters example

Stratum	Borehole records		Levels used for preliminary design[a]
	Top level (mOD)	Thickness (m)	
Topsoil	+80.3 to +84	0.4 to 0.5	+82
Weathered Glacial Till	+79.8 to +83.8	1.2 to 1.7	+81.5
Glacial Till	+78.6 to +82.1	21m or more	+80
Sandstone	+56.6 where encountered	Not proven	+56
Groundwater	No records		Assumed at 1m depth, need to monitor as part of construction

Note
a All to be confirmed by site specific ground investigation.

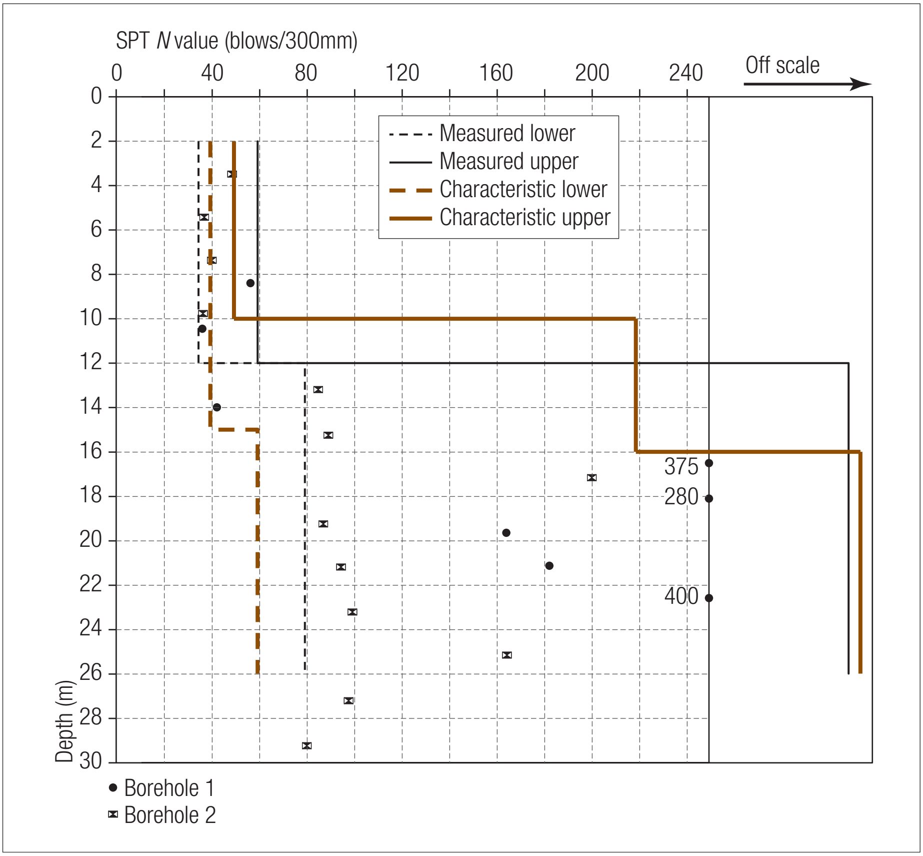

Fig A.3 SPT results

Limit states (see Section 2.3)
- Ultimate limit state: formation of a wall failure mechanism (rotation or wedge failure surface).
- Serviceability limit state: acceptable deflection of the wall less than 1 in 200.

Design situation (see Section 2.5)
- Persistent (normal conditions).
- Variable surcharge loading in park area ($Q_k = 5kN/m^2$).
- Variable surcharge on crest of slope.
- Acceptable retaining wall deflection for aesthetic considerations.
- Corrosion of sheet piles if adopted.
- Variation in ground conditions.

Design considerations
- Available land for construction of required slope.
- Surface degradation post completion of slope.

Design life (see Section 2.7)
- 50 years.

Assessment of characteristic geotechnical parameters:
The driller's description of the strata encountered are included in the borehole logs. In keeping with a glacial till there are records of cobbles and boulders. Such large size particles will not be recovered in the borehole arisings and as such the driller's description, along with a geological understanding of the strata, is important when interpreting the ground conditions and the *in situ* test data such as the SPT tests. For example, in borehole 2 at 11.2m depth a 0.8m thick layer of boulder clay is logged as being present suggesting the presence of boulder(s) as seen in borehole 1 at 21.35m depth. This observation is strengthened by a high SPT blow count (50 blows for 70mm penetration giving an extrapolated $N = 50 \times 300/70 = 214$); even without the SPT record the description should be cause for consideration when characterising the ground. The presence of boulders is possible at other levels within the stratum even though not encountered in the boreholes above a depth of 11m.

The SPT test results are presented in the borehole logs (see Figures A.1 and A.2) and in Figure A.3. It can be seen, especially in borehole 1, that on many occasions the SPT test has met an abrupt refusal after less than 300mm total penetration. Such results suggest a material that is variable and which may have softer zones (clay matrix) intermingled with harder zones (gravel, cobbles or boulders). Such observations should be remembered when addressing characteristic values.

It can be seen that there is a significant variation in the data with relatively low values ($N = 35$ to 40) in the upper 10 to 14m. At depths greater than 10 to 14m there is wider scatter with many SPT results showing values in excess of $N = 200$ but with a reasonable number of results (especially from borehole 2)

Table A.2 Range of SPT blow counts with depth

Stratum	Depth to top of layer (m)	Measured SPT N values	
		Lower bound	Upper bound
Topsoil	0	N/A	N/A
Weathered Glacial Till	0.5	Described as soft and firm clay	
Glacial Till	2	35	60 (+risk of boulders)
	12	80	400
Sandstone	24	Described as medium strong	

having SPT N values approximately equal to 90. Table A.2 presents the range of values that may be considered typical based on the borehole data.

The SPT blow count N is not a 'geotechnical' parameter that is used to carry out a geotechnical design albeit it is used in prescriptive methods. It can however be used to assess geotechnical parameters through empirical relationships. For the temporary works design of a retaining wall (i.e. the period for which undrained strength conditions may apply) the SPT blow count can be used to assess:

c_u for assessment of limiting active and passive pressures in the short term.
E_u for use in assessment of short-term wall deflection, bending moment and shear force.

Other parameters will also be required (e.g. *in situ* K_0) and thereafter for permanent conditions drained soil strength and stiffness parameters.

Stroud[111] presents a correlation between N and c_u for clays. A value of $f_1 = 4.5$ (in $c_u = f_1 N$, where c_u is in kN/m^2) is suggested as being appropriate for assessment of c_u values. Ratios between stiffness and N also exist with a typical value being in the range of $E_u = (400 \text{ to } 1000)c_u$ depending on material and likely wall deflection.

Characteristic value for installation:
Accepting that the design of the retaining wall must consider both installation requirements, and thereafter ULS and SLS design criteria, it is necessary to choose characteristic values of parameters which address these design situations. For installation of a retaining wall it is suggested that strong, dense or very coarse grained soils are problematic in sheet pile driving and to a lesser extent bored pile installation. Hence for installation the characteristic values (the characteristic value being a cautious estimate of the parameter controlling the limit state) in Table A.3 are proposed for preliminary considerations.

If the wall installation type is sensitive to values outside the range posed then it is recommended that the design ground investigation pays close attention

Table A.3 Characteristic SPT *N* value for wall installation

Stratum	Depth to top of layer (m)	Characteristic SPT *N* values for wall installation
Topsoil	0	Consider for piling mat design
Weathered Glacial Till	0.5	Consider for piling mat design
Glacial Till	2	50
	10	220 (The presence of inclusions will need to be considered)
	16	400 (The presence of inclusions will need to be considered)
Sandstone	24+	Below depth of wall

to the presence of cobbles/boulders and that consideration of different wall types be made.

Characteristic value for SLS and ULS design:
In contrast to the process of wall installation, when considering stability of the wall a weak soil is generally more onerous than a strong soil (larger wall deflections, bending moment shear forces etc.). Hence the characteristic value in this situation must consider how the soil varies towards the lower range of values that have been assessed. Table A.4 shows the characteristic

Table A.4 Characteristic values recommended for use in development of engineering design parameters for preliminary design

Stratum	Depth to top of layer (m)	Characteristic SPT *N* values for wall stability design	c_u (kN/m^2)
Topsoil	0	Assume low strength in design	N/A
Weathered Glacial Till	0.5	Adopt value in keeping with driller's logs (soft clay)	30
Glacial Till	2	40	180 (in keeping with driller's description as 'very stiff')
	15	60 (This value is based on a cautious estimate of the SPT blow counts with bias paid to the result that are representative of the clay matrix and not gravel, cobbles etc.)[a]	270 (in keeping with driller's description as 'very stiff')
Sandstone	24+	Below depth of wall	N/A

Note
a This assessment looks at the individual blows per 75mm and sees that a cautious estimate of the values that are not influenced by coarse layers (where refusal is often met) is less than the calculated *N* values plotted.

values recommended for use in development of design parameters for preliminary design (not installation assessment) during the excavation process; long-term parameters will also need to be assessed.

The process of taking the characteristic value of N and not first calculating undrained shear strength of the clay is not strictly in keeping with EC7. However, given that the high values of N are likely not due to clay but inclusions it is necessary in this case to deal in values of N and not c_u to maintain clarity in the assessment.

Appendix B Overall stability example

Introduction

This design example presents the process for the assessment of a safe slope angle for the sides of an excavation. It demonstrates how partial factors are applied to material strengths and actions within geotechnical design.

Task

Design the side slopes of an open cut excavation (see Figure B.1) that must be stable in the long-term. The ground at the top of the slope will be subjected to variable surcharge loading. The following is a summary of the conditions:

– Characteristic permanent (dead) load on crest of slope: $G_k = 0\text{kN/m}^2$
– Characteristic variable (live) load on crest of slope: $Q_k = 30\text{kN/m}^2$
– All surcharges are considered unfavourable.
– Crest of slope is at a level of +15mOD.
– Maximum excavation depth is to a level of +10mOD, i.e. a 5m slope height.

What is the maximum (design) slope angle β that can be accommodated?

GI data

A project specific ground investigation has been carried out returning the ground stratigraphy and parameters show in Table B.1.

The characteristic value of c' must consider that it is likely to vary more than the strength realised from tan φ'; the chosen value should take account of this.

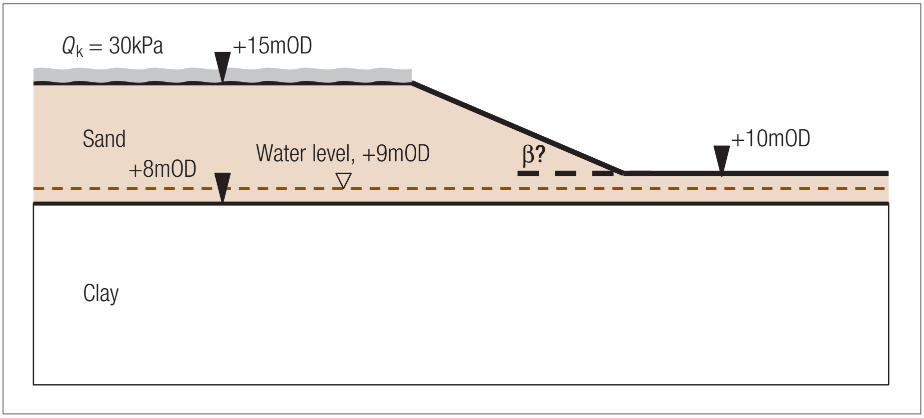

Fig B.1 Design problem

Table B.1 Summary ground strata and parameters – overall stability example

Stratum	Top level (mOD)	Bulk density (kN/m^3)	φ'_k (°)	c'_k (kN/m^2)
Groundwater	+9	10	–	–
Sand	+15	19 above water 20 below water	34	0
Stiff clay	+8	20	22	5
Intact rock	+0	Assumed rigid base	–	–

Geotechnical category (see Section 2.9)
– The Geotechnical Category is GC2.

Limit states (see Section 2.3)
– Ultimate limit state: formation of a failure mechanism (rotation or wedge failure surface).

Design situation (see Section 2.5)
– Persistent (normal conditions).
– Variable surcharge loading.
– Ground movements and effects on adjacent structures (not checked herein).
– Variation in ground conditions.

Design considerations
– Available land for construction of required slope.
– Surface degradation post completion of slope.

Design life (see Section 2.7)
– 50 years for earthworks slope.
– Maintenance (drainage) requirements must be stated in the Geotechnical Design Report and be included in the operational and maintenance manual.

Detailed design of slope
The design requires a long-term stable slope to be constructed.

The construction of the slope should not result in ground movement which impacts on adjacent structures (not addressed herein).

Section 5.3.2 presents the requirements for assessment of Overall Stability – Group 1 geotechnical collapse.

Load combinations considered:
By observation Design Approach 1 Combination 2 will be critical when compared to Design Approach Combination 1.

Design Approach 1 Combination 2 uses the following set of factors:

A2 + M2 (+R1) (see Section 2.11.3.4)

where:
A2 factors are as per Table 2.7a
M2 factors are as per Table 2.7b.

Calculation of design action F_d:
The calculation of F_d is given in Table B.2.

Table B.2 Calculation of F_d – overall stability example

Set A2:	$G_k = 0\text{kN/m}^2$, $\gamma_G = 1.0$ $Q_k = 30\text{kN/m}^2$, $\gamma_Q = 1.3$ for leading (only) unfavourable action Material densities are treated as permanent actions and are unaltered for A2 set of partial factors ($\gamma_G = 1.0$ on all permanent actions).	$G_d = 0\text{kN/m}^2$ $Q_d = 39\text{kN/m}^2$
	$F_d = (\gamma_G\, G_k) + (\gamma_Q\, Q_k) = (1.0 \times 0) + (1.3 \times 30) = 0 + 39$	$F_d = 39\text{kN/m}^2$

Calculation of design material strengths:
The calculation of design material strengths is given in Table B.3.

Table B.3 Calculation of design material strengths

Set M2:	**Gravel** $\varphi'_k = 34°$ $\gamma'_\varphi = 1.25$ $\varphi'_d = \text{atan}((\tan \varphi'_k)/\gamma'_\varphi)$ $\varphi'_d = \text{atan}((\tan 34)/1.25)$ ***Note*** $c'_k = 0$ and need not be considered further ($\gamma'_c = 1.25$)	$\varphi'_d = 28.4°$ $c'_d = 0\text{kN/m}^2$
	Stiff clay $\varphi'_k = 22°$ $\gamma'_\varphi = 1.25$ $\varphi'_d = \text{atan}((\tan \varphi'_k)/\gamma'_\varnothing)$ $\varphi'_d = \text{atan}((\tan 22)/1.25)$ $c'_k = 5\text{kN/m}^2$ $\gamma'_c = 1.25$ $c'_d = c'_k/\gamma'_c$ $c'_d = 5/1.25$	$\varphi'_d = 17.9°$ $c'_d = 4\text{kN/m}^2$

Calculation of limiting slope angle:
Using the geotechnical data in Table B.3 and the design surcharge loading F_d in Table B.2, a slope stability analysis was carried out using proprietary computer software. The computer software used in this assessment was Oasys Slope and the method of analysis used was Bishop[63] with inclined inter-slice forces.

The calculations have been carried out with design surcharge loading (Set A2) and design material strengths (Set M2) thereby requiring only that the utilisation ratio U is less than or equal to 1.0 (where $U = E_d/R_d \leq 1.0$) thereby satisfying the requirement:

$E_d \leq R_d$

Figure B.2 shows the output from the slope stability calculation. A slope with 5m height and 9m horizontal length (equivalent to an angle of 29°) is seen to satisfy the limiting material strength criterion as shown in Figure B.2 where the FoS based on DA1 C2 design actions and strengths is greater than 1.0 (i.e. $E_d < R_d$). It is also possible to express this result as a 'utilisation ratio'; the utilisation ratio U is equal to 1/FoS when the calculation of FoS is based on design values. Hence $U = 1/1.018 = 0.98$ (acceptable as $U \leq 1.0$).

The design slope angle of 29° is marginally greater than value of φ'_d which is 28.4°. It is noted that the surface material on the slope would fail at 28.4° assuming shallow slip failures; this has however been prevented in the calculation by specifying a minimum ground weight included in each slip circle. This is considered acceptable where the surface material will be stronger than the deeper material (higher φ'_d) as a function of lower effective stresses, vegetation and suction effects etc. Where there is a risk of imperfect surface protection then the slope angle will need to be reassessed and the minimum angle of 28.4° or less should be considered. Surface protection in the form of vegetation is recommended to provide protection against shallow degradation of the slope associated with rain etc. A discussion should be held with the organisation responsible for the slope to investigate any limiting conditions that they may have for slope maintenance (possibly requiring a flatter slope to allow safe access etc.) The assumption of water below the

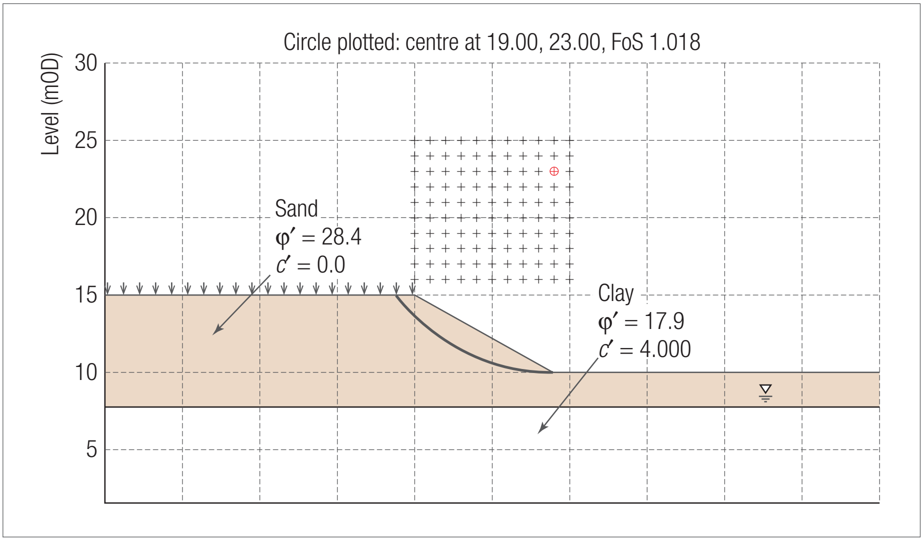

Fig B.2 Slope calculation with design (Set A2 + M2 factors) actions and material strengths: FoS = 1.02, $U = 0.98$.

base of the sand layer is critical in achieving the slope angles above; if there is a risk of higher water level this may result in a shallow slope being required.

It is noted that the required angle for a DA1 C1 calculation is found to be steeper than that for DA1 C2 with geometry of 5 to 7.5 horizontal being found to be non-critical.

Appendix C Spread foundation example

Introduction

The purpose of this design example is to illustrate the use of the 'verification by calculation' method for a spread foundation (see Section 2.4). The following aspects are addressed:
– combination of actions
– calculation of ground resistance
– use of the single source principle.

Combination of actions will be addressed in terms of ultimate (ULS) and serviceability (SLS) limit states. Of the various limit states applicable to spread foundations (see Section 6.2), bearing and sliding resistance failure will be examined.

Task

The task is to determine the design plan dimensions B_d of the square spread foundation shown in Figure C.1 which is subject to loading by an inclined column.

Ground conditions

A project specific ground investigation has been carried out. The data from the Geotechnical Investigation Report (GIR) has been examined and the ground model given in Table C.1 developed. The characteristic groundwater level and weight density are +7.0mOD and 9.8kN/m^3 respectively.

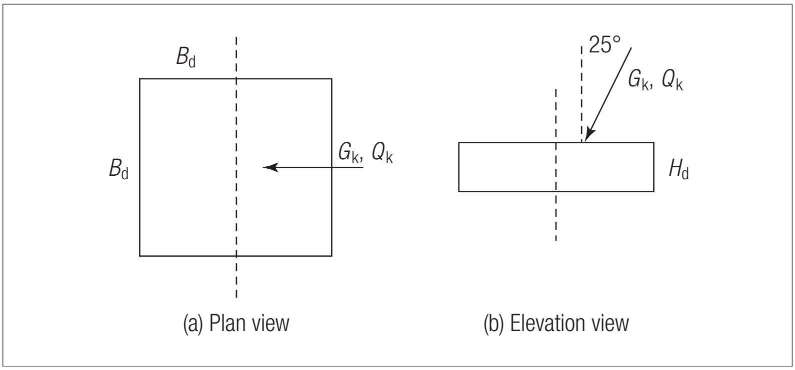

Fig C.1 Loading conditions

Table C.1 Ground conditions – spread foundations example

Stratum	Top level[a]	Weight density	Angle of shearing resistance	Angle of shearing resistance (constant volume)[a]	Young's modulus	Poisson's ratio
	a_d (mOD)	γ_k (kN/m^3)	φ'_k (deg)	$\varphi_{cv;d}$ (deg)	$E'_{v;d}$ (MN/m^2)	ν_d
Topsoil	+9.15	–	–	–	–	–
Sand/gravel	+9.0	20	36	29	40	0.2
Bedrock	−1.0	–	–	–	–	–

Note
a Determined directly as design values (see Section 2.11.2).

Simplifications
Some of the main simplifications which have been made to provide clarity to this example include:
– Geometrical values are assumed to be design values (see Section 2.11.2).
– SLS and ULS characteristic values are assumed to be the same.

Geotechnical category
– GC2 (see Section 2.9).

Limit states
– Bearing resistance – ultimate limit state (GEO).
– Sliding resistance – ultimate limit state (GEO).
– Settlement – serviceability limit state (SLS).

Design situation
– Persistent.
– The founding stratum is sand/gravel overlying competent bedrock.
– The characteristic permanent and variable actions from the column are $G_k = 800$kN, $Q_k = 200$kN.
– The design height H_d of the foundation is 1.0m (see Section 2.11.2).
– The line of the column passes through the centre of the base of the foundation and the column transfers no bending moment, so there is no eccentricity of the load on the foundation.
– The design working life is 50 years (see Section 2.7).
– The limit on settlement $C_d = 25$mm.

Partial factors
– The partial factors in Table C.2 are applicable to the GEO ultimate limit state.

Table C.2 Partial factors for Design Approach 1 – spread foundations example

Partial factor for…	Notation	Design Approach 1 Combination 1 value	Design Approach 1 Combination 2 value
Permanent favourable action	$\gamma_{G;stb}$	1.0	1.0
Permanent unfavourable action	$\gamma_{G;dst}$	1.35	1.0
Variable favourable action	$\gamma_{Q;stb}$	0.0	0.0
Variable unfavourable action	$\gamma_{Q;dst}$	1.5	1.3
Angle of shearing resistance	γ'_{φ}	1.0	1.25
Weight density	γ_{γ}	1.0	1.0

Note
Source: Section 2.11.3.

Design materials

Geotechnical

For this drained analysis, the peak angle of shearing resistance and weight density are characteristic values and therefore need to be modified by partial factors. For this example, a design value for the constant volume angle of shearing resistance has been selected directly. See PD 6694-1[34] or BS 8002[30] for further guidance.

The design values for the sand/gravel are therefore:
– $\varphi'_d = \tan^{-1}(\tan(\varphi'_k)/\gamma'_{\varphi})$
– $\varphi_{cv;d}$
– $\gamma_d = \gamma_k \gamma_{\gamma}$
which for Combinations 1 and 2 become the values shown in Table C.3.

Design actions

As the solution to the ULS calculation is iterative and the combined actions vary according to the foundation size, the values shown herein are based on

Table C.3 Material design values

Design value	Units	Combination 1	Combination 2	SLS
φ'_d	degrees	36.0	30.2	–
$\varphi_{cv;d}$	degrees	29.0	29.0	–
$\gamma_{\gamma;d}$	kN/m^3	20.0	20.0	20.0
$\gamma_{w;d}$	kN/m^3	9.8	9.8	–
$\gamma_{c;d}$	kN/m^3	25.0	25.0	–
$E'_{v;d}$	MN/m^2	–	–	40.0
v_d	–	–	–	0.2

an iterated foundation dimension $B_d = 2.4$m. In reality, an initial value for B_d would be selected and then iterative calculations performed to arrive at an economical solution.

Structural

The characteristic structural actions from the inclined column can be divided into the following vertical and horizontal component actions:
$G_k = 338$kN (horizontal component)
$G_k = 725$kN (vertical component)
$Q_k = 84$kN (horizontal component)
$Q_k = 181$kN (vertical component)

As the horizontal and vertical actions from the inclined column come from a single source (i.e. the axial column load), calculations only consider either (i) favourable horizontal and favourable vertical or (ii) unfavourable horizontal and unfavourable vertical actions, to co-exist.

If the horizontal and vertical actions were not from a single source then an unfavourable action could be considered to co-exist with a favourable action; such may be the case when verifying sliding resistance (i.e. favourable vertical and unfavourable horizontal action).

The characteristic structural action from the weight of the footing is:

$$G_k = \gamma_{c;d}\, H_d\, B_d^{\,2} = 25.0 \times 1.0 \times 2.4^2 = 144\text{kN}$$

For the ULS the structural component actions are then factored and combined (see Section 2.11.3) to give the values shown in Table C.4:

Table C.4 Structural component actions

Actions	Combination 1	Combination 2
Horizontal component – favourable: $F_d = G_k\,\gamma_{G;stb} + Q_k\,\gamma_{Q;stb}$ $= 338 \times 1.0 + 84 \times 0.0$	338kN	338kN
Horizontal component – unfavourable: $F_d = G_k\,\gamma_{G;dst} + Q_k\,\gamma_{Q;dst}$ $= 338 \times 1.35 + 84 \times 1.5$ $= 338 \times 1.0 + 84 \times 1.3$	583kN –	– 448kN
Vertical component – favourable: $F_d = G_k\,\gamma_{G;stb} + Q_k\,\gamma_{Q;stb}$ $= (725 \times 1.0 + 144 \times 1.0) + 181 \times 0.0$	869kN	869kN
Vertical component – unfavourable: $F_d = G_k\,\gamma_{G;dst} + Q_k\,\gamma_{Q;dst}$ $= (725 \times 1.35 + 144 \times 1.35) + 181 \times 1.5$ $= (725 \times 1.0 + 144 \times 1.0) + 181 \times 1.3$	1445kN –	– 1105kN

For the SLS a choice needs to be made as to which actions affect the limit state and whether the variable component is characteristic, frequent or quasi-permanent. For this example, the characteristic combination of vertical loads is assumed to affect the occurrence of the settlement limit state. These component actions are then factored (noting that the factor is 1.0) and combined (see Section 2.11.4) to give:

$F_d = G_k + Q_k = 725 + 144 + 181 = 1050\text{kN}$

Geotechnical
At present the characteristic groundwater level is 2m below the foundation base. However, it is possible that the groundwater level could rise above this level. As discussed in Section 4.2, a design groundwater level may be selected directly. For the purposes of ULS calculations it has been assumed that the groundwater may rise to the top of the foundation at +9.0mOD.

Therefore, the geotechnical action present for the ULS is the design groundwater pressure acting on the base of the foundation:

$u_d = \gamma_{w;d}\, H_d = 9.8 \times 1.0 = 9.8\text{kN/m}^2$

Bearing resistance
Using the equation for bearing resistance under drained conditions presented in Section 6.8.2, the design bearing resistance R_d can be calculated as follows:

$R_d = q'\, N_q\, d_q\, s_q\, i_q + \frac{1}{2}(\gamma'\, B'\, N_\gamma\, d_\gamma\, s_\gamma\, i_\gamma)$ (kN/m²) (See Table C.5)

Table C.5 Bearing resistance calculation

Calculation	Units	Combination 1	Combination 2
$q' = H_d\,(\gamma_{\gamma;d} - \gamma_{w;d})$	kN/m²	10.2	10.2
$N_q = e^{\pi \tan \varphi'd} \tan^2(45 + \varphi'_d/2)$	–	37.8	18.8
$s_q = 1 + \sin \varphi'_d$	–	1.59	1.50
$i_q = (1 - F_d/F_d)^{1.5}$	–	0.46	0.45
$d_q = 1 + 2 \tan \varphi'_d\,(1 - \sin \varphi'_d)^2 \tan^{-1}(H_d/B_d)$	–	1.10	1.12
$N_\gamma = 2\,(N_q - 1) \tan \varphi'_d$	–	53.4	20.6
$s_\gamma = 0.7$	–	0.70	0.70
$i_\gamma = (1 - F_d/F_d)^{2.5}$	–	0.27	0.27
$d_\gamma = 1.0$	–	1.00	1.00
$\gamma' = \gamma_{\gamma;d} - \gamma_{w;d}$	kN/m²	10.2	10.2
$R_d =$	kN/m²	434	195

Note
For i_q and i_γ the F_d values are the unfavourable values and the ratio F_d/F_d is the horizontal component divided by the vertical component.

For bearing resistance it is clear that unfavourable actions are going to dominate, so favourable combinations are ignored.

Verification
Aim: verify that $R_d \geq E_d$.

The total effect of the actions is equal to that shown in Table C.6.

Table C.6 Effect of actions – bearing resistance verification

Design value	Combination 1 (kN/m^2)	Combination 2 (kN/m^2)
$E_d = F_d/B_d^{\,2} - u_d$	241	182
Note For F_d the unfavourable vertical component is used.		

Therefore after checking several values of B_d it can be seen that for $B_d = 2.4$m, $R_d \geq E_d$ with unfavourable actions for Combination 2 being critical.

Sliding resistance
Using the equation for sliding resistance under drained conditions presented in Section 6.8.3, the design sliding resistance R_d can be calculated as shown in Table C.7.

Table C.7 Design sliding resistance

Design sliding resistance	Unfavourable		Favourable	
	Combination 1 (kN/m^2)	Combination 2 (kN/m^2)	Combination 1 (kN/m^2)	Combination 2 (kN/m^2)
$R_d = (F_d - u_d\, B_d^{\,2}) \tan \varphi_{cv;d}$	770	581	450	450
Note For F_d the vertical component is used.				

Verification
Aim: verify that $R_d + R_{p;d} \geq E_d$.
$R_{p;d}$ (resistance from passive earth pressure resistance) has been assumed to be zero.

The total effect of the actions E_d is equal to that shown in Table C.8.

Table C.8 Effect of actions – sliding resistance verification

Action	Unfavourable		Favourable	
	Combination 1 (kN/m^2)	Combination 2 (kN/m^2)	Combination 1 (kN/m^2)	Combination 2 (kN/m^2)
$E_d = F_d$	583	448	338	338
Note For F_d the horizontal component is used.				

Therefore, it can be seen that for ratio of horizontal to vertical actions, R_d is significantly larger than E_d.

Settlement
EC7 gives no specific equations for use in the calculation of settlement, however it does give guidance as the type of calculation that is acceptable. For the purposes of this example settlement will be calculated using a simple equation of the following form:

Settlement $E_d = q_n B (1 - v^2) I/E$

where:
q_n is the net bearing pressure
B is the foundation width $= B_d$
v is Poisson's ratio $= v_d$
I is the influence factor dependent on foundation geometry
E is Young's modulus of soil $= E'_{v;d}$

The groundwater level is not important in this case as the net bearing pressure is independent of this level. So,

$q_n = F_d/B_d^2 - H_d\,\gamma_{\gamma;d} = 1050/2.4^2 - 1.0 \times 20 = 162\text{kN/m}^2$

Note that for F_d the vertical component is used.

The influence factor (Terzaghi[112]) varies according to the ratio of the foundation length to width and the foundation rigidity. For this example the ratio is unity and the influence factor varies from 0.66 at the corner of a flexible foundation, to 0.90 for a rigid foundation, to 1.12 for the centre of a flexible foundation.

Substituting these values into the equation for settlement yields:
– for a flexible foundation, a range in settlement of 6 to 11mm
– for a rigid foundation, a settlement of 8mm.

The verification for the SLS is to show that $E_d \le C_d$ and in this case it can be seen that for either assumption, E_d is less than C_d of 25mm.

Appendix D Pile foundation example

Introduction
This design example presents the process for the design of an axial loaded pile in compression.

Task
Pile design (see Figure D.1), in a city centre environment, for the following column load:
– Characteristic permanent (dead) load: $G_k = 1.25MN$
– Characteristic variable (live) load: $Q_k = 0.50MN$
– All actions are considered unfavourable.

Ground conditions
A project specific ground investigation has been carried out, returning the ground stratigraphy and parameters shown in Table D.1. Figure D.2 shows the geotechnical SPT data (*N* values) on which the summary Table D.1 is based.

Geotechnical category (see Section 2.9)
– The Geotechnical Category is GC2.

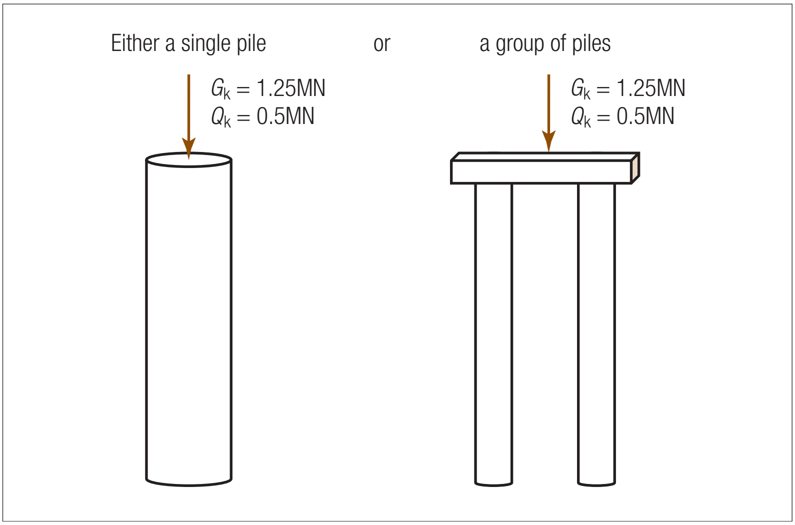

Fig D.1 Design task

Table D.1 Summary ground strata and parameters – pile foundations example

Stratum	Top level (mOD)	Density (kN/m^3)	φ'_k (°)	c'_k (kN/m^2)	SPT, N_k	$c_{u;k}$ (kN/m^2)[a]
Groundwater	+9	10	–		–	–
Made Ground/Brickearth	+15	18	25	0	7	–
Terrace Gravel: Rounded and sub-rounded sandy gravel – medium dense to dense	+10	20	38	0	30	–
London Clay: Stiff becoming very stiff silty clay	+6	20	–	–	$23 + 0.88z$[b]	$103 + 4z$[b]
Lambeth Group strata	−30	–	–	–	–	–

Notes
a $c_u = 4.5N$ (kN/m^2) based on Stroud[111].
b z is distance in metres below + 6mOD.

Limit states (see Sections 2.3 and 7.3)
– Ultimate limit state: formation of a failure mechanism (geotechnical pile failure).

Design situation (see Sections 2.5 and 7.4)
– Persistent (normal conditions).
– Variation in ground conditions.

Design considerations (see Section 7.5)
– Depth of adequate bearing stratum.
– Integrity of foundation once installed.

Design life (see Section 2.7)
– 50 years for buildings/common structures (a pile is likely to be durable for much longer and pile reuse should be considered during the documentation of the pile design and construction processes).

Pile design
Preliminary assessment based on Table 7.5 suggests that a pair of 0.6m diameter CFA piles may be appropriate for this load in the above ground conditions; use this as a starting place. For installation processes (not strictly an EC7 consideration for a bored pile) the upper bound SPT *N* values may be of importance in the assessment of ease of pile installation, the data is assessed so that pile installation should be possible.

Details of CFA pile being proposed:
– Number of piles per column: assume 2 Number.
– Diameter = 0.6m.
– Toe level = to be identified.

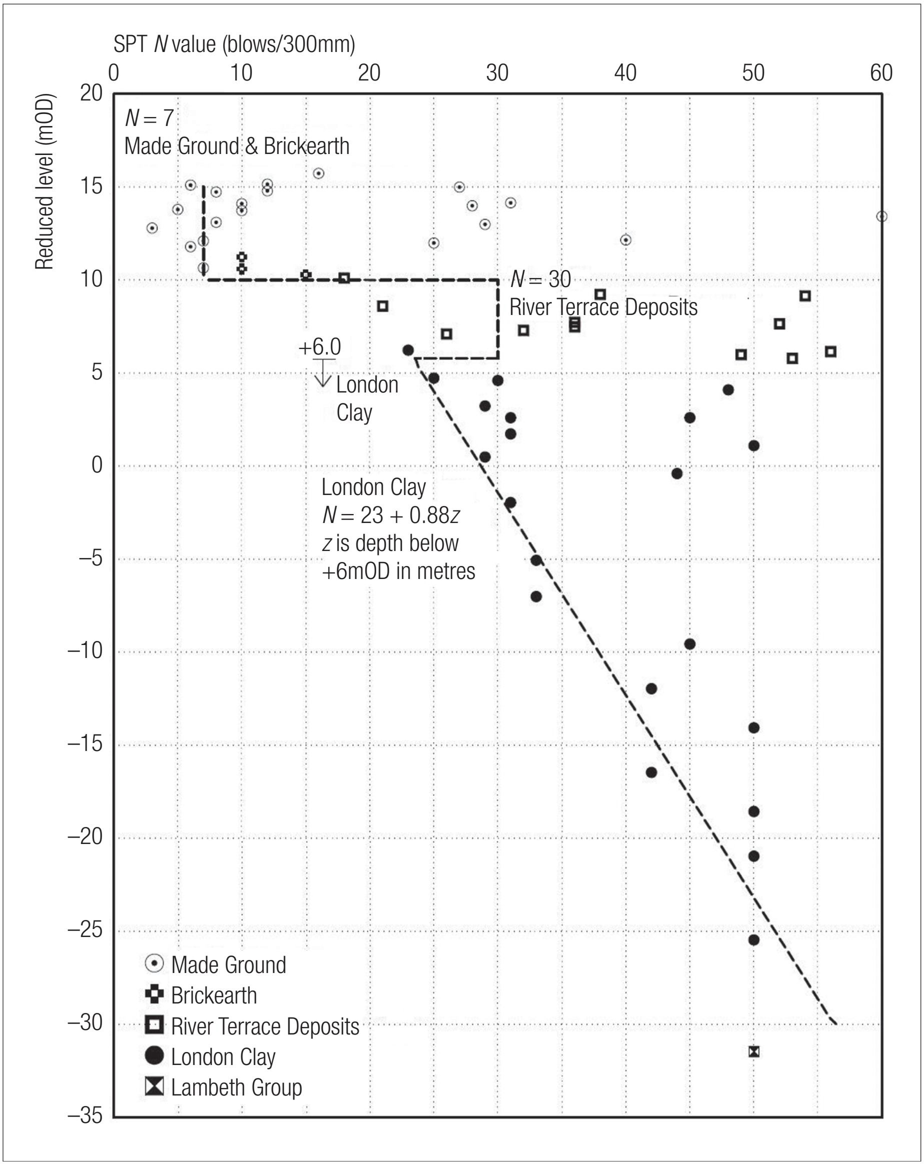

Fig D.2 Geotechnical data

– Method of construction: CFA pile bearing in London Clay. Limitation on pile depth for CFA piling plant proposed is 25m.
– Specification of pile construction to be carried out and agreed with piling contractor (specifically for CFA piles checks flighting during pile installation and on concrete pressure/concrete oversupply during concreting).

– All piles to be subjected to integrity testing.
– Working pile load tests to be carried out to 1.5 times the representative load ($G_k + Q_k$) on at least 1% of contract piles (no trial pile).

Pile design assumptions:
– Made Ground and Brickearth do not contribute positively or detrimentally to the axial design of the pile.
– CFA installation is such that shaft friction q_s can be mobilised within the Terrace Gravel stratum. $q_s = k \tan(\delta) = 0.7 \tan\left(\frac{2\varphi'}{3}\right)$.
– Shaft friction q_s in London Clay is limited to an average value of 110kN/m^2. $q_s = \alpha\, c_u$ (where $\alpha = 0.5$).
– Base resistance q_b in London Clay is subject to correct pile construction. $q_b = N_c\, c_u$ (where $N_c = 9$).
– Pile cap weight ignored herein (assumed ground bearing).

Pile design calculation
Load combinations considered:
Design Approach 1 Combination 2 uses the following set of factors:
– A2 + (M1 or M2) + R4 (see Section 2.11.3.4).
– A2 factors are as per Table 7.10.
– Where pile interaction with the ground is beneficial M1 factors apply (rather than M2 factors which apply when ground interaction with the pile results in pile loading). For M1, $\gamma = 1.0$ on all components of soil strength.
– R4 factors for pile design are as per Table 7.13.

Calculation of design action F_d:
The calculation of F_d is given in Table D.2.

Table D.2 Calculation of F_d – pile example

Set A2:	$G_k = 0.625$MN, $\gamma_G = 1.0$ for all permanent action $Q_k = 0.25$MN, $\gamma_Q = 1.3$ for leading (only) unfavourable action ($\gamma_Q = 0.0$ for favourable actions)	
	$F_d = (\gamma_G\, G_k) + (\gamma_Q\, Q_k) = (1.0 \times 0.625) + (1.3 \times 0.25) = 0.625 + 0.325$	$F_d = 0.95$MN
Note Calculation of F_d for one pile, each pile therefore carries 50% of column load.		

Calculation of design resistance R_d:
The pile design will use geotechnical parameters to calculate pile design resistance. The design resistance is:

$R_d = R_k/(\text{R4 factor})$

where:
$R_k = R_{cal}/\gamma_{R;d}$
$\gamma_{R;d}$ is the model factor
R_{cal} is calculated using characteristic ground parameters

Table D.3 Calculation of $R_{s;cal}$ and $R_{b;cal}$

Stratum	$R_{s;cal}$ (kN)	$R_{b;cal}$ (kN)
Made Ground/Brickearth	0	–
Terrace Gravel	297	–
Surface of London Clay to −4mOD	1159 (Check: average $q_{s;cal} = 62\text{kN/m}^2 < 110\text{kN/m}^2$, okay)	At −4mOD: 363
Surface of London Clay to −5mOD	1296 (Check: average $q_{s;cal} = 63\text{kN/m}^2 < 110\text{kN/m}^2$, okay)	At −5mOD: 374
Surface of London Clay to −6mOD	1436 (Check: average $q_{s;cal} = 64\text{kN/m}^2 < 110\text{kN/m}^2$, okay)	At −6mOD: 384
London Clay to −7mOD	1580 (Check: average $q_{s;cal} = 65\text{kN/m}^2 < 110\text{kN/m}^2$, okay)	At −7mOD: 394

Using the geotechnical data and the pile design basis above, the calculated resistance of the pile can be obtained as in Table D.3.

For pile design using characteristic soil parameters it is necessary to introduce a model factor into the design to calculate the characteristic resistance of the pile (the calculation of shaft and base resistance from characteristic ground parameters does not provide the characteristic shaft and base resistances but the calculated shaft and base resistances). The characteristic shaft and base resistance being obtained as follows:

$$R_{s;k} = R_{s;cal}/\gamma_{R;d}$$
$$R_{b;k} = R_{b;cal}/\gamma_{R;d}$$

where:
$\gamma_{R;d}$ is the model factor

It should be noted that if pile design is carried out using profiles of geotechnical data, as in pile design from CPT data, then an alternative approach is taken to arrive at the characteristic pile resistance.

As per Table 7.15, the value of $\gamma_{R;d}$ is 1.4 when there is no maintained pile load test taken up to the calculated ultimate (geotechnical) resistance. When maintained pile load test data taken up to the calculated ultimate (geotechnical) resistance exist, the value of $\gamma_{R;d}$ may be reduced to 1.2. The calculation of shaft and base characteristic resistances is shown in Table D.4.

For Design Approach 1 Combination 2 there are two values for the R4 resistance factors (for both γ_s and γ_b) which depend on the scope of pile load testing being proposed; Table 7.11 presents these. For a site with at least 1% of contract piles subjected to maintained load testing to 1.5 times the

Table D.4 Calculation of $R_{s;k}$ and $R_{b;k}$

Toe level (mOD)	$R_{s;k}$ (kN)	$R_{b;k}$ (kN)
−4	$(297 + 1159)/1.4 = 1040$	$363/1.4 = 260$
−5	$(297 + 1296)/1.4 = 1138$	$374/1.4 = 267$
−6	$(297 + 1436)/1.4 = 1238$	$384/1.4 = 274$
−7	$(297 + 1580)/1.4 = 1341$	$394/1.4 = 282$

representative load ($G_k + Q_k$) the partial factors on characteristic resistance of CFA piles are:
– Shaft: $\gamma_s = 1.4$ where $R_{s;d} = R_{s;k}/\gamma_s$
– Base: $\gamma_b = 1.7$ where $R_{b;d} = R_{b;k}/\gamma_b$
– (Total: $\gamma_t = 1.7$ where $R_{c;d} = R_{c;k}/\gamma_t$)

The partial factor γ_t on total or combined (shaft + base) characteristic resistance $R_{c;k}$ is provided but need not be used where shaft and base resistances are independently calculated.

The design resistance with depth per 0.6m diameter CFA pile is given in Table D.5.

Table D.5 Values of R_d for different pile toe levels

Toe level (mOD)	Pile length from +15mOD (m)	$R_{s;d}$ (kN)	$R_{b;d}$ (kN)	R_d (kN)
−4	19	$1040/1.4 = 743$	$260/1.7 = 153$	846
−5	20	$1138/1.4 = 813$	$267/1.7 = 157$	970
−6	21	$1238/1.4 = 884$	$274/1.7 = 161$	1045
−7	22	$1341/1.4 = 958$	$282/1.7 = 166$	1124

Required length of pile
The required inequality for design is:

$$F_d \leq R_d$$

For two 0.6m diameter CFA piles per column, assuming no preliminary pile tests and assuming at least 1% of contract piles are subjected to maintained load testing to 1.5 times the representative load, the required pile toe level is then −5mOD. This is a 20m long pile and is considered to be within the capability of the typical CFA piling plant.

Check per pile: $(F_d = 0.95\text{MN}) \leq (R_d = 0.97\text{MN})$ CORRECT

Design Approach 1 Combination 1 check
The calculation should be repeated for Design Approach 1 Combination 1 factors in which the same set of pile testing assumptions must be adopted (namely the model factor of 1.4). In this case the design action F_d per pile will be:

$$F_d = (\gamma_G\, G_k) + (\gamma_Q\, Q_k) = (1.35 \times 0.625) + (1.5 \times 0.25)$$
$$= 0.844 + 0.375 = 1.22\text{MN}$$

The design resistance of a pile with a toe level of −5mOD for DA1 C1 can be calculated:

$$R_d \text{ per pile (MN)} = R_{s;d} + R_{b;d} = R_{s;k}/\gamma_s + R_{b;k}/\gamma_b$$
$$= 1.138/1.0 + 0.267/1.0 = 1.405\text{MN}$$

Clearly for this DA1 C1 calculation for a pile to −5mOD,
F_d (1.22MN) $\ll R_d$ (1.405MN) and this is not the critical design case.

Other issues
– It is necessary to check the structural design of the pile (DA1 Comb 1: $F_d = 1.22$MN per pile, equivalent to a concrete stress of 4.31N/mm^2 assuming no moment) and to specify appropriate sulphate resistance for the pile.
– Consideration of settlement (SLS) should also be carried out.

Appendix E Retaining structure example

Introduction

The purpose of this design example is to illustrate the use of the 'verification by calculation' method for an embedded retaining structure (see Section 2.4). The following aspects are addressed:

– combination of actions
– avoidance of rotational failure
– calculation of prop resistance
– single source principle.

Task

The task is to determine the design depth of embedment d_d and design prop action p_d for the steel sheet pile wall shown in Figure E.1.

Ground conditions

A project specific ground investigation has been carried out. The data from the Geotechnical Investigation Report (GIR) has been examined and the ground model given in Table E.1 developed.

Simplifications

Some of the main simplifications which have been made to provide clarity to this example include:

– Geometrical values are assumed to be design values (see Section 2.11.2), a 0.5m allowance for overdig has been included for these ULS calculations.

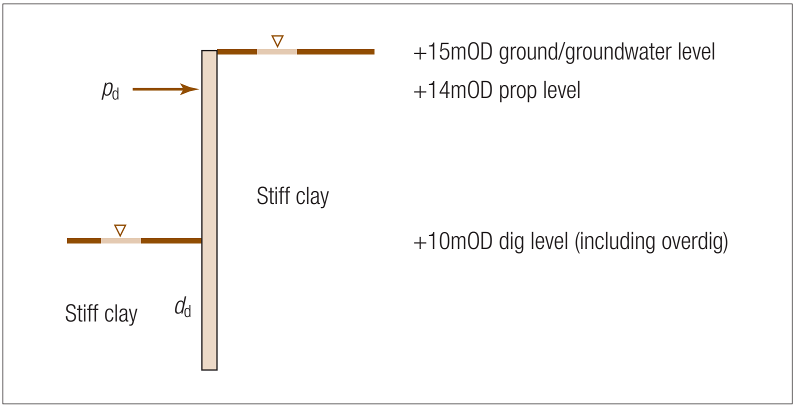

Fig E.1 Elevation view

Table E.1 Ground conditions – retaining structures example

Stratum	Top level	Weight density	Cohesion intercept	Angle of shearing resistance (peak)	Angle of shearing resistance (constant volume)
	a_d (mOD)	γ_k (kN/m^3)	c'_k (kN/m^2)	φ'_k (deg)	$\varphi_{cv;k}$ (deg)
Stiff clay	+15.0	20	0	28	18
Bedrock	−5.0	–	–	–	–
Groundwater	+15.0	9.8	–	–	–

– The characteristic groundwater level has been taken to be at ground level (see discussion on choosing water levels and pressures presented in Section 4.2.4).
– No surcharges are present (though in reality they are likely).
– Calculation by simple limit equilibrium.
– Seepage forces are calculated using a simple model.

Geotechnical category
– GC2 (see Section 2.9).

Limit states
– Rotational failure – ultimate limit state (GEO).
– Prop failure (STR).
– Wall structure failure (STR).

Design situation
– Persistent (implies amongst other things, drained conditions).
– The founding stratum is stiff clay overlying competent bedrock.
– Groundwater flow has reached steady state conditions.
– The design working life is 50 years (see Section 2.7).
– The desired excavation level is +10mOD.
– The wall is propped at +14mOD.

Partial factors
The partial factors in Table E.2 are applicable to the GEO ultimate limit state.

Design materials
Geotechnical
For this drained analysis, the peak angle of shearing resistance (assumed to be the value affecting the occurrence of the limit state) and weight density are characteristic values and therefore need to be modified by partial factors.

For this example, a design value for the constant volume angle of shearing resistance has been selected directly. See PD 6694-1[34] or BS 8002[30] for further guidance.

Table E.2 Partial factors for Design Approach 1 – retaining structures example

Partial factor for...	Notation	Design Approach 1 Combination 1 value	Design Approach 1 Combination 2 value
Water pressure (or effects of water pressure)	γ_G	1.35	1.0
Earth pressure (or effects of earth pressure)	γ_G	1.35	1.0
Angle of shearing resistance	γ'_φ	1.0	1.25
Effective cohesion	γ'_c	1.0	1.25
Weight density of ground	γ_γ	1.0	1.0

Notes

a Source: Section 2.11.3.

b As the earth and water are each single sources, the partial factors applied to each are the same on both the active and passive sides of the wall.

The ratio of the wall-soil interface friction angle δ and φ_{cv} depends on soil type, wall type, vertical loading and drainage conditions. EC7 does not provide long term values for $\delta_d/\varphi_{cv;d}$ for steel piles in clay so a value of 0.5 has been selected for the purposes of this example. Refer to Section 8.15 for further information.

The design values for the stiff clay are therefore:

$\varphi'_d = \tan^{-1}(\tan(\varphi'_k)/\gamma_{\varphi'})$

$c'_d = c'_k/\gamma'_c$ (in this example $c'_k = 0$)

$\varphi_{cv;d} = \varphi_{cv;k}$ (selected directly, note that $\varphi_{cv;d}$ is only used for calculation of wall friction, δ)

$\gamma_d = \gamma_k \gamma_\gamma$ (Note: the symbol γ is the same for density and partial factor)

$\delta/\varphi_{cv} = \delta_d/\varphi_{cv;d} = 0.5$

which for Combinations 1 and 2 become the values shown in Table E.3.

By reference to the design charts for the earth pressure coefficients in Section 8.15 the design values in Table E.4 are found.

Table E.3 Material design values

	Combination 1	Combination 2
φ'_d	28.0	23.0
δ_d	9.0	9.0
γ_d	20.0	20.0
$\gamma_{w;d}$	9.8	9.8

Table E.4 Design earth pressure coefficients

	Combination 1	Combination 2
$K_{a;d}$	0.34	0.40
$K_{p;d}$	3.50	2.80

The pore-water pressure distribution has been derived by assuming a constant hydraulic gradient around the soil-wall interface for ease of illustration; in reality it may not be constant. The design pore-water pressure distribution and effect of seepage due to steady state conditions has been estimated using the method for calculation of u_f (the pore-water pressure at the toe of the wall), as presented in Section 8.17.3.

Design actions

Geotechnical

The effects of geotechnical actions include the horizontal active earth pressure, passive earth pressure and water pressure.

The equation for calculating the horizontal pressure on a wall presented in Section 8.15 simplifies as follows (when the surcharge and cohesion are zero for the active side):

$$\sigma_{a;h}(z) = K_a \left(\int \gamma \, dz - u \right) + u$$

This equation can be re-written in design terms as:

$$\sigma_{h;d} = \gamma_{G;dst} K_{a;d} (\gamma_{\gamma;d} - u_d) + u_d$$

The equation for calculating the horizontal pressure on a wall presented in Section 8.15 simplifies as follows when the surcharge and cohesion are zero for the passive side:

$$\sigma_{p;h}(z) = K_p \left(\int \gamma \, dz - u \right) + u$$

This equation can be re-written in design terms as:

$$\sigma_{h;d} = \gamma_{G;stb} K_{p;d} (\gamma_{\gamma;d} - u_d) + u_d$$

Design resistances

Structural

The prop force is the action effect. The prop needs to be designed to provide this resistance.

Free end rotational failure

The verification of the embedment depth required to avoid free end rotational failure is undertaken by resolving moments due to the earth and water

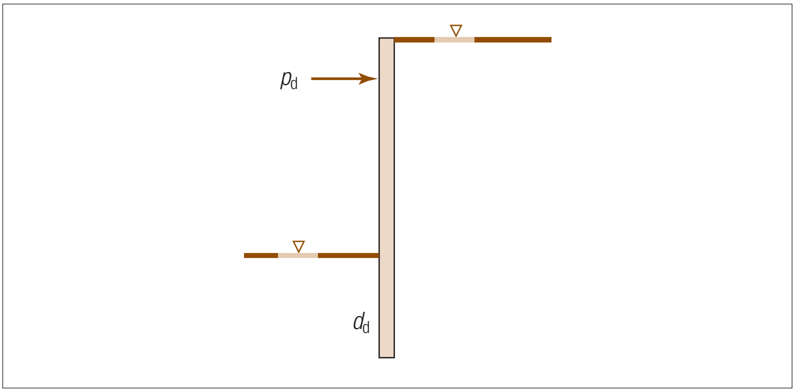

Fig E.2 Wall model

pressures about the point of application of the prop to the wall (see Figures E.2 and E.3).

The forces due to the ground (A, B) and water (C, D) are solved iteratively by varying the embedment depth d_d such that the sum of moments about the prop is zero. This would typically be done using computer software. The results for Combination 1 and 2 are presented in Tables E.5 and E.6.

It can be seen that the EC7 requirement that design effects of actions E_d be less than or equal to the design resistance R_d is achieved by comparing the moments resulting from areas A and C with areas B and D.

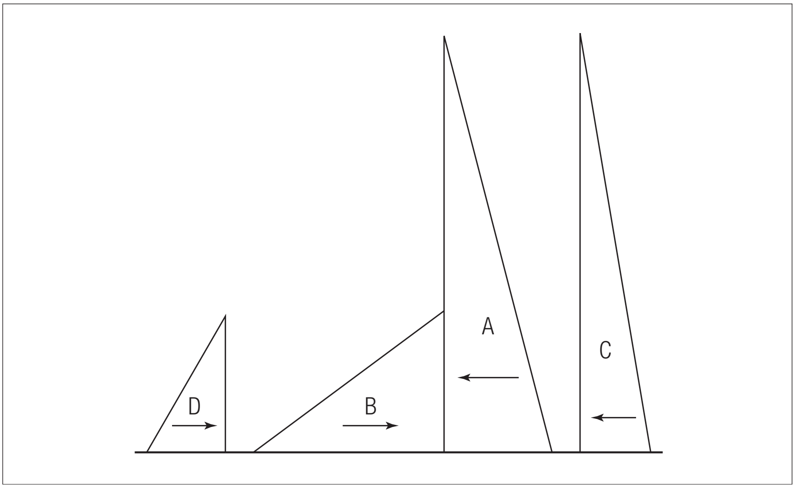

Fig E.3 Pressure diagram

Table E.5 Horizontal forces and moments – combination 1 for $d_d = 4.7$m

Area	Horiz. force (kN)	Moments about prop (kNm)
A	291	1586
B	−339	−2418
C	414	2259
D	−200	−1427
Sum	165	0

Table E.6 Horizontal forces and moments – combination 2 for $d_d = 6.0$m

Area	Horiz. force (kN)	Moments about prop (kNm)
A	311	1966
B	−351	−2801
C	424	2673
D	−230	−1838
Sum	154	0

It can also be seen that Combination 2, where soil strength is factored down, gives the critical design embedment depth.

Design prop load from limit equilibrium assessment
The calculation of design prop load E_d can be considered in a similar manner to that for rotational stability. Instead of summing moments about the prop, the horizontal forces on the wall can be resolved; in this case $E_d = 165$kN/m run (see last row of Table E.5). It is clear that the calculated prop load does not include temperature effects.

Design using proprietary computer software
While the example above has presented a hand calculation for assessment of wall embedment and prop load it is acknowledged that most wall designs will be carried out using computer software.

The input to a computer program will depend on the sophistication of the program but will likely include:
– soil strength data
– soil stiffness data
– wall stiffness data
– geometric data.

Depending on the nature of the program the actual values entered may be characteristic values, design values or pre-calculated values of K_a or K_p.

Even though the calculations being discussed here are ULS calculations (STR/GEO), the design stiffness values are typically taken to be equal to characteristic values which should reflect the limit state being considered. For wall design a moderately conservative (lower) stiffness value would be reasonable (EC0 Clause 4.2(8)[3]).

The following calculations should be carried out using characteristic values of soil and wall stiffnesses along with design soil strengths and a wall toe level which is in all cases 6.0m below formation level (as calculated above):

– DA1 C1: the approach taken here is to factor the effect of actions and not the actions themselves. To do this variable surcharges should be factored by the ratio of γ_Q/γ_G (=1.5/1.35 = 1.11) and then the resulting bending moment, shear forces and prop loads should be factored by γ_G (=1.35) to obtain the design bending moment etc.
– DA1 C2: partial factors on soil parameters and variable surcharge loading.

These two runs will provide the ULS design moments and shear forces as well as a check on the ULS prop load calculated above; the worse calculated actions should be taken forward to structural design.