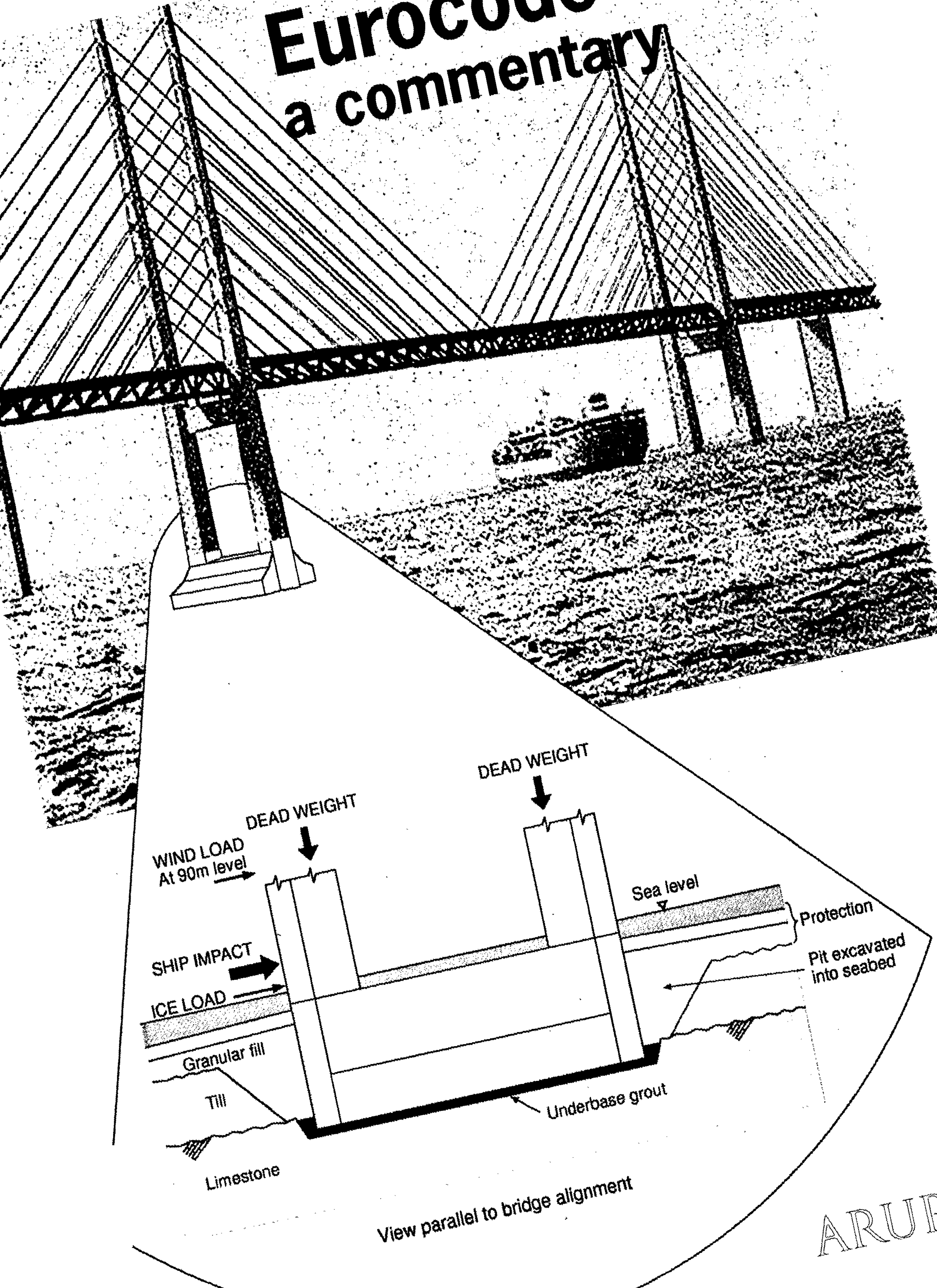
Eurocode 7
a commentary



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Eurocode 7

a commentary

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SUMMARY OF CONTENTS

FOREWORD

This Commentary consists of guidance and recommendations related to draft Eurocode 7 Part 1 – DD ENV 1997-1:1995. Its contents are necessarily of a general nature, and responsibility for the application of the commentary, in particular engineering projects, remains with the user.

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Eurocode 7: a commentary

Part A Fundamentals

CONTENTS

A1 INTRODUCTION

A1.1 Purpose of this commentary

This commentary is intended to help the reader to understand Eurocode 7, in the form published in 1995, by providing:

a reviews of new concepts;
b clarification of the text;
c comparisons against existing British practice;
d worked examples.

It does not attempt to replace text books on geotechnical engineering, but is limited to the task of explaining the intentions of Eurocode 7, especially where these differ from previous design approaches. The commentary does not debate alternatives to Eurocode 7; possible future changes to the Eurocode are discussed in Part D.

A1.2 Basis of this commentary

The basis of this commentary is Eurocode 7: Geotechnical Design – Part 1: General rules, published in 1995. The serial number of this document in the system of the Comité Européen de Normalisation is ENV 1997-1. (Note that in this reference '1997' is a reference number, not a year.) In this commentary it will generally be referred to as 'EC7-1', or, where it is obvious that Part 1 is referred to, simply as 'EC7'. In Britain, the British Standards Institution published DD ENV 1997-1:1995, containing EC7-1 together with its United Kingdom National Application Document (see A1.5 and A1.6).

The commentary is in five Parts, A to E, as explained in A3.1, and references to the commentary are in the form 'A1.2', 'C8.2.3', etc. References to Eurocodes are in the form 'EC7, 1.2.3', 'EC3-5, 5.3.4', etc. The latter of these references means 'Eurocode 3, Part 5, subclause 5.3.4'.

A1.3 Background of development of the Eurocodes

The objectives of the Eurocodes are set out in a foreword which is common to all of them. It appears as follows in ENV1997-1:1995, though in more recent Eurocodes item (2) has been omitted.

1 The structural Eurocodes comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.
2 They are intended to serve as reference documents for the following purposes:
(a) As a means to prove compliance of building and civil engineering works with the essential requirements of the Construction Products Directive (CPD).
(b) As a framework for drawing up harmonised technical specifications for construction products.
3 They cover execution and control only to the extent that is necessary to indicate the quality of the construction products, and the standard of the workmanship, needed to comply with the assumptions of the design rules.
4 Until the necessary set of harmonised technical specifications for products and for methods of testing their performance is available, some of the Structural Eurocodes cover some of these aspects in informative annexes.

In summary, the Eurocodes were originally intended as checking documents for design, proving compliance with the European regulations; the omission of Objective 2 makes this less clear. In their preparation there has been strong emphasis on the need to ensure fair competition in the construction industry and to harmonise requirements between countries. Their purpose is to give rules which will lead to acceptable assemblies of products. They are not intended to advise or educate; these tasks are left to text books. As item 3 above suggests, they contain relatively little about construction, which is left to other CEN standards. CEN committee TC288 has been responsible for the development of geotechnical construction standards (see Table A3.2 at the end of this Part).

The Eurocodes were initiated by the Commission of the European Communities (CEC) as a development of the Construction Products Directive (CEC Council Directive 89/106/EEC dated 21 December 1988, together with Interpretative Documents published 16 July 1993). The Construction Products Directive requires a series of harmonised European Standards which provide certain 'Essential Requirements' of safety, economy and fitness for use, but which do not, *by their* [national] *disparity, hinder trade within the Community.* The task of producing these Standards was delegated to the Comité Européen de Normalisation (CEN), of which national standards bodies such as BSI are members.

Initially, the Eurocodes were directed almost exclusively at building structures. However, their scope has gradually increased and now includes civil engineering structures such as bridges, with developments underway for towers, silos etc. Their main geotechnical application to date has been to bridge structures in Denmark (Braestrup (1996)) and France. In the geotechnical field, any of these structures may require a wide range of geotechnical design.

At the present stage of development (early 1998), nine Eurocodes have been nominated:

EN 1991 Eurocode 1 Basis of design and actions on structures
EN 1992 Eurocode 2 Design of concrete structures
EN 1993 Eurocode 3 Design of steel structures
EN 1994 Eurocode 4 Design of composite steel and concrete structures
EN 1995 Eurocode 5 Design of timber structures
EN 1996 Eurocode 6 Design of masonry structures
EN 1997 Eurocode 7 Geotechnical design
EN 1998 Eurocode 8 Design of structures for earthquake resistance.
EN 1999 Eurocode 9 Design of aluminium alloy structures

It is likely that Eurocode 1 will be divided into two documents: Basis of design separated from Actions on structures. The relationship of these Eurocodes to corresponding British codes is shown in Table A3.1 at the end of this Part.

A1.4 Historical note on the development of Eurocode 7

In 1976 the European Commission agreed to sponsor development of a set of European codes of practice for building structures. The purpose of these was to encourage free trade between member states. In 1980, an agreement was reached between the Commission of the European Communities (CEC) and the International Society for Soil Mechanics and Foundation Engineering (ISSMFE), according to which the Society should undertake to survey existing codes of practice for foundations within the member states and to draft a model code which could be adopted as Eurocode 7. In 1981, the ISSMFE established an ad hoc committee for this task. Following many consultations and international meetings, this committee produced in 1987 a 'draft model for Eurocode 7'. The CEC sponsored further work on this draft code for three years to 1990, after which the work was transferred for further development, issue and maintenance to CEN, with agreement that the EFTA secretariat would also support the work. CEN/TC250 was therefore set up, and this committee has overseen the development of Eurocodes since 1990. A sub-committee is responsible for each Eurocode, that for Eurocode 7 being CEN/TC250/SC7.

During the whole of this process, the development has been led and the main committees have been chaired by Dr Niels Krebs Ovesen of Denmark. The chairmanship of CEN/TC250/SC7 will pass to Dr Roger Frank, of France, in May 1998.

A1.5 National Application Documents

In the approach adopted to date, each nation is to publish a National Application Document (NAD) for each Eurocode. Numerical values of safety factors and some other quantities are given in the Eurocodes as 'boxed values', generally in square brackets [ ]. The boxed values are intended to be indicative, and each nation is to specify the values to be used for construction on its own territory (irrespective of the nationality or location of the designer). These are provided in the National Application Documents.

Some of the available NADs for EC7 have been quite extensive, and have added to or amended the rules of the Eurocode.

National Application Documents are written in the languages of their countries of origin, though most are being translated into English, at least informally. They may refer to other national codes which are not available in English. British engineers may reflect, however, on the great advantage they gain from the fact that English is the most common language in European usage.

The possible future status of NADs is discussed in D2.4.

A1.6 Current status of EC7 and the British NAD

Eurocode 7 Part 1 was published as an 'ENV' (a European pre-standard) by CEN in October 1994 and by BSI in 1995 as DD ENV 1997-1:1995. Two years later, in 1997, Eurocode 7 Part 1 was subjected to voting and accepted in July 1997 for further development to become an EN (European standard, or Euronorm). Since this process will take several years, the 1995 publication forms the basis of this commentary.

Also in 1997, ENV (pre-standard) versions of Parts 2 and 3 of Eurocode 7 were accepted for publication. These will probably be available in 1998, publication being withheld until translation into French and German is complete. Part 2 is a standard for 'Design based on laboratory testing' and Part 3 'Design based on field testing'. These are considered further in A2.5.

British Standard DD ENV 1997-1:1995 also contains the United Kingdom NAD. This is partly an 'NAD for the draft (ENV) EC7', but also principally a 'draft NAD for the final EC7 (EN)'. It is relatively short, though it contains some material in addition to the boxed values.

A2 HOW TO USE EUROCODE 7

A2.1 Who should use it

Eurocodes are intended to be used by engineers in general building and civil engineering design. They are particularly relevant on projects involving international cooperation or competition, especially on publicly funded work, where it may become a legal requirement to accept designs which satisfy the Eurocodes. They have also been found useful in some situations where the safety systems of existing national codes are difficult to apply.

Eurocodes are written for use by qualified and experienced personnel. The precise terms of these qualifications are not defined, except that they must be appropriate and adequate for the project in hand. In almost all cases, this demands that designs to EC7 are supervised by a qualified civil engineer with relevant geotechnical training and experience. It is also a basic assumption that all construction activities are carried out in accordance with relevant standards (EC7, 1.4).

It is not the main purpose of Eurocodes to provide assistance or information to designers, and they are not teaching manuals. Hence inexperienced designers will not be able to proceed adequately on the basis of the Eurocodes alone. This is particularly true for EC7.

The Eurocodes are intended to provide a check on the safety of proposed structures, so they will be used by both designers and checkers.

A2.2 The system of Eurocode documents

The current list of Eurocodes was provided in A1.3 above. Eurocode 1, Basis of design and actions on structures, sets out the basic approach to be adopted to design, especially calculations, and provides definitions for terminology in the limit state system. Some of this material is repeated in other Eurocodes, particularly EC2 and EC3, but it is intended that this repetition will be removed in the future. EC7 generally avoids repeating material from EC1, so it is necessary that the user of EC7 also has EC1 available and is familiar with it. It is possible that in the future 'Basis of design' will become 'EC0' – EN1990.

The Eurocodes aim to be an inter-dependent and consistent set, without repetition of material. Thus, to design a steel pile, EC7 will be used with EC3, or for a concrete retaining wall, EC7 will be used with EC2. The systems of factors of safety have been selected to make this possible. Design for seismic situations receives very little attention in Eurocodes 2 to 7 as these rely on EC8 for this purpose. EC8, however, refers back to other Eurocodes.

All the Eurocodes contain two levels of text: Principles and Application Rules. Principles are mandatory requirements of definitions. Application Rules are ways in which the design may be shown to comply with the Principles; they are not mandatory. This distinction is discussed further in C1.3.

It will be clear from A1.5 above that no current Eurocode is complete in itself, but must be supplemented by the NAD of the country in which construction is to take place. In Britain, EC7 and the UK NAD are published together by BSI, as noted in A1.6.

A2.3 Overview of Eurocode 7 Part 1

EC7-1 provides, in outline, all the requirements for design of geotechnical structures. It provides little assistance or information, which the designer must therefore seek in other texts. It relies upon other Eurocodes and CEN documents, as outlined in this section of the commentary.

The sections of EC7-1 may be grouped as follows:

Overall approach

1 General

2 Basis of geotechnical design

Ground investigation
3 Geotechnical data

Design aspects of construction activities
4 Supervision of construction, monitoring and maintenance

Design of specific elements
5 Fill, dewatering, ground improvement
6 Spread foundations
7 Pile foundations
8 Retaining structures
9 Embankments and slopes

All users should be familiar with Sections 1 to 4. In particular, Section 2 provides the basic approach to limit state design and to calculations, including the values of most of the partial factors of safety (but see A2.4 on the UK NAD); further factors are provided for design of piles and anchors in Sections 7 and 8. Users will generally then proceed to one of the sections 5 to 9 as required, though it should be noted that overall site stability is covered in Section 9, which is relevant to all the items considered in Sections 5 to 8.

Generally, the code provides specific rules for ultimate limit state requirements but leaves the criteria for serviceability limit states to be decided by designers and their clients. It outlines the basic requirements of calculation methods but in most cases does not give detailed formulae. The rules of CEN prohibit the inclusion of worked examples of calculations.

A discussion of the geotechnical design procedure based on EC7-1 is presented in B1.

A2.4 The United Kingdom National Application Document for Eurocode 7 Part 1

For design of projects to be constructed in the United Kingdom, the user of EC7-1 must use the UK NAD for EC7-1, contained in the BSI publication DD ENV 1997-1:1995. This provides the 'British' values of partial factors of safety and some other constants, which over-ride the 'boxed values' given in EC7-1 itself. In the 1995 publication, the British values are identical to the boxed values, but it would be unsafe to assume that this will remain the case in future, so it is wise to develop the habit of referring to the NAD for factor values, if the construction will be in the UK. Equally, for construction in another European country, the NAD of that country must be used, so the habit of using the NAD rather than EC7-1 itself will still be helpful.

The UK NAD also contains a short annex giving clarification of some of the more difficult clauses of EC7-1 and provides cross-references to relevant British codes. In particular, where EC7-1 has used phases like 'internationally recognised standards' or 'standardised procedures', the NAD requires, in its Annex A, that British Standards are used. (**Note:** In the 1995 publication of the NAD, the words 'Normative' and 'Informative' have been interchanged, in error, in the headings of Annexes A and B and in the list of references at the end of the NAD.)

A2.5 Eurocode 7, Parts 2 and 3

In general, Eurocodes do not consider testing of materials. EC1 Part 1 has a section on 'Design assisted by testing', but this is concerned mainly with testing of prototype structural elements. Nevertheless, the mandate for drafting Eurocode 7 required that it should include soil testing, and this led to the development of Parts 2 and 3, respectively 'Geotechnical design assisted by laboratory testing' and 'Geotechnical design assisted by field testing'. These documents were approved for publication as pre-standards (ENVs) in 1997. They are outside the scope of this commentary, but their future role is briefly described here.

In scope, Parts 2 and 3 of EC7 lie somewhere between BS 5930 on site investigation and BS 1377 on soil testing; they also provide rather more information on interpretation of test results. Part 3 provides guidance on the scope of site investigation required for common structures and Part 2 discusses the suites of tests which could be included. They are principally of use to engineers who have to specify and interpret testing, rather than to technicians responsible for carrying out the tests.

Both Parts lead to the calculation of 'derived values' for parameters, based either on direct calculation or on calibrations. For example, a method of deriving angle of shearing resistance from SPT results is given in Part 3. Each 'derived value' is based on a single test result; the set of derived values, possibly from more than one type of test, may be used in the assessment of characteristic values of the parameters to be used in design calculations. Thus derived values are not the same as characteristic values, which are discussed further in B4.

In their pre-standard (ENV) form, Parts 2 and 3 also contain some calculation methods based on the parameters considered. It is likely that these will either be moved into Part 1 or eliminated in the EN versions of these two parts.

A2.6 Other CEN and ISO documents

Lists of CEN and ISO standards relevant to geotechnical design and construction are presented in Tables A3.2 and A3.3, which also list corresponding or related British Standards.

Within the European system being developed by CEN, the Eurocodes are to be used together with codes on production of civil engineering products and on geotechnical construction practice. These are produced by various CEN technical committees, CEN/TC288 being responsible for the standards on geotechnical construction.

EC7, in the form ENV 1997-1:1995 was published before the current drafts of the TC288 documents and so does not refer to them. The TC288 standards refer to EC7, and also make some additions to its design requirements, most notably in relation to design of ground anchors in prEN 1537. However, it is intended that they will not contain design requirements when the system is complete, with all of them being adequately covered by EC7.

A2.7 Relationship to British Standards

Under the rules of CEN, codes published by national standards institutions that conflict with the principles of the Eurocode must normally be withdrawn when a CEN standard is produced at EN status. However, an exception has been made for Eurocodes, allowing a period of 'coexistence' of national standards with the EN for some years after publication of the EN.

For EC7 a particular problem arises in deciding which British Standards are in conflict with the EN. For example, BS 8002 contains material on the design of retaining walls which is not included in EC7. The same applies to other codes such as BS 8004, BS 8006, BS 5930, etc. The British codes provide much advisory information, which is 'informative' rather than obligatory, so they are somewhat different in nature from the Eurocodes; it might therefore be questioned whether they conflict with the Eurocode.

A possible way ahead for the user could be to design to the rules of EC7 but to use the British codes as supplementary advice. This approach is recommended by this commentary, though it would become difficult in a situation where the British code clearly demands a more conservative approach than does EC7. How would a court of law view a design which complied with the Eurocode but not with an equally current British code?

A possible solution to this dilemma might be for BSI to continue publication of the British codes, adding a rider that, in case of conflict, the Eurocode governs.

The situation is different for the relationship between EC7 Parts 2 and 3 and BS 1377 – Methods of test for soils for civil engineering purposes. BS 1377 provides precise descriptions of the apparatus and procedures required by tests, together with methods of calculating the results of individual tests. The use and interpretation of the tests are not within its scope, however. As noted in A2.5, EC7 Parts 2 and 3 consider mainly the planning, interpretation and use of suites of tests, but have relatively little detail on the test methods. Here again, the two codes are not in conflict.

A2.8 Some terminology

The Eurocodes introduce some terms which are not familiar in Britain. In some cases, English words are defined to have meanings which may be unexpected. Some of these terms are noted here, with references to other points in this commentary or the Eurocodes where more definitive definitions will be found. A fairly comprehensive list of terminology is given in EC1, 1.5, and this is supplemented by EC7, 1.5. See also C1.5.

Action
Generally equivalent to 'load'. EC1, 1.5.3.1 says that an action is an applied force or an imposed displacement. EC7, 2.4.2(1)P notes that *For any calculation the values of actions are known quantities. Actions are not unknowns in the calculation model.* See also C2.4.2.

Characteristic values
See B4.

Design values
See B2.4.

Execution
The construction of a project on site. See EC1, 1.5.1.10 and 1.5.1.1-5.

Limit states
See B2.1.

Principles and Application Rules
See EC1, 1.4 and EC7, 1.3. Principles are mandatory paragraphs; their numbers are followed by the letter P.

A3 HOW TO USE THIS COMMENTARY

A3.1 The five parts of the commentary

This commentary is divided into five sections which are intended to be used in different ways.

Part A – Fundamentals

The user should read through Part A to obtain a general background understanding to the use of Eurocode 7.

Part B – Important features of Eurocode 7 Part 1

Part B consists of a series of essays on concepts in EC7-1 which require clear understanding if the code is to be used effectively. Some of these concepts are relatively new and have been controversial during the development of EC7. They have been developed, however, because they are believed to offer the clearest way available to communicate the essence of good, systematised design practice.

The reader is advised to study Part B and to understand its concepts as well as possible.

Part C – Clause-by-clause commentary

Part C is drafted on the assumption that it will be read alongside EC7-1. It does not repeat the text of EC7-1, but is intended to provide clarification of its meaning, pointers to related clauses and guidance to other publications which assist the use of the clauses.

Part C should be used for reference whilst working with EC7-1.

Numbering refers specifically to the EC7 clause, subclause etc which is being discussed. Each numbered heading is prefaced by a 'C' to distinguish, for example, commentary on Clause 1.1 (C1.1) from Clause 1.1 itself (EC7, 1.1).

Part D – The way ahead

Part D considers the use of EC7-1 outside the United Kingdom, and also discusses future development and research needs.

Part E – Worked examples

Part E supplements earlier parts by providing extended examples of the application of EC7-1 to engineering design problems.

A list of errata for EC7-1 is provided as Appendix 2 in Part C.

A3.2 Abbreviations adopted

The following abbreviations are used in this commentary.

BSI	British Standards Institution
CEN	Comité Européen de Normalisation
ECn	Eurocode n
EC7-1	Eurocode 7 Part 1, as contained in the DD ENV 1997-1:1995
EN	Euronorm, European standard
ENV	published European pre-standard (Vornorm in German)
NAD	National Application Document. The United Kingdom NAD is contained in BSI publication DD ENV 1997-1:1995
prEN	Pre-norm – draft document circulated for comment but not generally published (similarly prENV)
SLS	serviceability limit state
ULS	ultimate limit state

A3.3 Requirements, recommendations and some administrative definitions

The Eurocodes use the verbs 'shall' and 'should' in a carefully defined manner. As noted in C1.3, 'shall' is used in Principles and 'should' in Application Rules.

In this commentary, the verb 'must' is used to mean that, in the opinion of the authors of the commentary, EC7-1 is imposing a mandatory requirement.

The word 'recommended' is used to indicate the recommendations of the authors of this commentary.

In C1.5.2, definitions are suggested for the words 'considered', 'assess' and 'evaluate', which are used repeatedly in EC7-1.

The following hierarchy of numbering is taken from 'Harmonised editorial style for Eurocodes' issued by CEN/TC250 in January 1996:

Section	3
Clause	3.1
Subclause	3.1.1
Paragraph	3.1.1(4)

In this commentary, these nouns have been omitted where the context allows. References to EC7-1 are in the form 'EC7, 1.2.3'. References to other Eurocodes appear as 'EC3-5, 5.3.4', etc., and references to the commentary are in the form 'A1.2', 'C8.2.3', etc.

Table A3.1 Eurocodes and corresponding British Standards

Eurocodes	**British Standards**
ENV1991 Basis of design and actions on structures: 1995 to 1997	BS 5400 Steel, concrete and composite bridges, Part 2, Specification of loads BS 6399 Loading for buildings
ENV 1992 Design of concrete structures: 1992 to 1996	BS 8110 Structural use of concrete BS 5400 Steel, concrete and composite bridges, Part 4, Design of concrete bridges BS 8007 Design of concrete structures for retaining aqueous liquids
ENV 1993 Design of steel structures: 1992 to 1998	BS 5950 Structural use of steelwork in building BS 5400 Steel, concrete and composite bridges, Part 3, Design of steel bridges
ENV 1994 Design of composite steel and concrete structures: 1994 to 1997	BS 5400 Steel, concrete and composite bridges, Part 5, Design of composite bridges
ENV 1995 Design of timber structures: 1994 to 1997	BS 5268 Structural use of timber
ENV 1996 Design of masonry structures: 1996 to 1997	BS 5628 Code of practice for the use of masonry
ENV 1997 Part 1 Geotechnical design: 1995	BS 5930 Site investigations BS 8002 Retaining walls BS 8004 Foundations BS 8006 Reinforced soil BS 6031 Earthworks BS 8081 Ground anchors BS 8103-1 Stability, site investigation, foundations and ground floor slabs for housing
ENV 1997 Part 2 Geotechnical design Standards for laboratory testing: 1995	BS 1377 Soils for civil engineering purposes, Parts 1 to 8
ENV 1997 Part 3 Geotechnical design Standards for field testing: 1995	BS 1377 Soils for civil engineering purposes, Parts 1, 4 and 9
ENV 1998 Design of structures for earthquake resistance: 1994 to 1997	No corresponding British Standard
EN 1999 Design of aluminium alloy structures	BS 8118 Structural use of aluminium, Parts 1 and 2

Table A3.2 Other Euronorms and corresponding British Standards[a]

Euronorms	British Standards
ENV 206 Concrete performance, production, placement and compliance criteria	BS 1881 Testing concrete BS 5328 Concrete Parts 1–4
EN 1536 Execution of special geotechnical works – Bored piles	BS 8004 Foundations BS 8008 Safety precautions and procedures for the construction and descent of machine-bored shafts for piling and other purposes BS 5228-4 Noise and vibration control applicable to piling operations
EN 1537 Execution of special geotechnical works – Ground anchors	BS 8081 Ground anchorages
EN 1538 Execution of special geotechnical works – Diaphragm walls	BS 8002 Retaining walls
EN 12063 Execution of special geotechnical works – Sheet pile walls	BS 8002 Retaining walls BS 5228-4 Noise and vibration control applicable to piling operations
EN 12699 Execution of special geotechnical works – Displacement piles	BS 8004 Foundations BS 5228-4 Noise and vibration control applicable to piling operations
EN 12715 Execution of special geotechnical works – Grouting	
EN 12716 Execution of special geotechnical works – Jet grouting	
prEN 12794 Execution of special geotechnical works – Precast concrete foundation piles	BS 8004 Foundations BS 5228-4 Noise and vibration control applicable to piling operations
prEN 13793 Building foundations – Thermal design to avoid frost heave	
prEN 288008 Execution of special geotechnical works – Micropiles	BS 8004 Foundations

[a] With the exception of ENV 206 and prEN 13793, all the above Euronorms have been developed by CEN committee TC288

Table A3.3 ISO and corresponding British Standards

ISO standards	British Standards
prEN ISO 13819-1 Offshore structures	BS 6349 Maritime structures, Parts 1 to 7
ISO 14688 Identification and classification of soils	BS 5930 Site investigations
ISO 14689 Geotechnics – Identification and description of rocks	BS 5930 Site investigations
ISO #BZNK Geotechnics – Laboratory and field investigation and monitoring	BS 1377 Soils for civil engineering purposes, Parts 1 to 9
ISO #BZNL Geotechnics – Foundations earthworks and retaining structures	BS 6031 Earthworks BS 8002 Retaining walls BS 8004 Foundations

Eurocode 7: a commentary

Part B Important features of Eurocode 7 Part 1

CONTENTS

B1 SUMMARY OF MAIN CONCEPTS

B1.1 Assumptions

Although it may well be applied elsewhere, it is important to realise that EC7-1 is drafted for use in Western Europe. Assumptions which follow from this are listed in Clause 1.4, which sets the standards for good practice to be followed on every project. The factors of safety used in the code are based on this good practice. Where standards of data collection, analysis, design, construction and maintenance fall below those standards, a more conservative design approach should be followed. Continuity between each of the stages of the project is also assumed, with a free flow of information and data between the ground investigators, designers and constructors.

The design process should therefore be an unbroken continuous process, with appropriately qualified and experienced personnel carrying out each stage.

Some problems in the interpretation of these assumptions are noted in C1.4.

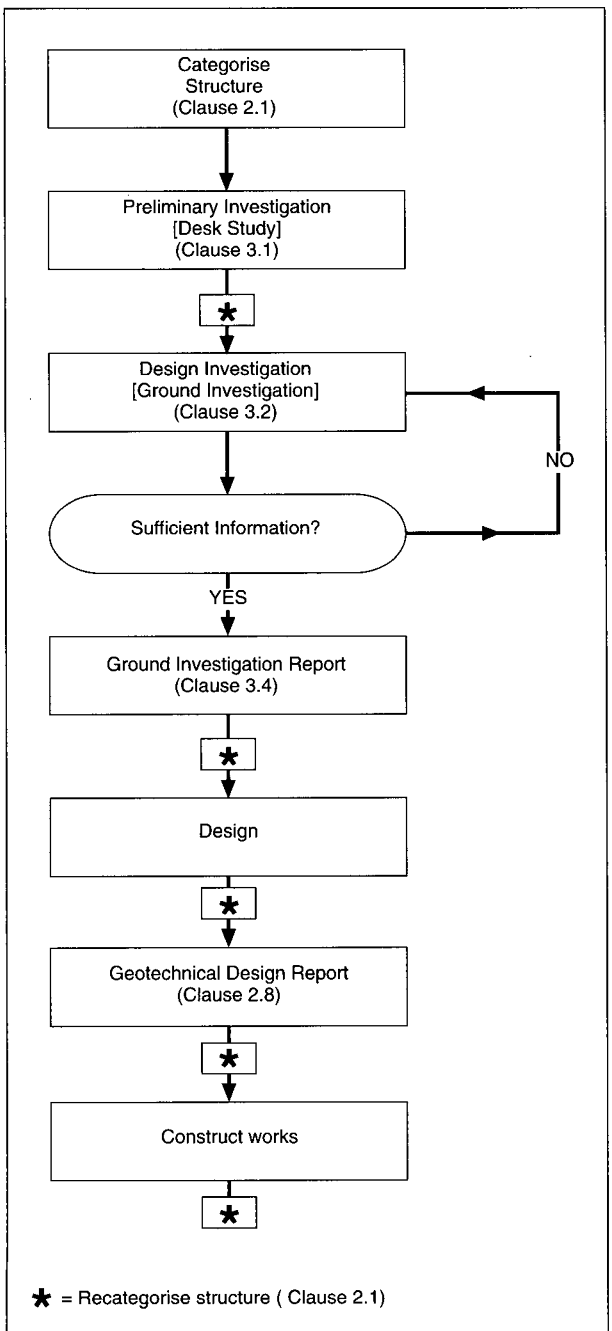


Figure B1.1 The EC7 design process

B1.2 Geotechnical categories

A flow diagram illustrating the recommended route through geotechnical design to EC7 is shown on Figure B1.1.

Clause 2.1 introduces the concept of Geotechnical Categories. EC7 divides structures into Geotechnical Categories 1, 2 or 3 according to a number of geotechnical design requirements, principally related to the complexity of the structure and previous experience of the particular ground conditions. Most engineered structures will fall in category 2, whilst very simple designs may be in Category 1 and complex problems fall into Category 3; Figure B1.2 is a flow diagram showing the decisions required in categorisation. The categories are used in the code to indicate the degree of effort required in site investigation and design. In C2.1, it is suggested that the categories also indicate the qualifications of the personnel required for the work.

Categorisation is not a mandatory part of the code, all reference to it being in application rules rather than principles. Concern has been expressed, particularly by foundation contractors, about its legal implications.

The intention is that a preliminary classification of a structure according to geotechnical category should normally be made prior to the geotechnical investigations. The category should be checked and possibly changed at each stage of the design and construction process, as indicated by the asterisks in Figure B1.1. The procedures of higher categories may be used to justify more economic designs, or where the designer considers them to be appropriate.

B1.3 Safety format

In common with all the Eurocodes, EC7 is based on the principles of limit state design. The application of this in geotechnical engineering is discussed in B2. Calculations are principally to be carried out by applying partial safety factors to characteristic values of soil parameters, which are discussed further in B4. Design is not entirely based on calculation, however, and use of observation and testing is also encouraged, as noted in B3.

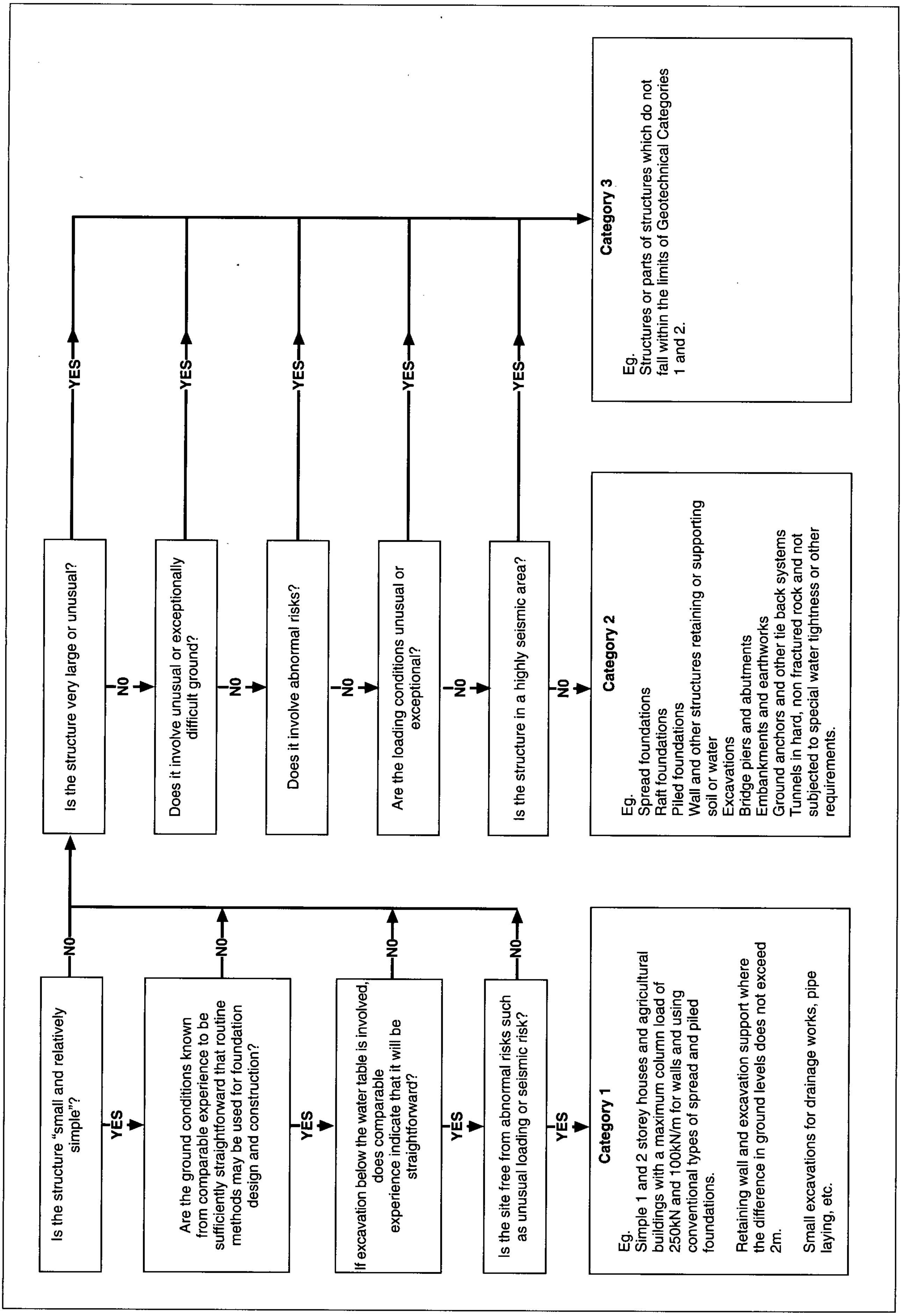


Figure B1.2 Geotechnical categorisation

B1.4 Geotechnical investigation

In Section 3, EC7-1 provides outline requirements for geotechnical site investigation, dividing the activities into **preliminary** and **design** investigations. The preliminary investigation (EC7, 3.1 and 3.2.2) corresponds to what is traditionally referred to in the UK as a 'Desk Study'. It identifies the ground related hazards which the structure will face both during construction (eg the need for dewatering) and in the permanent condition (eg the need to resist whatever onerous combination of loads is most critical). The design investigation (EC7, 3.2.3) corresponds to what used to be called in the UK 'Site Investigation' and is now more properly referred to as the 'Ground Investigation'. It investigates the hazards identified in the preliminary investigation and produces design parameters which are appropriate for the geotechnical category. This process is illustrated in Figure B1.1.

The code requires that the final design is accompanied by formal reports, both factual and interpretative, of the investigations on which it is based. The contractual issues which this raises are discussed in C3.4. The factual report on the design investigation is incorporated in the 'Ground Investigation Report' (EC7, 3.4) which also includes evaluation and interpretation of the data. The Ground Investigation Report can be included in the 'Geotechnical Design Report' (EC7, 2.8) which also includes design assumptions, design calculations and the plan for site supervision and monitoring required by the design.

Clause 3.3 also gives requirements for the process of evaluating the main ground parameters used in calculations (generally characteristic values for EC7). Methods of carrying out field and laboratory tests are not described in EC7 Part 1, but in Parts 2 and 3 (see A2.5).

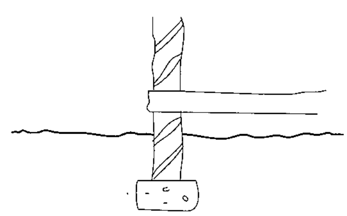
Job Title 'New start housing development Structure Reference: Strip foundations	Job No. / Sheet no of.............. Made by: Date .... Checked by: Date .... Approved by: Date ....
Reports used: Ground Investigation report (give ref. date) Factual: Bloggs Investigations Ltd report ABC/123 dated 21 Feb 95 Interpretation: Ditto	Section through structure showing actions: 
Codes and standards used (level of acceptable risk) Eurocode 7 Local building regs	Assumed stratigraphy used in design with properties: Topsoil and very weathered glacial till up to 1 m thick, overlying firm to stiff glacial till (c_u 60 kPa on pocket penetrometer).
Description of site surroundings: Formerly agricultural land. Gently sloping (4°)	
Calculations (or index to calculations) Characteristic load 60 kN/m. Local experience plus Local Building Regulations (ref ....) indicates working bearing pressure of 100 kPa acceptable. Therefore adopt footings 0.6 m wide, minimum depth 0.5 m (Building Regs) but depth varies to reach c_u 60 kPa – test on site.	Information to be verified during construction. Notes on maintenance and monitoring. Concrete cast on un-softened glacial till with c_u 60 kPa (pocket penetrometer)

Figure B1.3 Single page geotechnical design report

B1.5 Design procedures

The later sections of EC7-1 consider the design of some specific types of foundations and other geotechnical structures. The code generally does not specify the precise form of calculations to be used, but states what criteria are to be checked by the calculations.

Clause 2.8 requires that assumptions, data, calculations and results of the verification of safety and serviceability must be recorded in a Geotechnical Design Report. The level of detail of Geotechnical Design Reports will vary greatly, depending on the type of design. For simple designs, a single sheet may be sufficient. An example of such a sheet is given in Figure B1.3.

The scope of the report includes a plan of supervision and monitoring, as appropriate, and the clause requires that relevant parts of the report must be provided to the client.

B2 LIMIT STATE DESIGN

B2.1 Definitions

Limit state design is a procedure in which attention is concentrated on avoidance of limit states. Limit states are defined as 'states beyond which the structure no longer satisfies the design performance requirements' (EC1, 3.1(1)P). Strictly, it is the **exceedence** of a limit state which is not acceptable, though EC7 often refers to avoiding the **occurrence** of a limit state.

This definition of limit states is essentially practical and relates to the possibility of damage, economic loss or unsafe situations. It is not directly concerned with states of stress in materials or distinctions between elastic and plastic behaviour, though designers may need to consider these in order to demonstrate that limit states will not be exceeded.

Limit state design is concerned with **any** state in which a structure does not satisfy the design performance requirements. For example, cracking or distortion which has no more consequence than giving a disappointing appearance constitutes a limit state, just as does a catastrophic collapse. The severities of these two limit states are obviously very different.

It has been found convenient to categorise limit states as **ultimate** or **serviceability** limit states. EC1 defines ultimate limit states as *those associated with collapse or with other similar forms of structural failure* (3.2(1)P). Serviceability limit states *correspond to conditions beyond which specified service requirements for a structure or structural element are no longer met* (3.3(1)P). The serviceability requirements *should generally be determined in contracts and/or in the design* (3.3(4)).

EC1, Section 3 adds more detail to this description of ultimate and serviceability limit states. For geotechnical design, it is important to note that ultimate limit states include *failure by excessive deformation, ... loss of stability of the structure or any part of it.* Hence, a state in which part of a structure becomes unsafe because of foundation settlement or other ground movements should be regarded as an ultimate limit state, even if the ground itself has not reached the limits of its strength, to form a plastic failure mechanism. For example, large amounts of heave of plastic, over-consolidated London Clay have occurred over long periods following the removal of trees. While there is no question of the ground strength having reduced, to the degree that bearing capacity failure is approached, the movements have been large enough to induce collapse in a building, following loss of bearing in lintels over window and door openings. The application of this concept to retaining walls is noted in EC7, 8.4(2).

Limit states are generally checked by considering **design situations,** in which adverse conditions apply; **design values,** which are deliberately pessimistic, are used for both loads and material strengths. Design values are used in calculations for both ultimate and serviceability limit states, though the values will usually be different for the two states. The Eurocodes specify how design values are to be derived. The design values required for serviceability limit states are often equal to the characteristic values of parameters (formally, a partial factor of 1.0 is applied), but there is no fundamental reason why this must always be so. EC7 states that SLS design values will normally equal characteristic values for actions in 2.4.2(18) and for materials in 2.4.3(13).

B2.2 Basis of the method

The definitions considered above show that limit state design is concerned with what might go wrong. Attention is concentrated on states which it is intended will not occur rather than on what, it is hoped, will actually happen. Occasional exceedence of serviceability limit states might be economically tolerable, but generally ultimate limit states must be avoided. Thus the design should have *appropriate degrees of reliability* (EC1, 2.1(1)P).

The aim of limit state design is to avoid limit states in general, and to make very remote any possibility of an ultimate limit state. Ultimate limit states are intended to be **unrealistic** possibilities. Hence, in calculations, the codes sometimes require the adoption of design values for parameters which are unrealistically pessimistic.

It may be questioned whether there is anything distinctive about limit state design, or whether the definitions are so broad that they incorporate all design processes. This is particularly relevant in geotechnical design, where, historically, there has been more consideration of plastic failure mechanisms – undesired states – than on working states of elastic stress. The distinctive feature of limit state design is essentially one of emphasis, with attention concentrated on what might go wrong.

Limit state design is sometimes contrasted with **permissible stress design** in which attention is concentrated on prediction of the stresses in materials in the intended working state. This terminology becomes confused if **permissible stresses at the limit states** are considered – and there is no logical reason why these should not be used. Hence it is preferable to contrast limit state design with **working state design.**

Some of the pros and cons of limit state design have been discussed by Simpson (1997).

B2.3 Design procedures

The basic limit state design procedure has two stages:

a set up design situations;
b show that limit states will not be exceeded in the design situations.

EC1, Clause 2.3 states: *The selected design situations shall be sufficiently severe and so varied as to encompass all conditions which can reasonably be foreseen to occur during the execution and use of the structure.* Design situations are categorised as persistent, transient and accidental situations, and the limit states relevant to the various situations may vary. For example, for an accidental situation, which involves exceptional conditions, the structure may be required merely to survive without collapse; in this case serviceability limit states would not be relevant. More information on design situations may be found in EC1, 2.3 and EC7, 2.2.

The limit state method does not restrict the means by which it may be demonstrated that limit states will not be exceeded in the design situations. Often, calculations will be used for this purpose, but other approaches provide alternatives or supplements to design by calculation. These include load testing at full scale or on models, which is particularly relevant to design of piles and ground anchors (EC1, Section 8 and EC7, 2.6); prescriptive measures, in which well-established details are adopted without calculation (EC7, 2.5); and the Observational Method (EC7, 2.7). These methods are discussed further under the relevant clauses in Part C.

The definition of serviceability limit states often requires the specification of a limiting value of displacements or strain. It is essential that this is realistically assessed as values representing an unacceptable condition. Unnecessarily severe values may lead to highly uneconomic design.

B2.4 Calculations

Historically, the limit state method became popular at about the time that partial safety factors began to be adopted. The two are therefore often linked, though there is no fundamental connection between them. A calculation using a global factor of safety or directly assessed pessimistic design values could be sufficient to demonstrate that limit states will not occur. It was noted above that the limit state method does not necessarily require calculations as the basis of design.

Limit state calculations are usually carried out by showing that the **design** properties of materials are sufficient to withstand the **design** values of all applied actions (ie loads – see A2.8). The design values generally incorporate all the required safety elements, with no further overall factor of safety. Generally, design values of parameters, X_d, are derived from **characteristic** values, X_k, by applying partial factors γ:

for actions: $F_d = F_k \times \gamma$

for materials: $X_d = X_k / \gamma$

The derivation of design values by applying partial factors to less pessimistic characteristic values provides a means by which codes of practice can exert some influence over the degree of pessimism of the design values. The concept of 'characteristic' values in geotechnical engineering is discussed in B4.

The limit state design method requires that all possible limit states are considered and eliminated, with 'appropriate degrees of reliability'. In general, this will at least mean that ultimate and serviceability limit states must be considered. For geotechnical design, this puts an increased emphasis on the need to consider deformations, but EC7 aims to discourage excessive or spurious attempts to calculate displacements.

The purpose of partial factors is generally stated to be to allow for uncertainties and inaccuracies in the values of the parameters (EC1, 9.3.1 and 9.3.3). Some authorities deduce from this that the values of the partial factors may be derived from statistical studies of these uncertainties. In this approach, the factors used for ULS design have no bearing on the serviceability of the structure. This contrasts with the use in BS 8002 of a 'mobilisation factor', which is effectively a partial factor, but its stated purpose is to prevent stress levels in materials reaching a point at which displacements become unacceptable; that is, the factor's role is mainly in serviceability.

Referring to partial factors on actions, Eurocode 1 takes a broader view in 9.4.3(3): *The values have been based on theoretical considerations, experience and back calculations on existing designs.* The calibration of the ULS factors based on experience and back calculations will necessarily mean that their values make some provision for serviceability as well as ultimate requirements. EC7 notes in 2.4.1(7) that in some situations it is necessary to use factors applied in the analysis of one limit state in order to cover another, for which calculations are not reliable.

In the authors' opinion, this pragmatic approach to the use of partial factors is realistic. The factors adopted are inevitably calibrated against previous designs and therefore make some provision for serviceability as well as ultimate safety. Where EC7 makes additional requirements for checks on serviceability, these should be followed, however.

B3 DESIGN BY CALCULATION, PRESCRIPTIVE MEASURES, TESTING AND THE OBSERVATIONAL METHOD

The fundamental design requirements for limit state design are set out in EC7, 2.1. In 2.1(7), it is stated that these requirements may be achieved by:

a use of calculations;
b adoption of prescriptive measures;
c experimental models and load tests; or
d an observational method.

It is clear that design based on calculation is not the only process envisaged. The same paragraph says that these four approaches may be used in combination. This often forms the basis of good geotechnical engineering.

Prescriptive measures (EC7, 2.5) *involve conventional and generally conservative details in the design, and attention to specification and control of materials, workmanship, protection and maintenance.* They may be used *when calculations are not available or not necessary.* They could be used for design for durability, for example, and will often be based on the observed performance of existing structures. More generally, they might be used to make a quick, conservative design in cases where the cost of extensive site investigation and analysis cannot be justified. In Hong Kong, the Geotechnical Engineering Office is preparing to publish a series of recognised prescriptive measures for stabilisation of small slopes, for example.

Design of piles and ground anchors has traditionally been based very largely on load testing. This is in the category *design by experimental models and load tests,* in which confidence in the safety of the design depends on test results, either in place of or in combination with calculations. This use of test results in design is discussed further in EC7, 2.6, 7.5–7.7 and 8.8, and in E7, E8 and E12. EC7 mentions the use of model testing in 2.6, but does not enlarge on this.

EC7, 2.7 is a specific clause about the Observational Method, which has received a great deal of support from the geotechnical community. It is used in recent publications Safety of New Austrian Tunnelling Method (NATM) Tunnels, by HSE and the CIRIA report on the Observational Method (Nicholson et al (1997)). Since the Observational Method relates mainly to the design of temporary works, it is considered further in B6.

B4 CHARACTERISTIC VALUES

B4.1 Significance

Characteristic values of geotechnical parameters are fundamental to all calculations carried out in accordance with the code. Their definition, in geotechnical terms, has been the most controversial topic in the whole process of drafting Eurocode 7. Some of the more difficult issues will be addressed here. More straightforward matters will be left to Part C. The most important text is in EC7, 2.4.3.

Two factors underlie the importance and controversy of characteristic values.

a Calculations are to be carried out by applying partial safety factors to characteristic values in order to obtain **design values** of parameters. The partial factors are specified by the code, so the selection of characteristic values is the main point in calculations at which engineers are to apply their skills and judgment, with the possibility of dangerous mistakes.

b Engineers have always had the responsibility for selecting values of material parameters for calculations. This process has sometimes been referred to as a 'black art', and it is difficult to find helpful advice on the thought processes necessary to derive appropriate values from site investigation and other information. In particular, the degree of conservatism necessary in choosing values for design purposes is rarely discussed.

Eurocode 7's definition of characteristic values is intended to make full use of the skills and judgment of experienced engineers, whilst helping less experienced engineers to choose values which are both reasonable and safe. This was, and remains, a major challenge.

B4.2 Characteristic values in Eurocode 1 and in structural design

Characteristic values, as used in Eurocode 7, are intended to comply with Eurocode 1 as far as possible, whilst remaining true to principles of sound geotechnical engineering. Although it arguably remains within the spirit of Eurocode 1, the definition adopted for geotechnical purposes differs from that of Eurocode 1 in some important respects. To understand this, it is necessary first to consider what Eurocode 1 says about characteristic values, X_k.

Eurocode 1, Subclause 9.3.3 states:

The design value X_d *of a material or product property is generally defined as:*

$X_d = \eta X_k / \gamma_M$ *or* X_k / γ_M

where:

γ_M *is the partial factor for the material or product property, given in ENVs 1992 to 1999, which covers:*

- *unfavourable deviations from the characteristic values;*
- *inaccuracies in the conversion factors; and*
- *uncertainties in the geometric properties and the resistance model.*

η *is the conversion factor taking into account the effect of the duration of the load, volume and scale effects, effects of moisture and temperature and so on.*

Characteristic values are introduced in Eurocode 1 Section 5 thus:

(1)P Properties of materials (including soil and rock) or products are represented by characteristic values which correspond to the value of the property having a prescribed probability of not being attained in a hypothetical unlimited test series. They generally correspond for a particular property to a specified fractile of the assumed statistical distribution of the property of the material in the structure.

(2) Unless otherwise stated in ENVs 1992 to 1999, the characteristic values should be defined as the 5% fractile for strength parameters and as the mean value for stiffness parameters.

Note: For operational rules, see annex D, for fatigue, information is given in annex B.

(3)P Material property values shall normally be determined for standardized tests performed under specified conditions. A conversion factor shall be applied where it is necessary to convert the test results into values which can be assumed to represent the behaviour of the material in the structure or the ground (see also ENVs 1992 to 1999).

This text specifies the following features:

a Characteristic values take account of the statistical distribution of the property. That is, the range of uncertainty of the property is relevant to their selection.

b They can normally be derived by a statistical process applied to a series of tests on specimens of the material. However, in principle they relate to *a hypothetical, unlimited test series*, so some correction may be required when test series are limited.

c For strength properties, they are to correspond to the 5% low fractile of the test results; this is the strength below which 5% of test results fall.

d Nevertheless, the characteristic values are said to *represent the behaviour of the material in the structure or the ground*, and corrections to test results may be needed in order to achieve this.

e For stiffness, mean values are to be used. This is considered further in B4.12.

These definitions of characteristic value are clearly intended to be general. Eurocode 1 does not at this point mention the mode of failure or type of limit state being discussed, or the severity of its consequences.

In structural design, characteristic values are generally defined using statistical procedures applied to the results of tests on material specimens. The specimen is generally not obtained from the structure and its relationship to material in the structure depends more on control of workmanship than on the designer's observation or judgement. In this respect, the definition of characteristic value for ground materials given in Eurocode 7 is distinctly different.

B4.3 Characteristic values used in geotechnical design

In Eurocode 7, the characteristic values of geotechnical material parameters are based on an assessment of the material actually in the ground and the way that material will affect the performance of the ground and structure in relation to a particular limit state (EC7, 2.4.3(2,3,4)). Field and laboratory tests are to be used, but they are only one means of assessing what is in the ground; characteristic values are not derived directly or solely from the test results. Statistical manipulation of test results will generally have only a minor role in this process, if any. The resulting value is inevitably subjective to some extent, being influenced by the knowledge and experience of the designer. However, this is considered preferable to an alternative, mechanical approach which has arithmetic objectivity but jettisons established engineering knowledge.

In many situations, the known geology of a stratum, and existing experience of it give a fairly good indication of its parameter values. Soil tests are used as a check. It is good practice to base the selection of characteristic values on a combination of well established experience and the test results (EC7, 2.4.3(2,4)). If unusually good test results are obtained, engineers will normally spot this and treat them with greater caution, unless further investigation is possible to establish that they are relevant. Unusually bad results may lead to further investigation, or may otherwise be taken at face value unless the evidence of other experience is overwhelming.

Construction activities may affect the properties of the ground, adversely or beneficially (EC7, 2.4.3(4)). Common examples occur during boring or driving of piles, or excavating to a level on which concrete will be cast. In many cases this will occur after any investigation and testing are complete. Nevertheless, the characteristic value is to account for these construction effects. Information from previous experiences and publications will contribûte to the selection of characteristic values in these circumstances.

Having reviewed these items, EC7 says that *the characteristic value of a soil or rock parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state* (EC7, 2.4.3(5)). This is standard engineering practice. The relationship of the 'cautious value' to mean values will be considered in B4.9 to B4.11 below.

B4.4 Characteristic values dependent on failure mode

The characteristic value of one parameter in one stratum is not necessarily the same for two different failure modes. It may depend on the extent to which a particular mode averages out the variable properties of the stratum (EC7, 2.4.3(4, 6)).

Figure B4.1 shows a small industrial building, founded on pad footings near a long slope. The underlying materials are estuarine beds, mainly of sands with some impersistent lenses of clay occurring at random. In this type of situation, the design of the footings would probably assume that they might be founded on clay, the more adverse condition for foundation design.

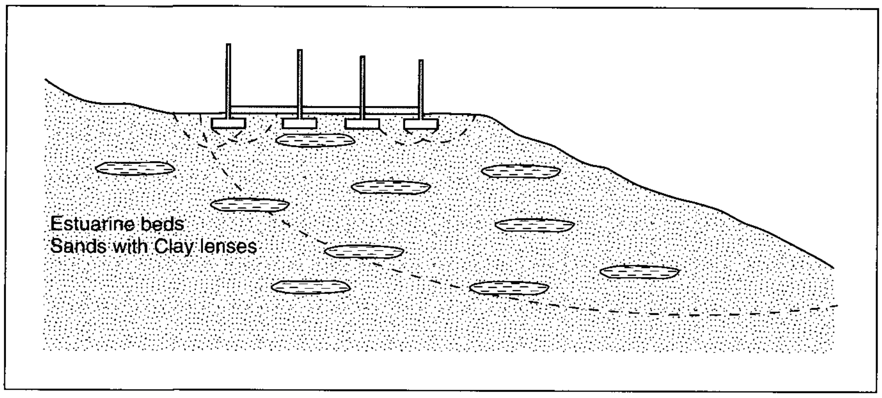


Figure B4.1 Small building on estuarine beds near slope

(An alternative could be, in some cases, to require an inspection and probe at each footing, so avoiding this adverse condition.) On the other hand, when the possibility of a large slip surface is considered, it is inconceivable that this will lie entirely, or even mainly in clay. In this type of situation, the characteristic values for strength parameters of the beds would be different for the footing design and for the slip, though their safety is controlled by the same stratum in both cases.

Figure B4.2 shows results of a CPT test in a mixed, estuarine deposit which has been overconsolidated, variably, by desiccation. A piled foundation is to be constructed in this material. If the piles are of fixed length (perhaps limited by construction equipment), the characteristic values of soil strength for the base and shaft may be quite different. The shaft averages the properties of a large amount of material, from many periods of deposition, whilst the base could be formed in one of the weaker layers. In this case the characteristic values of soil strength for the shaft would be higher than that for the base, in the same deposit. On the other hand, if the construction process allows the base to be tested, by pile driving for example, the characteristic value for the base could be higher that the averaged value used for the shaft. This discussion must also be modified to take account of any systematic variation of strength with depth.

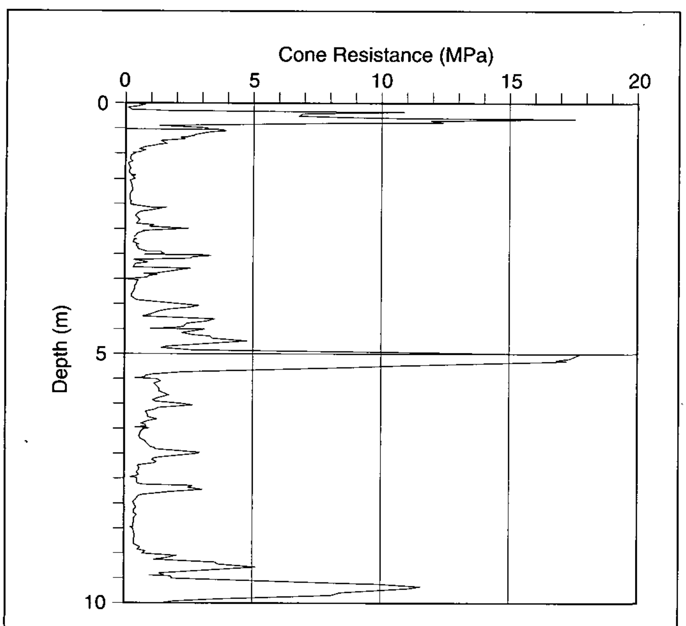


Figure B4.2 CPT results in variable deposit

B4.5 Which value – peak, critical state, residual, mobilised ...?

The question has been asked: **Which** *value is the characteristic value?* It is sometimes necessary to chose from one of the following, depending on circumstances:

a peak, critical state or residual shear strength;
b ultimate strength or a 'mobilised' value;
c strength of intact material or strength on joints;
d strength at first loading or after repeated loading;
e stiffness of intact rock or of the jointed material;
f stiffness on first loading, or on unload-reload.

In all cases, the answer of Eurocode 7 is: *the one that is relevant to the prevention of the limit state under consideration.* EC7 does not differ in this respect from normal practice. For some particular situations, the code is able to specify which of these values is relevant. For example, where concrete is to be cast against ground, which might therefore be disturbed, the critical state value for the angle of shearing resistance is required (EC7, 8.5.1(4)). In considering rocks, a study of the joint patterns will determine whether intact or joint strength is relevant (EC7, 3.3.9).

This answer to the question is not the same as: *the one which would become relevant if the limit state was not prevented.* For example, in most plastic clays, if a slip occurred, the angle of shearing resistance would eventually fall to the residual value. Nevertheless, it is not necessary to design for residual strength in clays which have not previously slipped. Similarly, it may be unnecessary to design for critical state values, though brittleness and ductility must be considered, as noted in EC7, 2.1(9) and C2.1.

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

B4.6 Relationship to other texts and practices

With regard to characteristic values, the intention of the drafters of EC7 was to clarify existing practice, rather than to introduce something new. The main problem was the difficulty of defining existing practice. Nevertheless, some texts give helpful indications of the way in which parameter values are to be chosen and it is relevant to compare these with characteristic values.

CIRIA Report 104 suggests that design may be based on *moderately conservative values* of parameters. 'Moderately conservative' is defined (p 40) as meaning *conservative best estimate.* It could be objected that the latter term is contradictory, since a value cannot be both conservative and a best estimate simultaneously. CIRIA 104 states that *this approach is used most often in practice by experienced engineers.* The authors consider that the *conservative best estimate* values of CIRIA 104 are essentially the same as the *characteristic* values of EC7.

In BS 8002, design values of soil strength (ie values entered into calculations) are derived by factoring **representative** values. For effective stress parameters, there is a further requirement that the design value must not exceed the representative critical state value. A representative value is defined (1.3.17) to be a *conservative estimate of the mass strength of the soil.* 'Conservative values' are further defined (1.3.2) as *values of soil parameters which are more adverse than the most likely values. They may be less (or greater) than the most likely values. They tend towards the limit of the credible range of values.* The authors suggest that this definition makes representative values essentially the same as moderately conservative values in CIRIA 104 and characteristic values in EC7.

The Dutch standard NEN 6740 (in Dutch) provides a more statistical approach to derivation of characteristic values. German recommendations for waterfront structures (EAU (1980, p38)) discuss the statistical background to characteristic values, and also provide some more pragmatic suggestions: *When a large number of shear parameters have been determined, the characteristic value can also be estimated as being that value which occurs immediately below the* *mean of all tests made ... With only three determined values, which have been obtained from three samples of the investigated layer taken at well separated locations, the lowest value may also be used as the characteristic value if the values do not differ too much from one another.*

B4.7 Why are structural and geotechnical characteristic values different?

The designer of a structure is concerned with the properties of materials which generally do not exist at the time of design, but which can be specified with fair precision. The range of uncertainty of their properties is fairly well known, and, in many cases, may be better understood by the drafters of codes than by designers in practice. Hence, it is appropriate that codes give specific rules about the measurement of characteristic values and that the possible range of uncertainty is entirely accommodated in factors prescribed by the code writers.

In geotechnical design, however, the designer is in possession of information not available to the code drafters. He knows where the site is located, what is its geology, and he has test results, relevant publications, observations of nearby constructions, and so on. The designer is therefore in a much better position than the code drafter to make allowance for the range of uncertainty of the parameter values. It is this extra information which Eurocode 7 requires the designer to incorporate in his selection of characteristic values.

B4.8 Relationship to mean values

EC7 says that the characteristic value of a soil or rock parameter shall be selected as a **cautious** estimate of the value affecting the occurrence of the limit state (EC7, 2.4.3(5)). The probability that the characteristic value will, in fact, prevail in such a way as to govern the occurrence of a limit state is fairly remote, nominally 5%.

It has been suggested that the characteristic value should be defined to be a **mean** value. Unfortunately, there is some confusion about different meanings of the word 'mean'. For the purpose of this discussion, three mean values will be defined: statistical, spacial and probabilistic.

a A **statistical** mean will be taken to be the simple average of established data. These could typically be test results, adjusted where necessary to allow for differences between the test and field situation.

b A **spacial** mean is the average of a parameter over some space. This could be the volume which is compressed under a load or the surface over which a slip might occur. Many limit modes are governed by the average performance of such a volume or surface, and for these a spacial mean of the parameter value is appropriate. The decision to use a spacial mean does not dictate the degree of pessimism which may be attached to the chosen value.

c A **probabilistic** mean is a value, taken from a range of uncertainty, such that the value which will actually be found to govern the limit mode has a 50% chance of being worse than the probabilistic mean. Most often, this probability must be assessed by the engineer in advance of the actual events. One advantage of using a probabilistic mean is that it is equal to the statistical mean value of a set of relevant test results, provided they have been adjusted for any difference between the behaviour of the soil in test and in situ.

In many situations, the characteristic value required by EC7 should be a cautious assessment of a spacial mean. If there is to be, in fact, a 5% chance that a worse value will govern field behaviour, then the cautious spacial mean will be much less pessimistic than the 5% fractile of relevant, adjusted test results. This reflects the fact that many limit modes average out the variabilities of a lot of ground.

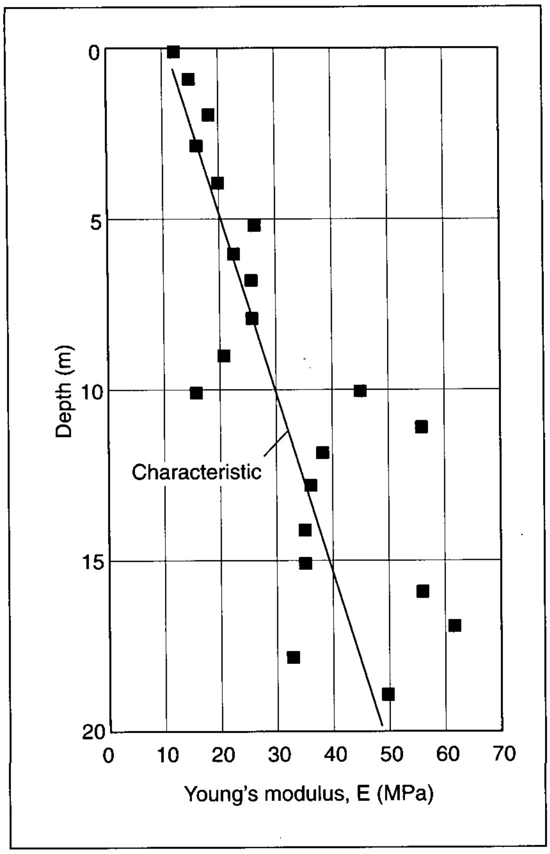


Figure B4.3 Profile of Young's modulus measurements

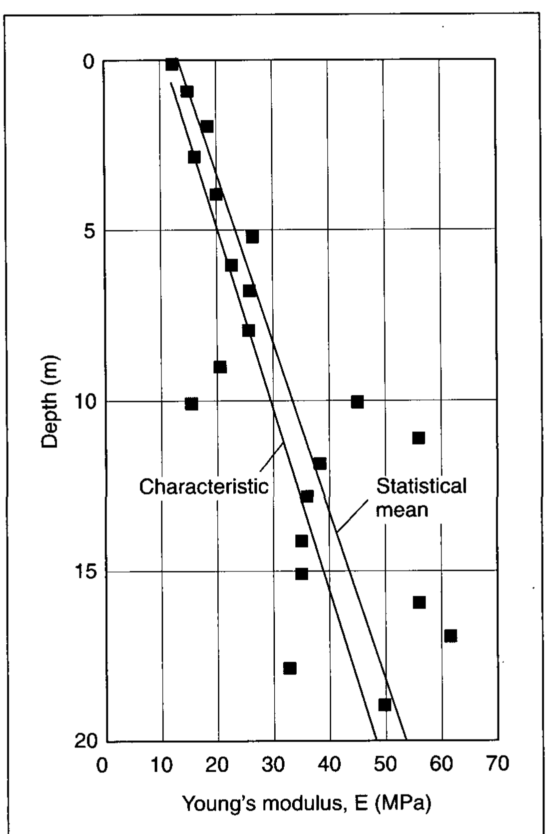


Figure B4.4 Characteristic and statistical mean lines

Figure B4.3 shows the results of a series of test results from which a Young's modulus is to be obtained for calculation of settlement beneath a foundation. There is a clear increase of stiffness with depth, but the designer has checked that there is no systematic variation of the test results with position on the site, so the variations shown can be treated as random. The foundation will load a large volume of ground and it is unreasonable to assume that its settlement could be determined by material from the lower end of the range of variation, such as the lowest 5% fractile. A suggested profile representing a cautious mean is shown on the figure. This could be used as the characteristic stiffness, varying with depth.

Figure B4.4 shows the same data and characteristic profile as Figure B4.3, with the statistical mean, obtained by linear regression, added. It can be seen that these two are fairly similar in this case. In general, where the range of possible parameter values is narrow, it will be acceptable to adopt a statistical mean as the characteristic value, the cautious spacial mean. However, where the range of values which could govern the limit mode is large, the cautious spacial mean should be more pessimistic than the statistical mean.

Characteristic values for stiffness parameters are considered further in B4.12.

There are some situations in which spacial means are not relevant, or, at least, must be chosen so specifically that they are not easily recognised as means. For example, if stability of a rock cutting is being considered, the mean strength of the rock may be of no relevance: what is needed is the strength along joints which are inclined towards the cutting. It could be argued, of course, that it is the (spacial) mean strength of the joints which is needed. Similarly, for a small foundation or the base of a pile, the mean strength of the stratum may be irrelevant if there is a possibility that the small zone of soil affecting the foundation is less strong. In cases such as these, the characteristic value is considerably more pessimistic than the statistical (or probabilistic) mean for the stratum as a whole.

One alternative approach, which defines the characteristic value as the statistical mean (or possibly the probabilistic mean), might yet find favour in the Eurocodes. In this, the designer is given some discretion in the value of the partial factors, depending on his assessment of the uncertainty of the parameter.

B4.9 Significance of statistical methods

EC7 states that statistical methods may sometimes be helpful in assessing characteristic values, but that they *should allow a priori knowledge of comparable experience with ground properties to be taken into account for example by means of Bayesian statistical methods* (EC7, 2.4.3(6)). This demands a high order of statistical technique, available from very few designers who have committed their time to training and experience in geotechnical engineering. Attempts by statisticians to tackle geotechnical design have often ended in ridicule, and it is very difficult for one person to have a sufficient grasp of both disciplines that he can combine them sensibly.

Nevertheless, some pointers to more general rules for assessment of data might be obtained from statistical analysis. Schneider (1997) has proposed that, where a spacial mean is relevant, the characteristic value might be taken as half a standard deviation from the mean. Trials, like the one described in B4.11 below, suggest that this rule could be a useful guide.

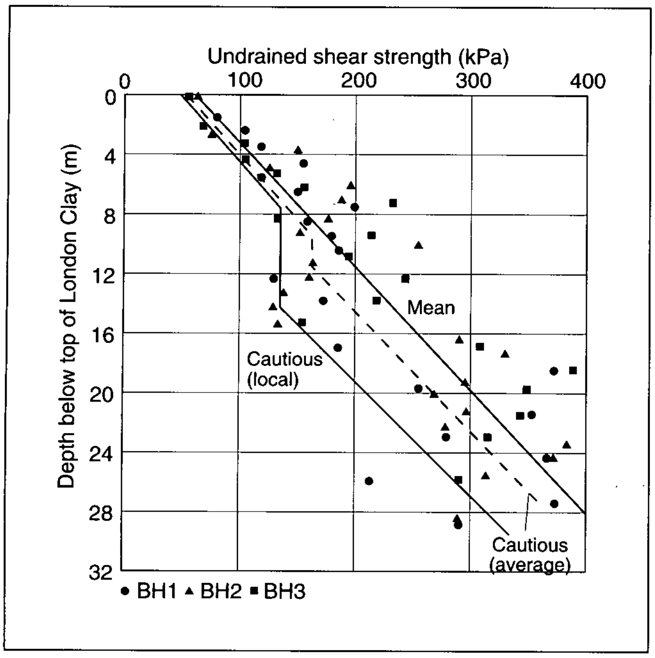


Figure B4.5 Undrained shear strengths measured at one site using 100 mm diameter UU triaxial tests

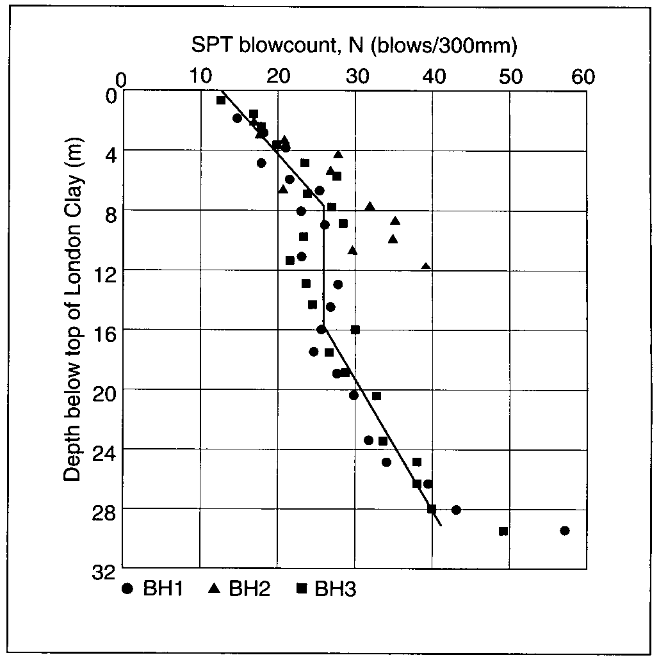


Figure B4.6 SPT results from tests at the same site

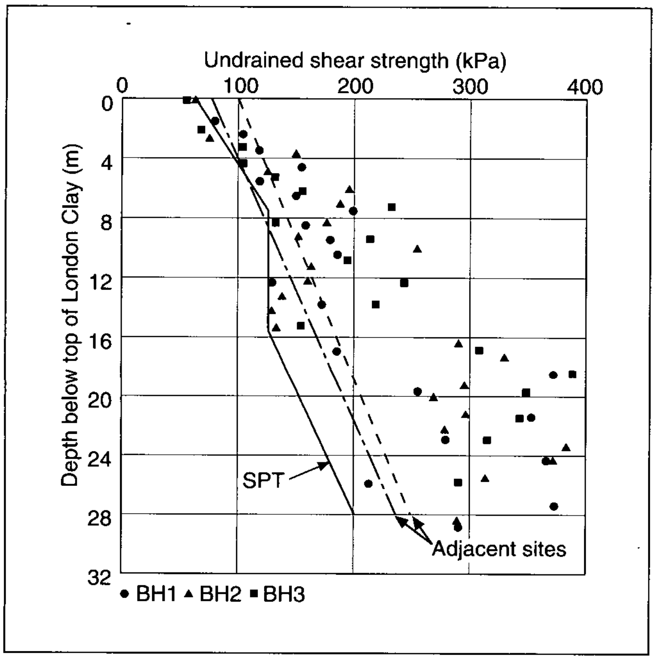


Figure B4.7 SPT results and results from adjacent sites compared with the UU triaxial results

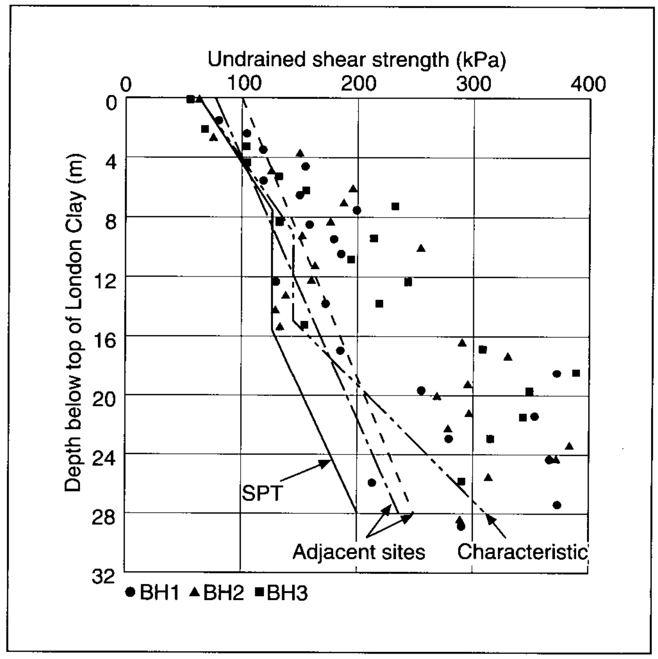


Figure B4.8 An assessment of a 'characteristic' profile of undrained strength

For some specific problems, statistical studies have proved very useful. Studies of correlation distances in slope stability problems (Vanmarcke (1977)) and of variability in cone penetrometer testing (Been and Jefferies (1993)) provide interesting examples.

B4.10 A London example

Figure B4.5 shows the results of a series of undrained shear strength measurements in London Clay. The measurements were made using unconsolidated undrained triaxial tests. A statistical mean line has been drawn through the data and it is clear that undrained strength increases with depth. A characteristic line is required, and this should depend on how the characteristic values will be used – what is the limit mode being considered? For example, if the undrained strength is needed for calculation of ground movements around a retaining wall, a value such as the 'cautious (average)' value shown on the figure could be used. However, for a problem in which failure might take place in a small zone of soil, such as an isolated foundation placed at a deep level, a more cautious value – the 'cautious (local)' value – should be adopted. From these boreholes, results from standard penetration tests were also available, as shown in Figure B4.6. In London Clay, there is usually a constant factor between standard penetration and undrained shear strength results; the factor is about 4.5 to 5. However, if the mean line from the SPT results is transferred onto the undrained strength plot, as in Figure B4.7, it appears that the normal correlation does not work. In fact, the measured undrained strengths are remarkably high: they are consistent with very low water contents, which were measured, but this might simply mean that the samples had dried out on the way to the laboratory, though there was no reason to suspect this. Figure B4.7 also shows lines representing mean values through data from other nearby sites, both for undrained shear strength and SPT results. The usual close correlation applies to these, and it is clear that the undrained strengths for the new site are remarkably high.

On the basis of these inconsistent data sets, what value should be used as the characteristic undrained strength? The values measured in the triaxial tests should not be ignored, but the SPT results and the data from adjacent sites should also affect the decision. The characteristic value proposed for these data is shown on Figure B4.8. This is less than the initial assessments in Figure B4.5, which were based on the triaxial results only, and is closer to a lower bound of this particular set of triaxial results.

Engineers often need to follow this sort of process when trying to interpret real data. It may be that statistical methods could trace a similar logical sequence. However, this would require quite advanced methods and any statistical approach which failed to take account of the diverse array of data, typically available, would be harmful to the design process.

B4.11 The Johannesburg experiment

Figure B4.9 shows a set of 'relevant' test results for the angle of shearing resistance of a soil stratum. In Johannesburg, an audience of geotechnical practitioners was told that there was no systematic variation of these results and the limit mode would average a large body of the material. Having been introduced to EC7, they were asked to assess the characteristic value they would adopt. The reader is invited to make his own assessment.

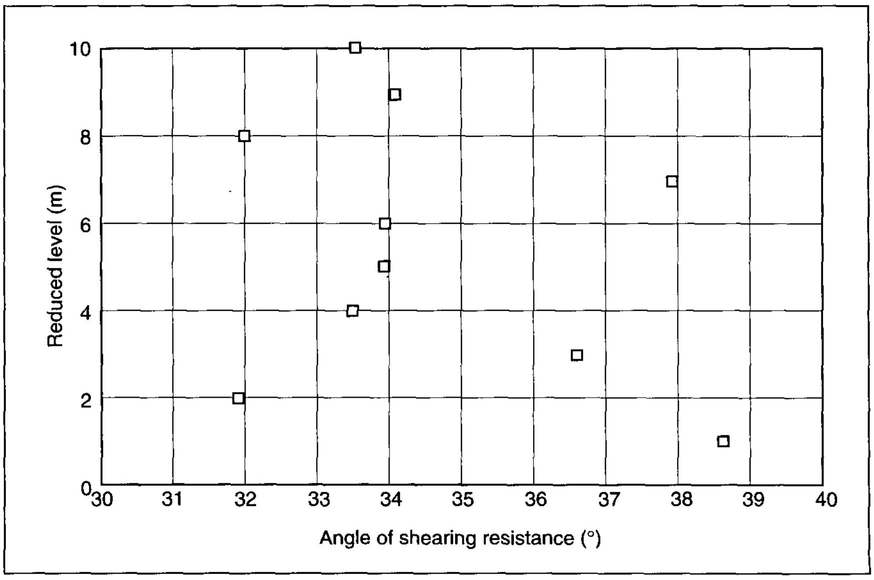


Figure B4.9 Measured angle of shearing resistance (1)

Figure B4.10 Assessment of characteristic value (1)

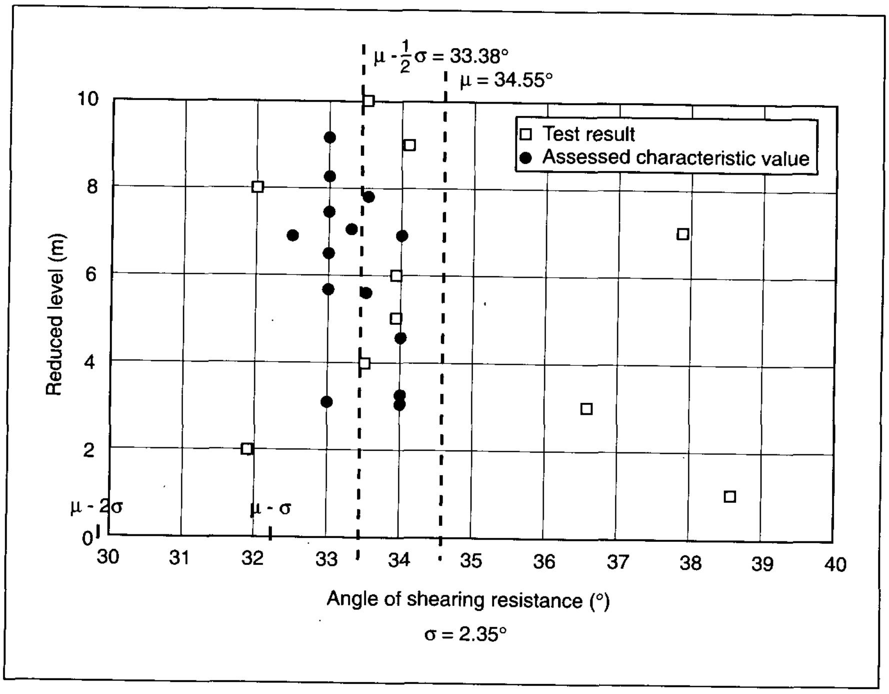


Figure B4.11 Measured angle of shearing resistance (2)

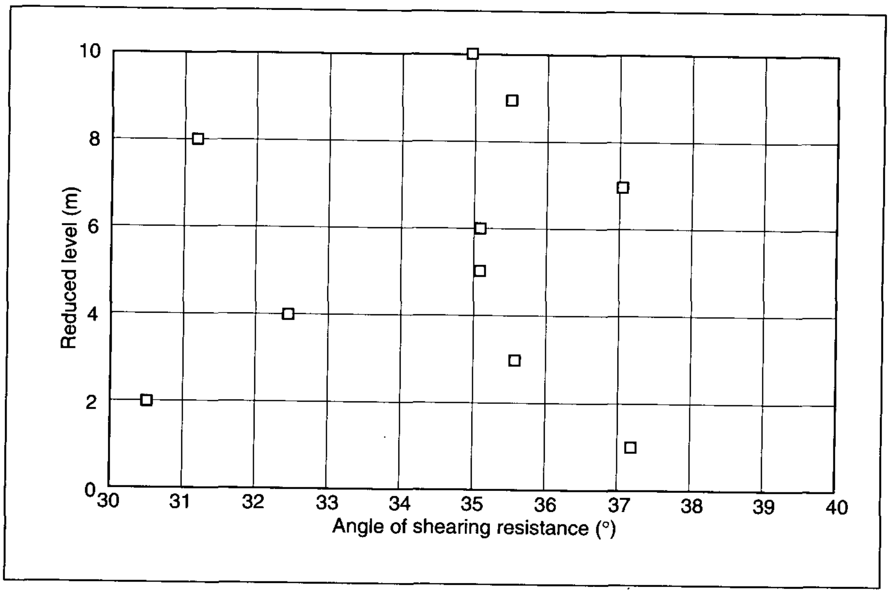
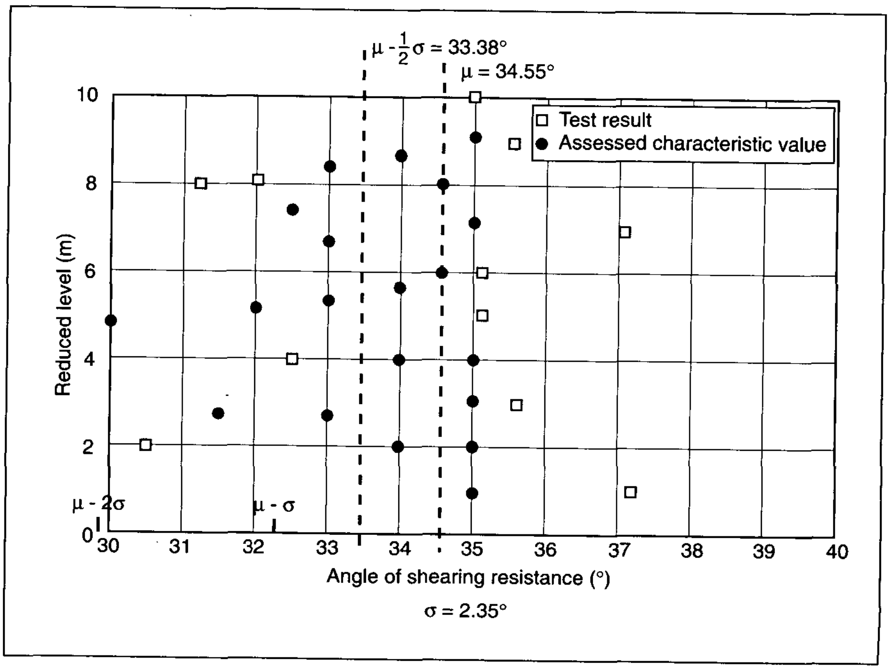


Figure B4.12 Assessment of characteristic value (2)



The assessments made in Johannesburg are shown on Figure B4.10, together with the mean and values half and two standard deviations from the mean. The assessments cluster around half a standard deviation from the mean, as suggested by Schneider. Comments from the audience were that they felt that the assessment they had made was essentially the same as they would previously have made in geotechnical design practice.

The same audience was then shown Figure B4.11. Again the reader is invited to make an assessment of characteristic value. The Johannesburg results, shown in Figure B4.12, were more uncertain in this case, despite the fact that the data had the same mean and standard deviation. Engineering experience was suggesting that the low values on this graph could not be dismissed so easily.

This experiment is considered to support the idea that 'half a standard deviation from the mean' is a useful guide, whilst also suggesting that a reliance on statistics alone would be dangerous.

B4.12 Characteristic values of stiffness and unit weight

It was noted in B4.2 that EC1, 5(2) defines characteristic values for stiffness to be mean values. The context of this definition is probably ULS structural design, in which values of stiffness are needed for analysis, but they rarely play a dominant part in determining the occurrence of a limit state.

In problems involving ground-structure interaction, however, the stiffness of the ground is often a very important parameter. In these cases, the use of a mean value for stiffness is questionable, since the calculations would then imply a 50% probability that the limit state would be exceeded, for the given design loads. If the limit state under consideration is a SLS for which displacements are being derived, partial load factors would be unity, so displacements calculated using a true 'mean value' stiffness would be best estimates, with no reserve of safety. In design practice, engineers rarely take this approach, preferring to make a more pessimistic estimate when there is significant uncertainty. EC1, 5(2) is under review in the light of the foregoing reasoning (early 1998).

EC7-1 therefore uses the same definition of characteristic value for stiffness as for strength. That is, a **cautious estimate**, not a mean value.

In assessing the occurrence of a limit state, a further parameter is involved, besides stiffness. This is usually a 'limiting value' of displacement or distortion (EC7, 2.4.6). It would, in principle, be possible to include sufficient conservatism in the specified limiting value that the actual occurrence of limit states could be made 'sufficiently improbable', even though mean values were used for stiffness. However, this would mean that the limiting values should be chosen in relation to the level of uncertainty of ground stiffness, which is somewhat irrational.

EC7's definition of characteristic value also applies to the unit weight of soil and rock. However, the uncertainty about unit weight is usually sufficiently low that there is no need to make a distinction between mean and cautious values. For fill materials behind retaining walls, special checks are required by 8.3.1.1. Variability of unit weight is considered further in B5.4.2.

B5 CASES A, B AND C

B5.1 Background

B5.1.1 Origins

The adoption, meaning and purpose of 'Cases A, B and C' have competed with characteristic values to be the most debated aspect of EC7. The cases are sets of partial factors to be applied to actions and to soil material properties. Their function is similar to the 'load cases' familiar to structural design, but their scope is extended to include factors on soil materials as well as loads. All three cases refer to design calculations for ultimate limit state, and values of the partial factors are given in the Table below (taken from Table 2.1 of EC7)].

Table B5.1 Partial factors – ultimate limit states in persistent and transient situations

	Actions			Ground Properties			
	Permanent		Variable	$\tan\phi$	c'	c_u	q_u [a]
	Unfavourable	Favourable	Unfavourable				
Case A	[1.00]	[0.95]	[1.50]	[1.1]	[1.3]	[1.2]	[1.2]
Case B	[1.35]	[1.00]	[1.50]	[1.0]	[1.0]	[1.0]	[1.0]
Case C	[1.00]	[1.00]	[1.30]	[1.25]	[1.6]	[1.4]	[1.4]

[a] Compressive strength of soil or rock

Cases A, B and C are first defined in Eurocode 1, Table 9.2, which is reproduced as Figure C2.1. This comes in the middle of a section which deals with combination rules for actions (ie load combinations), and the three cases supplement these, rather than providing an alternative. Table 9.2 of EC1 has some important footnotes which will be discussed later.

The origin of Cases A and B lies in structural engineering codes: they are the load combinations considered for 'stability' and 'strength'. In this structural context, 'stability' refers to situations in which loads balance each other so that there is negligible reliance on the strength of a structure for safety. Typical structures of this type include canopies, and many structures during construction, including both structural frames and balanced cantilever bridges. Case A is derived from such 'stability' considerations, requiring a reduction factor to be applied to weight which acts in a stabilising manner. In the majority of structures, structural strength is a dominant feature in ensuring safety, and for these the partial load factors of Case B are similar to the basic requirements of BS 8110.

Case C came from earlier drafts of Eurocode 7. It has a unit factor on permanent loads (generally weight) and a relatively small factor on variable loads. EC1 and EC7 in combination make it clear that in Case C the main factors are applied to the strength of ground materials. Thus, in Case B the non-unit factors are all on loads and in Case C mainly on ground strength. In all three cases, the values of partial factors to be applied to strength of structural materials are to be taken from the other Eurocodes, as relevant; they do not, in general, vary between cases.

The use of the three cases in EC1 Table 9.2 came about largely to assist integration of EC7 with the other codes, and EC7 therefore follows the concept of the three cases. However, it uses Case A in a special way by applying it to potentially buoyant structures, which depend on a balance of weight and water pressure, often with ground strength playing only a minor role. EC7 recognises that there may be many situations which depend largely on the balance of forces, but in which soil strength nevertheless plays a role. So, even for Case A, partial factors are applied to the strength of ground materials.

An example of the use of the three cases in checking the ultimate limit state of a simple, potentially buoyant structure is given in E18. The reader may find it helpful to refer to that example to clarify the concepts described above, before studying the rest of this section.

B5.1.2 The debate

In most situations, it is found that when geotechnical calculations are carried out to determine the required geometry of foundations or other structures, Case C is critical and therefore determines the geometry. Similarly, it is most often found that Case B governs the structural design.

Engineers involved in the development of Eurocode 7 are concerned that the use of the cases will require some additional calculations, at least until the user has sufficient experience to be sure which calculations are critical. There would clearly be advantage in reducing the cases, particularly B and C, to a single case. This might be achieved during further development of EC7, but the situation at early 1998 is that opinion is split fairly evenly as to how this might be done. Some engineers argue for omission of Case B entirely, whilst others would prefer to use only Case C for geotechnical design (giving the sizes of structures) and only Case B for structural design (checking their strength). Still others would omit Case C entirely.

It is fairly clear, though still debated, that ENV 1997-1:1995 requires that, in principle, *all designs must comply with all three cases in all respects, both geotechnical and structural.* The authors recommend that this be accepted until a better approach has found broad agreement.

B5.2 The concept of 'cases'

The concept of load cases is familiar in structural design. It is normal that these require several sets of calculations, only one of which will prove to be critical and so govern the design decision. Thus, in the calculations, each structural element may be at a limit state for one load case but will not be at limit states for others. The basic requirement is that it does not exceed a limit state in any of the calculations.

In structural design, some, perhaps almost all, of the cases can often be dismissed by inspection as being not the most critical for a particular calculation. It is only necessary to carry out calculations for those cases which might prove to be the most critical.

The 'cases' used in EC1 and EC7 have the characteristics of load cases described above. In addition, however, the partial factors applied to ground strength are varied between the cases.

B5.3 Current requirements of EC1 and EC7

Several key elements of the relation between EC1 and EC7 are contained in the footnotes to EC1 Table 9.2.

Footnote 1 requires that *The design shall be verified for each case A, B and C separately as relevant.* The term 'as relevant' has caused much debate. One interpretation, favoured by some, is that only Case B is relevant to structural design and only Case C to geotechnical design. However, it will be noted below that this leads to confusion in some cases, and possibly to over-optimistic design in others.

EC7 repeats EC1 Footnote 1 in 2.4.2(12)P. Paragraph 2.4.2(15) says that *Where it is clear that one of the three cases is most critical to the design, it will not be necessary to carry out calculations for other cases. However, different cases may be critical to different aspects of a design.* The clause then gives guidance on what aspects, both geotechnical and structural, of various types of design are most likely to be governed by one of the three cases. From this it is clear that EC7 requires that, in principle, **all designs must comply with all three cases in all respects, both geotechnical and structural.** That is, the cases are treated exactly like load cases, as discussed above. Calculations need only be prepared for cases which cannot be dismissed as obviously 'less critical'.

Footnote 3 of Table 9.2 says that ... *the characteristic values of all permanent actions* **from one source** *are multiplied by [1.35] if the total resulting action is unfavourable and by [1.0] if the total resulting action is favourable* (authors' emphasis). Obvious examples of this arise in considering water pressure, which may act in both a disturbing and restoring sense simultaneously;

it would be irrational to factor the disturbing and restoring components of the same pressure differently.

For retaining walls, Paragraph 2.4.2(17) of Eurocode 7 effectively defines 'the ground' as *one source*. That is, it says that *All permanent characteristic earth pressures* **on both sides of a wall** *are multiplied by [1.35] if the total resulting action is unfavourable and by [1.0] if the total resulting action is favourable* (authors' emphasis). Since the term 'earth pressures' is defined, earlier in the same clause, to include water pressures, it is clear that this factor of 1.35 applied to all earth pressures will have the effect of increasing bending moments and shear forces in the same ratio.

Paragraph 2.4.2(17) accepts that in some cases it is physically unreasonable to increase earth pressures, especially water pressures, by the factor 1.35. For these situations, EC7 allows the alternative of carrying out the calculations using unfactored characteristic earth pressures, then multiplying resulting bending moments and internal forces by 1.35, for Case B. EC7 says that this treats the factors on actions in EC7, Table 2.1 as 'model factors'.

Although the statements in 2.4.2(17) are restricted to earth pressures, the authors recommend that this approach can be used more widely, but only where direct application of the factor to earth pressures or to weight of earth leads to forces which are physically unreasonable.

Footnote 5 of EC1 Table 9.2 acknowledges that the factors applied to ground properties in Cases B and C may be different, as is required by EC7 Table 2.1. Footnote 6 confuses matters, however, stating that the use of design ground properties **may** be introduced in accordance with ENV 1997 **instead** of using γ_G [1.35] and γ_Q [1.50]. These words effectively describe Case C, which is presented as an 'additional', rather than 'alternative', requirement in EC7.

Footnote 6 notes that a model factor may be used with Case C, and this possibility is also mentioned in EC7. Paragraph 2.4.2(15) says that a model factor *may be introduced as relevant* for structural design, and a similar statement is made in 8.6.6(4) in relation to retaining structures. However, no boxed values are offered for these factors and none are supplied in the British NAD. Information on the possible future use of model factors is included in D2.2.

The only specific value attached to a model factor in EC7 is the use of [1.35], noted above, where it would be physically unreasonable, in Case B, to multiply earth and water pressures by this factor.

B5.4 Reasons for the requirements

There are two main arguments in favour of the approach that, in principle, *all designs must comply with all three cases in all respects, both geotechnical and structural.* These are:

a the need to have equilibrium in structural calculations, and
b the possibility that one case alone might be lead to over-optimistic design.

B5.4.1 Equilibrium in structural calculations

There has long been controversy over the choice of the factors 1.35 and 1.5 for structural design. Some geotechnical engineers have argued that factors on **weight** should always be 1.0, and this view is supported by some structural engineers in the UK and, more generally, in Scandinavia. Simpson (1992) suggested that the factor 1.35 may be reasonable, in structural design, if it is viewed as an allowance for uncertainty of load path as well as of weight itself. The same paper argued that the factor 1.35 was not relevant to geotechnical design, but the considerations presented below are now thought to outweigh this view.

There is no real prospect that the factors 1.35 and 1.5 will be changed in Eurocodes 1 to 4, at least, or in their NAD's for most countries, including the UK. It is therefore necessary that EC7 enables designers to complete the design of structures in which these factors have been applied to loads in the structural calculations. This becomes impossible if the geometry of the

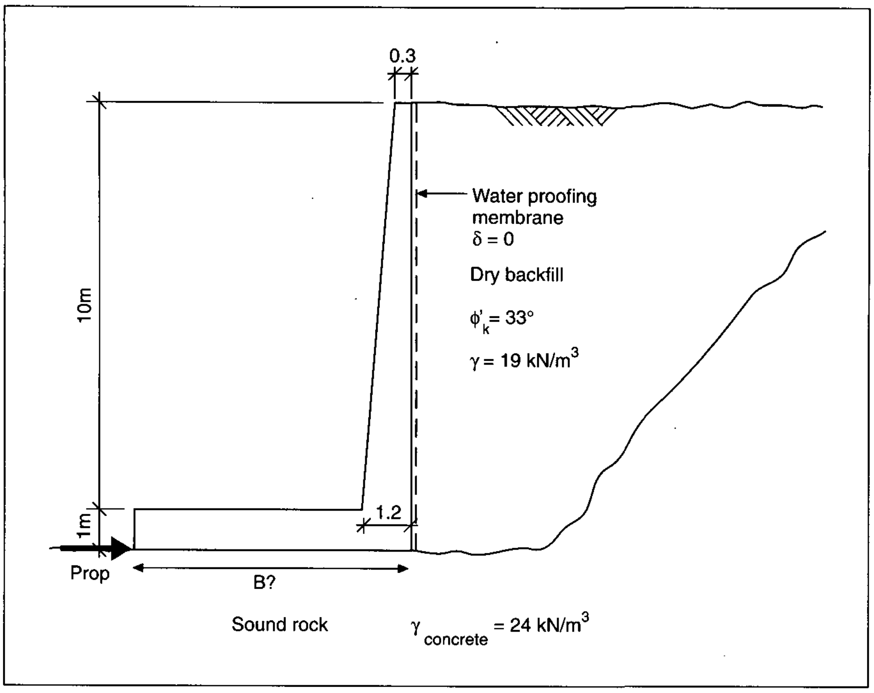


Figure B5.1 Retaining wall founded on sound rock

structure cannot provide equilibrium, at its interface with the ground, with these factored loads.

Figure B5.1 shows the design requirements for a retaining wall which is founded on very good ground and is propped at its toe, so neither bearing capacity nor sliding are at issue. Eurocode 7 has no 'middle third rule' or specified factor of safety against overturning, though Subclause 6.5.4 considers footings with highly eccentric loads. Analysis of this wall to ULS Case C requires a footing width of 6.6 m, being determined by the eccentricity of the resultant force between the footing and the ground. However, if this width is used in a calculation for Case B it is found that the resultant force is more eccentric and does not pass through the footing; a width of 7.0 m is required. Hence a structural designer would be led to an impasse. (For comparison, a conventional design requiring a factor of safety on 'overturning' of 2.0 would require a footing width of 9.05 m, whilst a design to BS 8002 would require 6.25 m. For unfactored characteristic conditions, a footing width of 5.4 m would prevent overturning.)

Similar problems may occur for other structures subject to large lateral loads, such as chimneys and bridge piers. For retaining walls, the problem becomes worse when there is water pressure in the backfill, or fill of low density or low angle of shearing resistance is used. A further example involving a tension-piled structure is noted in E12.

Problems of this type only affect a minority of designs, but they occur sufficiently often to require that they are accommodated by the basic approaches of EC7.

B5.4.2 Insufficient safety in one case alone

Few engineers who have studied this problem would argue for omitting Case C and using Case B alone. Since the strength of soil is generally derived from friction, simply increasing its design weight, in slope stability calculations, for example, provides no safety margin. In principle, it might be possible to determine which zones of soil have a net disturbing effect and which a net restoring effect, and to factor the unit weights of these separately, but in practice this would be very difficult. In any case, the idea of factoring the unit weight of ground has little appeal as it leads to situations too remote from reality.

If Case B were omitted in the structural calculations for retaining walls, such as that shown in Figure B5.1, the 'load factor' on bending moment for ultimate limit state structural design would be as low as 1.2 to 1.25 for dry soil, and even lower if the soil is saturated. This is uncomfortably low, particularly for walls of masonry or unreinforced concrete which have little ductility and ability to release unexpectedly high earth pressures. (Design of the wall structure for compaction pressures is discussed in C8.6.6.)

Similarly, for foundations subject mainly to permanent load, if Case B is omitted, the factor of safety on some structural stresses falls to unity for ULS design. (Shear stresses in spread foundations are one example.) It is clear that this approach cannot be acceptable to structural designers and could not be agreed with the drafters of the other Eurocodes.

B5.5 Problems caused by the requirements

Two main objections are raised to the principle that all designs must comply with all three cases in all respects, both geotechnical and structural. These are:

a the designer has more work to do, carrying out additional (and, it is implied, unnecessary) calculations, and

b the resulting designs are uneconomic.

It was noted above that parallel sets of calculations for more than one case are only required when it is not obvious which case will dominate. For most situations, the engineer is soon able to recognise by inspection which case will govern the design, so only one set of calculations is needed. This situation is similar to the use of load cases in structural design, as discussed above, or to the approach required by CIRIA Report 104 for the design of embedded retaining walls. Since design is increasingly carried out by computer, the repeat of a set of calculations with a different set of partial factors is usually not onerous, if it is necessary. Hence, objection **a** is considered by the authors to be of little concern.

Objection **b** is important, however. To date (early 1998), it has been raised only in relation to design of flexible retaining walls, particularly steel sheet pile walls. These are considered in the next section.

B5.6 Steel sheet pile walls

There has been a long-standing difficulty in the design of the length and strength of embedded retaining walls. This is epitomised in CIRIA Report 104 in which the length is determined by one calculation, generally factoring the strength of the ground in some way, and the strength of the wall is determined by a separate calculation with unfactored ground strength. The calculations do not check that the wall is strong enough to use its full length of embedment, should this ever be required. This causes confusion to designers. The requirement in EC7 to check both Cases B and C in both respects (structural and geotechnical) has the potential to remove this uncertainty.

Nevertheless, walls have been designed by this type of approach in the UK and elsewhere in Europe for many years. They have performed satisfactorily and appear to be sufficiently strong. It might be concluded that they are unnecessarily long; in fact, UK designs to CIRIA Report 104 are probably longer than many continental designs.

Design to EC7 Case C will generally lead to walls which are shorter than those of CIRIA 104, but the calculated bending moments may, at first sight, be bigger. Two points may be noted, however:

a whilst CIRIA 104 specifies that the earth pressure distributions adopted are to be simple, linear, active and passive lines, as in Figure B5.2a, Eurocode 7 has no such restriction;

b it might be appropriate to use full plastic moments of resistance in designs to the Eurocodes.

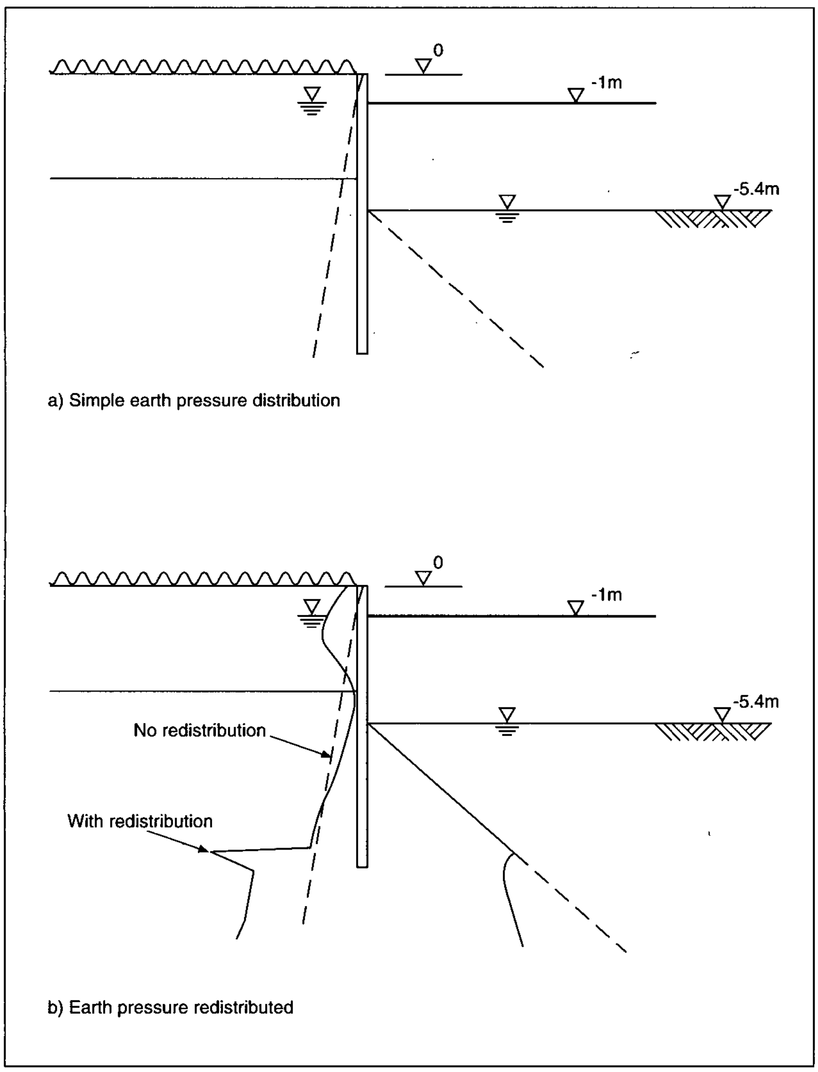


Figure B5.2 Earth pressure distributions for propped wall

EC7's requirements are merely that partial factors should be applied to strengths and loads, and that equilibrium be achieved in a manner compatible with the deformation characteristics of the materials. For propped walls, it therefore allows redistribution of earth pressure, as in Figure B5.2b, provided it is justified by either theoretical or empirical means. Such redistribution of earth pressures reduces the bending moments in propped walls. E15 shows how the effects of redistribution may be obtained using the method of Rowe (1952, 1957), or software such as finite element programs, FREW or SPOOKS. For this example, the resulting bending moments for ultimate limit state design to EC7 are similar to those of CIRIA Report 104. If further trials show this to be the case consistently, it could be justified to use the simpler methods of CIRIA 104 as a direct substitute for the requirements of EC7.

The design ULS moments of resistance of steel sheet pile sections are considered in Eurocode 3 Part 5 Clause 5.2. For most sheet pile sections specified in practice, this allows full plastic moments of resistance to be used. Use of these in combination with Case C bending moments will return results which are closer to CIRIA 104 designs, a desirable objective since these have been found to be sufficiently safe, whilst providing a self-consistent design approach. An example of this may be seen in E14.

B5.7 Case A

The use of Case A to check 'stability' in structural analysis, and its adoption in EC7 as a check on buoyant structures were noted in B5.1.1 above.

For potentially buoyant structures, EC7 Table 2.1 requires a factor of safety on weight of $\gamma_G = [0.95]$. This factored weight must then be greater than the worst uplift pressures *which could occur in extreme circumstances* (Paragraph 2.4.2(10)P). No factors are applied to the uplift pressures. Other factors, which usually play a minor role, are the effects of soil strength and of variable loads; these are ignored here.

The boxed value of the factor [0.95] has been agreed by most European countries, but the authors advise caution in the use of a value so close to unity. The density of construction materials is usually known to good accuracy, and it is often found that structures contain slightly more material than the nominal amount. However, it is also possible that, at some point in the structure's life, some material might be removed, or substituted by a lighter material, and it is reasonable to make some allowance for this possibility. BS 6349, Maritime Structures, requires a factor of safety not less than 1.2 in these circumstances (BS 6349-3, 2.5.21), which equates to a factor of 0.83 (= 1 / 1.2) in the terms of EC7. An allowance of 5% is very small, and a figure of at least 10% is recommended, making the factor $\gamma_G = 0.90$.

In principle, Case A could be applied to problems, other than buoyancy, in which forces and weight must balance with little involvement of material strength. The retaining wall in Figure B5.1 is an example of this. However, no examples of this type have been found in which Case A would govern the design, and EC7 does not require that Case A be checked if there is no potential for buoyancy.

B6 TEMPORARY WORKS AND THE OBSERVATIONAL METHOD

B6.1 Consequences of failure

Eurocode 7 does not allow any reduction in safety levels merely because of the temporary nature of works or design situations. In relation to partial factors, however, 2.4.2(14)P says that *where it can be justified on the basis of the possible consequences, less severe values may be used for temporary structures or transient situations.* This paragraph refers to partial load factors, and it is perhaps unfortunate that Eurocode 7 does not have a similar sentence referring to partial material factors. However, the principle applied here is that the justification for less severe safety requirements lies in the possible consequences, not the temporary nature of the situation. On a construction site, there are several features which reduce the consequences of failure. The two main ones are:

a Engineers and experienced construction workers are present on site and are able to observe both the actual design situations which apply and the behaviour of the geotechnical structure. In some cases, the behaviour might be being monitored precisely by measurement, but often simple observation by experienced personnel is all that is needed to check that there is no imminent limit state.

b If observation leads to the conclusion that the structure is near a limit state, then the personnel on site often have the knowledge and authority to take action to prevent a failure, or at least to move people and equipment out of the area of danger.

It is therefore often reasonable to argue that the consequences of failure are lower during construction than they would be for the completed structure when untrained, unobservant members of the public are relying on its safety.

B6.2 The observational method

EC7, 2.7 introduces design by the Observational Method. This allows the design to be varied, in a planned manner, during the course of construction in response to the observed performance of the structure. The essence of the method is a precise plan of monitoring and of the response to be made to the results found.

A more detailed discussion of the history and use of this approach can be found in the recent CIRIA report (Nicholson et al (1997)), which sets out in some detail the philosophy, safety and technical implications of the Observational Method and also discusses the contractual framework in which it can be used. In the terms of the latter report, EC7, 2.7 is aimed primarily at the area of 'parameter uncertainty'. It is the intention of both publications to ensure that the Observational Method may be used in a manner that is no less safe than conventional design, provided that the requirements of the method are rigorously followed.

Combining EC7, 2.4.2(14)P and 2.7, it could be concluded that calculations carried out in support of an observational approach would use the same characteristic values for parameters, but would have smaller partial factors of safety. This is at variance with the conclusions of Nicholson et al, who state that it is preferable to retain the same nominal factors of safety in calculations, but to apply the factors to 'best estimate values', which are less cautious than the characteristic values of EC7. This may either be regarded as a decision not to adopt characteristic values for this purpose, or it could be said that characteristic values are defined differently when used with the observational method. The choice between these two approaches may depend on the perceived reason for using factors of safety – uncertainty or displacement control, as discussed earlier in B2.4.

Eurocode 7: a commentary

Part C Clause-by-clause commentary

CONTENTS

C1 GENERAL

C1.1 Scope

C1.1.1 Scope of Eurocode 7

Eurocode 7, to be used in conjunction with Eurocode 1, applies to the geotechnical aspects of design of buildings and civil engineering structures for strength, stability, serviceability and durability. It provides rules for calculating design earth pressures, and, in this respect, acts as a loading code for use with other Eurocodes. It also gives some minimum standards for construction requirements.

It does not cover seismic design, which is the subject of Eurocode 8 and in 9.1(1) it is stated implicitly that dykes and dams are also excluded. EC1, 1.1 states that it does not completely cover structures *which require unusual reliability considerations, such as nuclear structures.* By implication, this applies to the full suite of Eurocodes.

C1.1.2 Scope of ENV 1997-1

See A2.3.

C1.1.3 Further parts of Eurocode 7

Parts 2 and 3 of EC7, for laboratory and field testing, respectively, have been accepted as pre-standards (ENVs, early 1998). Besides providing basic descriptions of the performance of tests, these documents indicate how parameters required for design may be derived from the test results. Their status, contents and further development are discussed in A2.5 and D2.3.

C1.2 References

The reasons for noting the two ISO documents on Units and Symbols is not stated in the text. In practice, there is no need to refer to them.

C1.3 Distinction between principles and application rules

At this point Eurocode 7 repeats the definitions, given in Eurocode 1, of principles and application rules. Throughout the text, the modal verb 'shall' is used in principles, whilst 'should' is used in application rules. The following explanation is taken from 'Harmonised editorial style for Eurocodes' issued by CEN/TC250 in January 1996:

a *Principles are* **requirements** *for which 'shall' is appropriate;*
b *Application Rules are* **recommendations** *for which 'should' is appropriate;*
c *for* **permissive** *provisions 'may' is appropriate;*
d *in statements of* **possibility** *the appropriate form is 'can';*
e *'may' is used only to allow, never as an alternative for 'might' or 'can perhaps';*
f *'can' is never used to allow or authorise, but only to state a fact.*

Occasionally, EC7-1 uses the form 'shall normally' in principles. This is intended to indicate a requirement which is mandatory unless the designer can justify a claim that the circumstances are exceptional.

In this commentary, the words 'must' and 'recommended' are used to convey the opinions of the authors, as explained in A3.3.

C1.3(5)

Eurocode 7 gives precise calculation rules for relatively few situations, though it does provide formulae for bearing capacity factors and coefficients of earth pressure (Annexes B and G). In cases such as these, simpler formulae could be used provided it was shown that they give results at least as conservative. For example, it might be convenient for a designer to use established charts in place of the rather complex formulae provided in the code. This paragraph may become more important in the future if more detailed calculation rules are included in the code.

C1.4 Assumptions

It is an underlying assumption of the Eurocode system that *structures are designed by appropriately qualified and experienced personnel* and that construction is carried out to standards which are accepted as good practice in western Europe. The factors of safety proposed do not provide for deviation from these assumptions. The Site Investigation Steering Group (1993) provide recommendations on appropriate qualifications for personnel involved in geotechnical work.

It is not entirely clear who is to make the assumptions listed in this clause, and why. Firstly, they are assumptions made by the code drafters. This means that the code does not make any provision for non-compliance with the assumptions listed. For example, the code makes no provision for lack of necessary maintenance in a completed structure or for poor standards of construction. Features such as these are not allowed for within the margins of safety specified by the code.

The designer is permitted to make the assumptions which are listed, though he also has influence, to some degree, over ensuring compliance with them. This is particularly true for the first three assumptions: data collection, qualifications of the designer and communication within the design and construction team. He may have little or no power to ensure compliance with the remaining assumptions which concern actions taken after design, but he should at least be satisfied that compliance with these assumptions is practical and clearly intended at the time of design.

C1.5 Definitions

C1.5.1 Terms common to all Eurocodes

Much of the terminology of limit state design is defined in EC1 and not repeated in EC7. In fact, repetition of any kind is avoided as far as possible. It is therefore essential that users of EC7 have EC1 available. Some of the common terminology of limit state design is noted in A2.8.

C1.5.2 Special terms used in ENV 1997-1

C1.5.2(1)P

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 deemed to be particularly relevant.

Besides using theory and calculations, geotechnical engineering relies heavily on experience. However, experience which is only partly understood and partly applicable can be very dangerous. This paragraph demands clear communication of experience so that it is sufficiently objective to be both understood and challenged by all competent parties.

It is recognised as part of the sound geotechnical process that the performance of similar structures in the locality of a new design will be investigated, and lessons will be learnt from both success and failures. In British practice, this is often achieved by a visit to local Building Control offices.

The emphasis on relevant, and especially local, experience occurs repeatedly in the document. It is hoped that the required documentation of experience will make it more readily available to all comers. 'I can't explain why – it's just my engineering judgement' is not good enough!

Ground: soil, rock and fill existing in place prior to the execution of the construction works.

'Ground' is taken to mean the material which was present on site without the control of the geotechnical designer. It will normally need to be investigated. It may include fill as well as natural ground. Fill 'placed during execution of the construction works' is included within the definition of 'structure'.

Three other verbs are used repeatedly in the text, and it would be beneficial if they were defined:

'Considered'. This verb means that the engineer has given attention to an item, and should generally make a note of the fact, possibly by no more than a tick on a check list. The consideration may be very brief, leading to an immediate conclusion that the item is not significant for the design in hand. The verb 'consider' often does not imply the need for calculation, though this may be appropriate in some cases.

'Assess'. A quantity is said to be 'assessed' when a numerical value is determined by an engineer. This process may involve calculation, but not necessarily so. It could include processes of rough estimation or determination simply on the basis of comparable experience. In most cases this refers to prediction of the likely behaviour of a structure.

'Evaluate'. This verb is used to mean 'derive a value' for a parameter. In most cases this refers to derivation of characteristic values for material properties.

C1.6 SI units

C1.7 Symbols common to all Eurocodes

See C1.5.1.

C1.8 Symbols used in Eurocode 7

Geotechnical symbols have generally followed the recommendations of the International Society for Soil Mechanics and Foundation Engineering (ISSMFE (1977)). In particular, the symbol c_u is retained for undrained shear strength in preference to s_u. The symbol γ is also retained for unit weight, despite possible conflict with the symbol for partial factors.

C2 BASIS OF GEOTECHNICAL DESIGN

C2.1 Design requirements

C2.1(1)P

A structure shall be designed in compliance with the general design principles given in ENV1991-1 Eurocode 1 'Basis of Design'.

Section 2 sets out the approach to be taken in geotechnical design, including, but not restricted to, design based on calculation. It is based on the limit state approach and relies on Eurocode 1 for definitions of some basic terms. It cannot be used without Eurocode 1. Part B of this commentary discusses many of the concepts which underlie the drafting of Section 2.

C2.1(2) to (5)

Summary

The approach to be taken to geotechnical design depends on the level of complexity of each item of design. This concept is developed in (2) to (4)P and leads in (5) to an application rule describing the use of three Geotechnical Categories:

Geotechnical Category 1:	Small and relatively simple structures
Geotechnical Category 2:	Conventional types of structures and foundations with no abnormal risks
Geotechnical Category 3:	More difficult structures

Initial categorisation will probably be made before site investigation, and this will be revised as necessary during the stages of design and construction.

It is expected that designs for Geotechnical Category 1 will be based on experience and generally will not involve calculations; those for Geotechnical Category 2 will require conventional calculations and those for Geotechnical Category 3 will require specialist treatment.

Figure B1.2 shows the series of decisions needed to establish the appropriate category, whilst Figure B1.1 emphasises the fact that the category must be reviewed at all stages of the design and construction process.

Comment

Paragraph (5) is an application rule, which indicates that the use of categorisation is not mandatory, though a designer must nevertheless be able to show that he has taken due account of the complexity of the design.

The important message of Paragraph (5) is that sound geotechnical design does not always require calculation. In straightforward circumstances, it is acceptable to say, 'This has worked before in similar circumstances, so it is appropriate here'.

Design to Geotechnical Category 1 may not require the involvement of a qualified engineer. However, an essential feature of both Categories 1 and 2 is that the designer is competent to judge that the situation is not more complex than allowed within the category.

Broadly, items in Geotechnical Category 2 could be designed by an experienced civil engineer with some geotechnical knowledge, whilst those in Category 3 will normally need experienced specialist input.

In parts of some European countries, notably the Netherlands, pile design is very routine and may not need the involvement of a qualified engineer in simple cases. However, the authors of this commentary consider that pile design in the United Kingdom should not be considered to lie in Geotechnical Category 1.

It is essential to realise that it is individual items of construction which are to be categorised, not complete projects. Construction projects will often include both simple and difficult items, some of which can be designed by simple appeal to experience and others of which need either conventional engineering calculations or much more complex investigation and analysis.

Although it is initially attractive, the authors have some doubt about the value of this categorisation. It tends to introduce hard boundaries between categories where no hard boundaries exist. It also elevates the simple statement 'I need some advice here' to the level of a major management decision. This is as likely to prevent acquisition of the necessary expertise as it is to ensure it, particularly since raising the category of an item of design might mean passing control, conspicuously, to another engineer or another company.

It is notable that although Eurocode 7 sets out the three categories in some detail, little use is made of them in the rest of the text. If they were really valuable, they would perhaps have been used more extensively later in the document.

C2.1(6)P and (7)

Summary

For each geotechnical design situation it shall be verified that no relevant limit state is exceeded. This design requirement may be achieved by use of calculations, adoption of 'prescriptive measures', experimental models and load tests or the observational method. These four approaches may be used in combination.

Comment

These two short paragraphs are fundamental.

Paragraph (7) makes it clear that the use of calculations is not the only basis of design in the limit state method. It is good practice to use a combination of the four approaches. These are described in more detail later.

C2.1(8)P and (9)

Consideration of brittleness and ductility is fundamental to good design. This is noted here, and mentioned again in the context of retaining wall design in 8.6.1(4) and 8.6.6(3). In soils which may lose strength as strains develop, their operational strength must be considered in deriving characteristic values. This does not necessarily mean that their lowest (critical state, residual, etc) strength will be adopted as the characteristic value, but equally it suggests that an over-optimistic use of peak strengths would be unwise. For example, in conventional practice it is unusual to adopt angles of shearing resistance, ϕ', greater than about 38° for dense sands despite the fact that their peak value may be a few degrees higher. Though this value is well above their critical state angle, its use illustrates the balanced caution required here. Similarly, it is normal practice, at least in Britain, to use only small or zero values of c'.

C2.1(11)P

It is a code requirement that comparable experience is not ignored in design. No amount of calculation can negate this principle.

C2.2 Design situations

Eurocode 1 specifies that calculations and other assessments shall be based around design situations. EC1, 2.3(1) requires that: *The selected design situations shall be sufficiently severe and so varied as to encompass all conditions which can reasonably be foreseen to occur during the execution and use of the structure.* Setting up design situations is part of the process of ensuring adequate safety.

This clause provides a check list which should help to ensure that nothing is omitted from the design situation. One omission from this list, however, related to groundwater levels etc. is the question of desiccation, caused by tree roots or other effects, which may lead to swelling or heave of the ground. The list could also include construction sequence and situations arising during construction.

C2.3 Durability

Durability is to be considered and again a check list is provided. Durability generally relates to manmade materials which are covered in other Eurocodes, in particular Eurocode 2, Part 3 for concrete in the ground and Eurocode 3, Part 5 for steel in the ground.

C2.4 Geotechnical design by calculation

C2.4.1 Introduction

C2.4.1(3)P, (4) and (7)P

Two distinct types of calculation are noted here. The first, in (3)P and (4), involves a direct description of behaviour in the ground. This may involve the analysis of a failure mechanism or direct calculation of displacements. (Note in (4) that although this paragraph addresses design by calculation, the possibility that displacements may be based on comparable experience is included. Calculation of displacements is often difficult, and there is an intention to avoid spurious calculation).

Paragraph (7)P allows the possibility that avoidance of one specific limit state may be ensured by carrying out analysis for a different limit state *using factors to ensure that this* [the first] *limit state is sufficiently improbable.* This could include the common practice of relatively large factors of safety in foundation design in order to prevent unacceptable settlement. The factors are applied in a mechanism calculation (apparently ultimate state), in order to ensure compliance with the serviceability state requirement.

Taking (4) and (7)P together, the need for displacement calculations could be avoided in situations where experience clearly shows that design with an adequate factor of safety against failure will not displace unacceptably.

C2.4.1(5)P and (6)

It is intended that all methods of analysis adopted with this code 'are either accurate or err on the side of safety'. In general, factors of safety are not used to allow for unconservative calculation methods. If a basic method is adopted which may sometimes 'err on the unsafe side', then it should be modified by an appropriate factor to make it safe. This is a 'model factor'; its value depends on the individual method and is additional to other factors required by the code.

EC1, 1.6 defines model factors as partial factors associated with the uncertainty of the resistance or action or action effect model. Since 1994, there has been much discussion about the use of 'model factors' in EC7, and it is possible that they will play a more prominent role in future versions (see D2.2).

C2.4.1(7)P

In geotechnical design, it is commonly the case that precise analysis of significant items is impossible due to a lack of reliable data, theoretical understanding, or both. It is therefore common practice to carry out such calculations as may easily be done, and to apply sufficient factors of safety to ensure that other requirements are adequately covered.

The most common example of this is the use of calculations for a plastic failure mechanism in the design of foundations, especially piling, with factors of safety intended to be sufficient to ensure that displacements will be acceptable. The plastic failure mechanism would usually constitute an ultimate limit state, whilst a state in which displacements become unacceptable could be either a serviceability or ultimate limit state, as discussed in B2.1.

In Paragraph 8.4(2), for retaining walls, it is stated that *the design methods and factors of safety required by this code for ultimate limit state design are often sufficient to prevent the occurrence of this type of limit state (ie serviceability) provided the soils involved are at least medium dense or firm, and adequate construction methods and sequences are adopted.* This statement is probably more generally applicable, and the relationship of the values of the partial factors to serviceability requirements is still under discussion in CEN/TC250/SC7 (early 1998). BS 8002 has a similar statement.

C2.4.1(8)P and (9)
The code emphasises that analytical calculation should never become remote from field experience. It also allows calculations which are based entirely on correlations set up by field observation. In 7.4.1 on design methods for piling, an approach is set out which combines both calculation and observation. General application of this thinking is sound practice.

C2.4.2 Actions in geotechnical design

Eurocode 1 (ENV 1991-1:1994), 1.5.3.1 defines 'action' as follows:

a *Force (load) applied to the structure (direct action)*
b *An imposed or constrained deformation or an imposed acceleration caused, for example, by temperature changes, moisture variation, uneven settlement or earthquakes (indirect action).*

The use of the word 'action' with the meaning adopted in the Eurocodes is unfamiliar to English speakers. It might be preferable to substitute the word 'load', provided that this definition is sufficiently broad to include imposed displacements as well as forces. (This may be acceptable when it is remembered that 'load' is used with a broad meaning in other branches of engineering, for example the 'load' on an electrical system, which is not a force). British engineers will remember that Newton spoke of 'actions' and 'reactions'. We still use the term 'reaction' to mean a force, but not the term 'action'.

In the Eurocodes, an 'action' is a force which is not a 'reaction'. It is a force which is known at the start of a particular calculation; its value is not derived within that calculation. 'Reactions' are know as 'action effects'.

The following wording, proposed in UK comments on EC1, might be clearer: *For any calculation, the values of actions are defined quantities, and are not derived from the calculation model. In certain circumstances, some quantities may be either actions or action effects. For example the downdrag on a pile through an embankment may be considered an action in designing the pile and as an action effect when designing the embankment.*

It is fruitless to try to find a more fundamental or philosophical definition of 'action'. We are accustomed to using the term 'load' in precisely the way intended here. Consider, for example, the 'load' on a pile. Calculations are first carried out for the building to be supported, using a frame analysis in which the loads are the weights of the structural elements and other items carried by them. At this stage, a 'reaction' force is derived in a column in the lowest storey of the structure. The structural engineer then turns a page in his calculations and starts to derive the structural section required for the column; in that particular calculation what was previously a reaction has become the 'column load', and it may have a variety of factors of safety applied. The same force later becomes the 'pile load', possibly with different factors applied. The essential feature of all these 'loads' or 'actions' (but not the 'reactions') is that they are fixed values in the particular calculations in which they are used and partial load factors can be applied to them. In this context, the purpose of the factors is to account for the possible variations in the forces which are adverse for each of the elements to be designed, considered separately.

In geotechnical engineering, the question has often been asked: 'Are earth pressures actions?' The answer to this is that they are actions in calculations in which their values are know and fixed, but they are not actions in calculations in which they are derived. Similar considerations apply to downdrag (negative skin friction).

C2.4.2(12)P

A full discussion of Cases A, B and C is presented in B5.

Paragraph (12)P is copied directly from EC1. The use of the words 'as relevant' is unfortunate. Paragraph (15) makes it clear that all designs must comply in all respects with all three cases. However, for each aspect of the design it will often be obvious which case is critical and others can be dismissed by inspection. EC7 does not demand calculations where they are obviously unnecessary.

Table 9.2 of EC1 is reproduced in Figure C2.1, and presents the load factors to be used with Cases A, B and C. It also describes the three cases, showing their origin. Unfortunately, this could be taken to mean that only one of the three cases needs to be applied: in particular only Case B for structural design and only Case C for geotechnical design. However, this procedure leaves unsafe gaps between the cases. In some circumstances, a structural design required by Case B is not sufficiently strong when used with the geometry required by Case C, or a design to the Case C geometry is not stable under the loading of Case B. Consistency is achieved by requiring that designs must satisfy all three cases.

The load factors of EC1 are adopted directly in EC7 for Cases B and C. The concept is extended by varying also the material factors on ground properties for these two cases. The material factors on structural properties are not varied and remain consistent with the other Eurocodes. Case A, which is used in other Eurocodes to ensure stability against simple toppling failures and the like, is used in EC7 to ensure stability of potentially buoyant structures. The load factors required by EC7 for Case A are therefore different from EC1.

C2.4.2(14)P

Table 2.1 of EC7 provides values for partial load factors for conventional situations. Paragraph (14)P notes that more severe values should be adopted *in cases of abnormally great risk or unusual or exceptionally difficult ground or loading conditions.* It also makes an allowance for temporary works ('temporary structures or transient situations') but only on the basis that the possible consequences of failure are less than in other situations. This can be justified in one of two ways:

a during the temporary state, it will be possible to ensure that there is no danger to personnel and that the economic loss is limited, or

b because the structure is being regularly observed (possibly including precise monitoring), there will be warning of a potential failure and it will be possible to take remedial or avoiding action. This is discussed further in B6.

C2.4.2(15) and (17)

The wording of Paragraphs (15) and (17) is particularly important and should be read carefully. Examples of its application are given in E12, E14, E15 and E18.

C2.4.2(19)P, and (20)

Although the Eurocode system is based mainly on the use of partial factors, the possibility that design values may be derived by other methods, including direct assessment by the engineer, is allowed. This is obviously relevant to parameters for which no partial factor values are provided, but which can affect limit state conditions.

The designer is also allowed to decide that application of the conventional partial factors is not appropriate. However, he should follow this course with

Case[1)]	Action	Symbol	Situations	
			P/T	A
Case A Loss of static equilibrium; strength of structural material or ground insignificant (see 9.4.1)	Permanent actions: self weight of structural and non-structural components, permanent actions caused by ground, groundwater and free water - unfavourable - favourable	 γ_{Gsup}[4)] γ_{Ginf}[4)]	 $(1.10)^{2)}$ $(0.90)^{2)}$	 (1.00) (1.00)
	Variable actions - unfavourable	γ_Q	(1.50)	(1.00)
	Accidental actions	γ_A		(1.00)
Case B[5)] Failure of structure or structural elements, including those of the footing, piles, basement walls, etc., governed by strength of structural material (see 9.4.1)	Permanent actions[6)] (see above) - unfavourable - favourable	 γ_{Gsup}[4)] γ_{Ginf}[4)]	 $(1.35)^{3)}$ $(1.00)^{3)}$	 (1.00) (1.00)
	Variable actions - unfavourable	γ_Q	(1.50)	(1.00)
	Accidental actions	γ_A		(1.00)
Case C[5)] Failure in the ground	Permanent actions (see above) - unfavourable - favourable	 γ_{Gsup}[4)] γ_{Ginf}[4)]	 (1.00) (1.00)	 (1.00) (1.00)
	Variable actions - unfavourable	γ_Q	(1.30)	(1.00)
	Accidental actions	γ_A		(1.00)

P: Persistent situation T: Transient situation A: Accidental situation

1) The design should be verified for each case A, B and C separately as relevant.

2) In this verification the characteristic value of the unfavourbale part of the permanent action is multiplied by the factor [1.1] and the favourable part by the factor [0.9]. More refined rules are given in ENV 1993 and ENV 1994.

3) In this verification the characteristic values of all permanent actions from one source are multiplied by [1.35] if the total resulting action effect is unfavourable and by [1.0] if the total resulting action effect is favourable.

4) In cases when the limit state is very sensitive to variations of permanent actions, the upper and lower characteristic values of these actions should be taken according to 4.2(3).

5) For cases B and C the design ground properties may be different, see [ENV 1997-1-1.]

6) Instead of using γ_G (1.35) and γ_Q (1.50) for lateral earth pressure actions the design ground properties may be introduced in accordance with ENV 1997 and a model factor γ_{Sd} is applied.

Figure C2.1 Table 9.2 of EC1

great caution, ensuring that the design values are no less conservative than the factored values derived using Table 2.1.

Paragraph (20) provides an overall requirement that design values assessed directly should be such that *a more adverse value is extremely unlikely to affect the occurrence of the limit state.* It acknowledges that use of direct assessment may be appropriate where application of Table 2.1 is clearly impossible.

The authors recommend that designers should always check that design values for ultimate limit state calculations, however derived, are at the limits of credibility or preferably beyond. It is unfortunate that this is not required by the code if the normal route to design values via characteristic values is followed. The designer's responsibility then stops at characteristic values.

C2.4.3 Ground properties

C2.4.3(1)P

As for actions, the main method to be used for deriving design values of ground properties is division of a characteristic value by a partial factor. However, direct assessment is also allowed by 2.4.3(14)P.

C2.4.3(2) to (6)

For a detailed discussion of the concept of characteristic values in geotechnical design, with illustrative examples, see B4.

Characteristic values for ground materials are based on an assessment of what is actually in the ground and the way that material will affect the performance of the ground and structure in relation to a particular limit state. Field and laboratory tests are to be used, but they are a means of assessing what is in the ground; characteristic values are not derived directly or solely from the test results. Paragraph (3) makes it clear that a conversion factor may need to be applied to test results to make them more representative of the ground conditions. Paragraph (4)P emphasises that *geological and other background information, such as data from previous projects* must be taken into account besides the results of soil tests. It also states that the properties of the ground might be changed by workmanship and construction activities, and this is to be included in the assessment of characteristic values. The relevance of individual test results will depend on the type of geotechnical structure being designed, and this may also affect the assessment of characteristic value.

The critical conclusion of this thought process is contained in Paragraph (5)P: *The characteristic value of a soil or rock parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state.*

Paragraph (6) says that statistical methods may be employed provided that they take account of comparable experience. In practice, this requires a very high order of statistical competence which will rarely, if ever, be available within the geotechnical community. This paragraph is helpful in setting a level of probability to be adopted for characteristic values if statistical methods are used, namely, 5%. That is, there should be a 5% probability that the strength governing (ie dominating) the behaviour of the ground at the limit state will be less than the characteristic value. The 'ground' to be considered in this assessment is the body or surface of ground (such as a slip surface) which would need to fail in order for a limit state to occur.

C2.4.3(7)P, (10)P and (11)

These paragraphs deal with situations in which soil strength acts in an unfavourable manner, such as the case of downdrag (negative skin friction). In these situations, the 'cautious assessment' required by (5)P is an upper characteristic value, greater than the most likely value.

Paragraph (11) is an application of (10)P and is not relevant to other earlier paragraphs. It refers to situations where the ground's strength is acting in an unfavourable manner, but displacements will not be large enough to cause full mobilisation of the strength of the ground.

C2.4.3(8)P
Minor uncertainties in the geometrical data, including ground surface, ground water level and the levels of interfaces between strata are to be taken into account in the characteristic values in the ground material properties. This paragraph may be erroneous – see further under C2.4.5.

C2.4.3(9)P
This is the pivotal paragraph of this clause, referring to Table 2.1 for the values of partial factors.

Eurocode 7 differs from some other codes (eg BS 8002) in applying different partial factors to c' and $\tan\phi'$. This reflects a view that values for c' are generally less reliable than those for $\tan\phi'$. The alternative view is that both parameters are derived from the same tests and are used together, so they should be factored together, as in BS 8002. The distinction between the factors is particularly unpopular in Germany, where relatively large values of c' are often used in design. E17 illustrates the importance of c'.

In practice, soil strengths are usually of no more than minor importance to Case A. It might therefore be convenient to use the Case C factors for soil strength in Case A, which is then regarded merely as a variant of Case C. The resulting design may be slightly more conservative than required by EC7, however, as illustrated in E12.

C2.4.3(12)P
Because the design of piles and anchorages depends largely on load testing, a different approach is taken for these in Sections 7 and 8.

C2.4.3(13)P
For serviceability limit states, characteristic values essentially become design values. This means that if the occurrence of a limit state depends only on the strength of ground materials, then there will in principle be a 5% probability of infringing the serviceability limit state. In practice, this may be tolerable since there is usually some margin in the definition of this state and a minor infringement of it is generally not very significant.

The same comment applies where the serviceability limit state depends only on the stiffness of the ground. The discussion in B4.12 is relevant here.

C2.4.3(14)P and (15)
See discussion at C2.4.2(19)P and (20).

Parameters for which partial factor values are not given in EC7 include ground permeability, which affects water pressures in non-hydrostatic situations, and stiffness, which may influence distribution of earth pressures and hence structural load effects, even at ultimate limit state. The latest version of the Hong Kong 'Guide to retaining wall design' (GEO (1993)), includes partial factors on permeability. These are set to 1.0 for soil and rock, and to 10.0 for filter and drainage materials.

Partial factors for stiffness in ULS calculations are not given in EC7. In many cases, values of unity could be appropriate, unless the calculated displacements completely dominate the occurrence of a limit state. Reference to 7.6.4(2)P, for settlement of piles, may be helpful.

C2.4.4 Design strength of structural materials

Summary

Design strengths of structural materials are based on the other Eurocodes.

C2.4.5 Geometrical data

Summary

Geometrical data include level and slope of the ground surface, water levels, levels of interfaces between strata, excavation levels, the shape of constructions, etc. Minor deviations of these are allowed for in the design values for material parameters but major deviations must be allowed for directly. For ultimate limit states, the design values should be the most unfavourable values which could occur in practice.

Comment

It is reasonable to state that the **design values** of material parameters (or the partial factors) accommodate some minor deviations in geometric data. The problem noted earlier in C2.4.3(8)P was the statement that the **characteristic values** accommodate deviations in geometric data. It is very difficult to see how this could be possible on the basis of other statements and definitions about characteristic values. The statement that such deviations are 'minor' implies that they are small compared with the other uncertainties in the design, represented in part by the partial material factors. They can therefore be accommodated within these factors.

C2.4.6 Limiting values for movements

This subclause is useful but not exceptional. Paragraph (4)P requires agreement between the geotechnical designer and the designer of the supported structure on limiting values for movements.

C2.5 Design by prescriptive measures

Summary

This clause presents an alternative to design by calculation, as discussed further in B3. The key to it is observation of the performance of other existing structures. This may be contrasted with the Observational Method (EC7, 2.6), which involves observation of the performance of the structure itself. Where it is possible to specify a safe design merely on the basis of observation of other structures, this is permitted by the code. This approach relies on comparable experience (EC7, 1.5.2(1)P) and will generally be relevant to Geotechnical Category 1 designs (EC7, 2.1). It is also relevant to features such as design against frost action, corrosion, etc. for which calculations are usually not performed.

C2.6 Load tests and tests on experimental models

This clause presents a further alternative to design by calculation, as discussed in B3. In practice, load tests, or model tests, and calculations should normally be used in combination. See also C2.1(7).

C2.7 The observational method

This clause allows the user of the code to design and proceed with construction by the observational method. It sets out basic requirements of the method, which must be applied conscientiously in order to comply with the code. The method allows construction to proceed, based on designs which may not comply with the partial factors required for calculations. Generally, these will be temporary works for which the method becomes an extension of 2.4.3(14)P.

The essential feature is that contingency measures are available and monitoring is sufficient to ensure that contingency measures will be implemented if they become necessary. In this regard, it is noted that monitoring need not necessarily involve accurate measurement. Sometimes, mere observation by site personnel is sufficient to give warning of an impending limit state.

See also B3.

C2.8 The geotechnical design report

It is a requirement of the code that a design report is produced summarising the assumptions, data and calculations on which the design is based. This is also to include a plan of supervision and monitoring and an addendum to the report is to be prepared to show that this has been carried out. An extract of the report containing requirements for supervision, monitoring and maintenance is to be provided to the owner/client.

Paragraph (2) states that for simple situations a single sheet may constitute an adequate report. An example of this type is shown in Figure B1.3.

See also C3.4 on the Ground Investigation Report.

C3 GEOTECHNICAL DATA

This section lays down basic principles of geotechnical investigation and derivation of parameter values. It also requires that a geotechnical investigation report must be produced. The section deals with basic principles, not with details. It will be supplemented by Parts 2 and 3 of Eurocode 7 which contain main requirements of site investigation and test procedures, with discussion of derivation of parameter values from individual tests. Further information on Parts 2 and 3 is presented in A2.5 and D2.3.

Much of the section lays down sound requirements which are not contentious. It is relatively short, however, and British readers will notice that it does not cover soil description and classification. CEN/TC250/SC7 has been instructed to cooperate with ISO in agreeing an international system of classification, but progress is slow.

C3.1 General

C3.2 Geotechnical investigations

C3.2.1 Introduction

C3.2.1(2)

It is required (as an application rule) that ground conditions which may influence the decision about the geotechnical category should be determined as early as possible. The paragraph then makes a distinction between the extent of investigation required for Category 1 and that for Categories 2 and 3.

C3.2.2 Preliminary investigations

C3.2.3 Design investigations

C3.2.3(10)

There is a specific requirement here for exploration points 'normally' spaced in the range 20 m to 40 m. These may include penetration tests or geophysical soundings besides borings or trial pits. For piles, it is required that the exploration extends to at least 5 diameters below the pile base. No mention is made of under-reamed piles with large bases, though it is acknowledged that there will be cases where deeper soundings or borings are needed.

C3.3 Evaluation of geotechnical parameters

C3.3.1 General

This subclause contains by implication a strong warning against unthinking use of test results in deriving characteristic values for parameters. Factors which may affect tests and their relevance to the field situation are listed. Checking of correlations between tests and comparisons with other experience is also emphasised strongly.

Specific examples of this general point are presented in the subclauses which follow.

C3.3.2 Characterisation of soil and rock type

C3.3.3 Unit weight

The terms unit weight, relative density (3.3.4), degree of compaction (3.3.5) and compactability (3.3.15) all relate to the density of the ground but have relevance to different parameters. Unit weight shows how heavy the ground is. Relative density and degree of compaction are different measures of its state of compactness which relate to strength and stiffness, whilst compactability shows how readily the material may be brought into a compact state.

C3.3.4 Relative density

See C3.3.3.

C3.3.5 Degree of compaction
See C3.3.3.

C3.3.6 Undrained shear strength of cohesive soils
See C3.3.1.

C3.3.7 Effective shear strength parameters for soils
See C3.3.1.

C3.3.7(4)
British designers seldom take advantage, explicitly, of the difference between triaxial and plane strain effective strengths. This is common in Danish practice, however.

C3.3.8 Soil stiffness
See comment on 3.3.1. Analysis of observed behaviour is recommended here.

C3.3.9 Quality and properties of rocks and rock masses
C3.3.9.1 Uniaxial compressive strength and deformability of rock materials

C3.3.9.2 Shear strength of joints

C3.3.10 Permeability and consolidation parameters

C3.3.11 Cone parameters
Although this section comments on parameters rather than individual tests, some parameters used in design can only be derived from one type of test. Cone resistance, sleeve friction and pore pressure during penetration are parameters of this type. See C3.3.1.

C3.3.12 Blow count for standard penetration tests and dynamic probing
See C3.3.11.

C3.3.13 Pressuremeter parameters
See C3.3.11, C6.5.2.3 and Annex C.

C3.3.14 Dilatometer parameters
See C3.3.11.

C3.3.15 Compactability

C3.4 Ground investigation report

It is a requirement of the code that a Ground Investigation Report be produced. This will often be incorporated into the Geotechnical Design Report described in Clause 2.8. The Ground Investigation Report is to include both factual material and a geotechnical evaluation of the information, stating the assumptions made in the derivation of the geotechnical parameters. These parts may be combined into one report or divided into several reports.

The code makes no comment on the various parties who must prepare these reports and to whom they will be communicated. The third item in 1.4(1)P is relevant: adequate continuity and communication must exist between the personnel involved in data – collection, design and construction. The code does not sanction a procedure in which these activities are carried out by different parties with no opportunity for discussion and clarification.

The subject of the code is geotechnical design and it is therefore clear that at the end of the process of geotechnical design, the designer must have available a statement of factual information, a statement of geotechnical evaluation and a statement of the design, including calculations. It may be that other parties, earlier in the contractual arrangements, will have prepared other versions of these reports which, under the contractual arrangements, are not passed on to the designer. This is particularly true of the geotechnical evaluation. In this case, in order to comply with the code, the designer must himself prepare any reports which are not supplied to him in an adequate form.

The code provides checklists intended to ensure that significant items are not omitted from the ground investigation report.

C3.4.1 Presentation of geotechnical information

C3.4.2 Evaluation of geotechnical information

C4 SUPERVISION OF CONSTRUCTION, MONITORING AND MAINTENANCE

This section has three main themes:

a Supervision of construction, monitoring and maintenance shall be undertaken as appropriate. The term 'as appropriate' is repeated many times. In many respects, the whole section is simply a check list reminding engineers of many of the items which might be appropriate in various circumstances.

b Designers must specify what is appropriate and must communicate this in specifications and record it in the geotechnical design report.

c Supervision of construction, monitoring and maintenance must be carried out in an orderly manner and must be recorded. The records of supervision and monitoring must be made available to the designer.

As in other parts of EC7, this section does not specify precise contractual arrangements. It does, however, specify what tasks must be undertaken and that an appropriate flow of information must be maintained. The section is concerned principally with the actions of the designer and with information others should make available to him. It gives no details of requirements for workmanship, for which reference should be made to the documents developed by CEN committee TC288 (Table A3.2). It is not the designer's responsibility to ensure that maintenance and post construction monitoring are carried out, but he is responsible to prepare and communicate specifications for these (EC7, 4.5(2)P and 4.6(1)P).

The section does not deal specifically with safety matters, for which reference should be made to the Construction (Design and Management) Regulations – the CDM Regulations (Health and Safety Commission, 1994).

C5 FILL, DEWATERING, GROUND IMPROVEMENT AND REINFORCEMENT

This section is effectively a check list of items which must be considered in the design of fills, ground improvement schemes and ground reinforcement. In general, it does not provide specific details.

C5.1 General

C5.2 Fundamental requirements

C5.3 Fill construction

This sub-section covers selection, placement, compaction and checking of fills. It provides a checklist of items which should all be covered in specifications for earthworks.

As for natural ground, design calculations involving fill materials require the assessment of characteristic values of the material properties. At the time of design, the fill to be used may not have been identified, though its properties will be specified. The assessment of characteristic values should follow the principles of 2.4.3, requiring a 'cautious estimate' of the properties which will be available to prevent the occurrence of a limit state.

C5.3.1 Principles

C5.3.2 Selection of fill material

C5.3.3 Selection of fill placement and compaction procedures

C5.3.4 Checking the fill

C5.4 Dewatering

The importance of control of groundwater is emphasised in 2.4.2(8)P and other places in the code. Design of dewatering is largely based on the Observational Method (EC7, 2.7). The code provides a check list of items to be considered by designers.

C5.5 Ground improvement and reinforcement

The code provides little direction on ground improvement and ground reinforcement, and so relies on the abilities of specialist designers. It is stated in (3) that in many cases these works should be classified in Geotechnical Category 3. Referring to 2.1(5), it appears that the basis for this must be that ground improvement and reinforcement are not considered to be *conventional types of structures and foundations* which could be classified under Geotechnical Category 2.

C6 SPREAD FOUNDATIONS

This is the first of four sections which deal with the geotechnical design of the different elements of construction.

The layout of these sections is the same in each case, consisting of:

a A general statement of scope.
b A list of limit states to be considered. The code encourages an orderly approach to calculations, first listing the cases to be considered and the loads to be applied.
c Actions and design situations to be considered.
d Design and construction considerations. These are mainly aspects of construction which must be considered during the process of design.
e Design for ultimate limit states.
f Design for serviceability limit states.
g Other items, such as 'Foundations on rock' in Section 6.

C6.1 General

Section 6 relates to spread foundations, which are usually located near to a ground surface but might be used at depth, such as in a deep excavation.

C6.2 Limit states

The list of limit states for spread foundations does not include failure by overturning. This is considered to be a form of bearing resistance failure, in the ultimate limit state, and to be covered by 'excessive settlements' in the serviceability limit state.

The list includes both *combined failure in ground and in structure* and *structural failure due to foundation movement.* The first of these is intended to refer to situations where a failure mechanism forms in the ground and the structure is ruptured at the same time. The second refers to situations where the ground itself has not failed, but displacements are sufficiently large to rupture the structure. The displacements could be settlement, heave, horizontal movement or rotation, or a combination of these.

C6.3 Actions and design situations

C6.4 Design and construction considerations

C6.4(1)P

The third item in the check list relates to depth of penetration of frost. This is dealt with in BS 8004 (3.2.9.1) by the requirement that all foundations shall be placed at depths not less than 0.45 m. The checklist requires that *future excavations* **for services** *close to the foundation* should be considered. It does not specifically say that future excavations for other purposes should be considered, though this would be equally relevant. However, it does not demand that spread foundations be designed to withstand all conceivable future excavations, even for services. It merely requires that the designer considers the situation and adopts a reasonable course of action.

C6.4(3)P

This paragraph allows spread foundations to be designed either by formal limit state calculations or by the use of presumed bearing pressures, related to serviceability limit state loads. This latter approach is essentially a 'prescriptive measure' as defined in Clause 2.5. The section does not develop the use of presumed bearing resistance or suggest values, except for the case of foundations on rock (in 6.7 and Annex E). It emphasises the importance of comparable experience, as specially defined in 1.4.2(1)P. The process of finding out what has typically been done for similar structures in the same ground conditions is seen as an important aspect to geotechnical design, offered as an alternative, or better as an adjunct, to calculation.

Some suggested values of presumed bearing resistance are given in BS 8004, Tables 1 to 3.

C6.5 Ultimate limit state design

C6.5.1 Overall stability

Besides checking the stability and serviceability of individual foundations, it is essential to check the stability of the site, or part of the site on which a structure is to be built. The reader is referred to Section 9 and to D2.2.

C6.5.2 Bearing resistance failure

C6.5.2.1 General

The basic requirement is represented by the inequality:

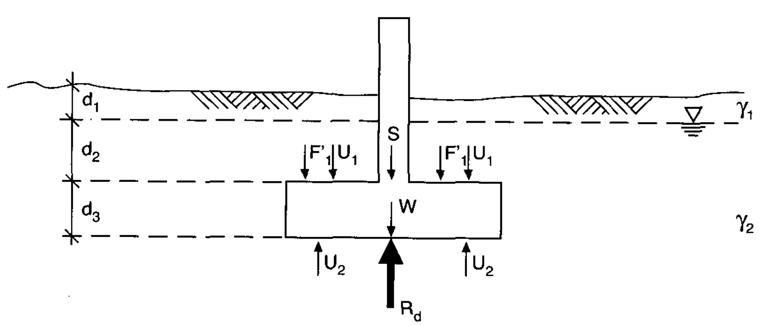
$V_d \leq R_d$

where V_d is the ultimate limit state design load normal to the foundation, and R_d is the design bearing resistance of the foundation against loads normal to the foundation.

This may appear to be a trivial statement, but it has two important implications.

a Having incorporated partial factors into both the design actions (V_d) and design resistance (R_d), no further factor of safety is required, but merely that R_d is not less than V_d.

b The check is to be made in terms of forces normal to the foundation base. Note that the action V_d includes the weight of the foundations and of any backfill material placed on top of them.



All design values (subscript d omitted for clarity):

A_b = area of footing base

A_c = area of column cross-section

γ_1 = total unit weight of soil above water table

γ_2 = total unit weight of soil below water table

S = action from superstructure

W = weight of footing

U_1, U_2 = forces due to water pressure

F_1 = action effect of overburden

$= (\gamma_1 d_1 + \gamma_2 d_1)(A_b - A_c)$

$= F'_1 + U_1$

$= (\gamma_1 d_1 + (\gamma_2 - \gamma_w) d_2 + \gamma_w d_2)(A_b - A_c)$

where F'_1 = effective action effect of overburden

$U_2 = \gamma_w (d_2 + d_3) A_b$

Design value $V_d = S + W + F_1 - U_2$

$= S + W + F'_1 + U_1 - U_2$

This must be matched by design resistance R_d, which in this case is an effective force.

Figure C6.1 Actions on a footing with hydrostatic water pressure

The code recommends that water pressures are included explicitly in the calculations. For drained conditions, it suggests that water pressures are generally included as actions, for which a typical situation is illustrated in Figure C6.1. It is consistent with the definition of actions to treat drained water pressures in this way, since they are quantities which are fixed at the start of the calculation (EC7, 2.4.2(1)). This approach implies that resistance is calculated in terms of effective stress. Note that the force due to pore pressure on the underside of the footing, U_2, acts so as to reduce the value of V_d.

The principle of actions from a 'single source' noted in Table 9.2 of ENV 1991-1 is important here. Its development for retaining structures, given in 2.4.2(17), is equally applicable to spread foundations. When designing for Case B, water pressures should be multiplied by γ_G (generally 1.35, but 1.0 if favourable). However, provided they are all treated as actions, hydrostatic water pressures will all cancel out regardless of the factors applied. If pressures above the footing were treated as actions, but those below as resistances, then an unreal imbalance would be created by the application of the partial factors in Case B.

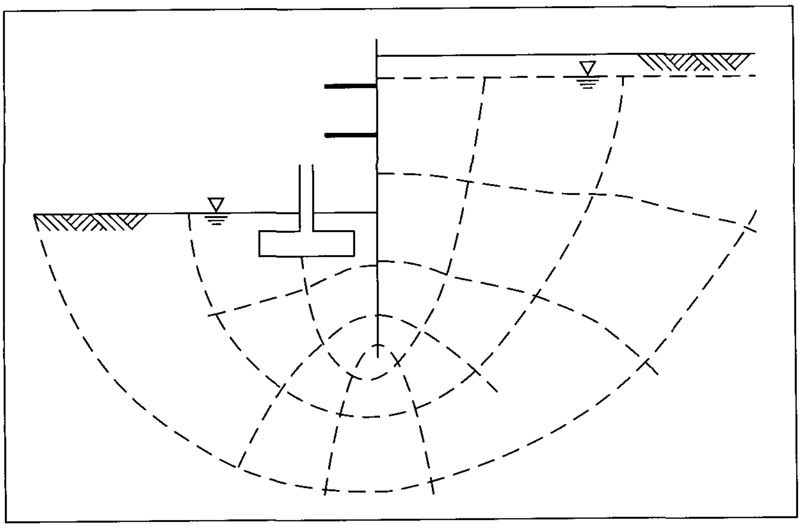
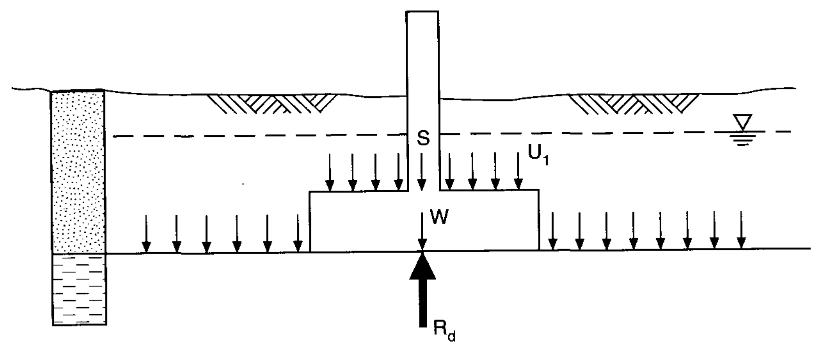


Figure C6.2 Footing in ground subjected to seepage

Figure C6.2 shows a situation of steady seepage in which water pressures are fixed (and therefore are actions) but are not hydrostatic. In this situation, application of the factors $\gamma_G = 1.35$ in Case B will lead to an increase in hydraulic gradient. However, the 'single source' principle would also require that 1.35 be applied to the weight of the soil, so it is likely that the more critical case is $\gamma_G = 1.0$, with the soil/water system acting favourably to support the footing.

Figure C6.3 shows a situation where a drained material with water pressures lies above the footing and an undrained material beneath it. Again, the 'single source' principle will generally show that $\gamma_G = 1.0$ is critical.



Arrows show hydrostatic water pressures (vertical components only). Those acting on the footing are included in V_d, whilst those alongside the footing augment R_d. In this (undrained) case, R_d is a total force acting on the base, which includes both the hydrostatic pore pressures and excess pore pressures due to undrained shearing.

The action on the ground, V_d, which is to be supported by the undrained resistance R_d, is given by:

$V_d = S + W + F'_1 + U_1$ (all design values)

F'_1 is defined in Figure C6.1.

A possible alternative approach is to include the **hydrostatic** water pressures in the clay as part of V_d, as for the drained case; these act upwards on the base of the footing, reducing the value of V_d, as in Figure C6.1. The **excess** pore pressures generated at the base of the footing due to shearing remain part of R_d, however. This is consistent with the definition of actions in 2.4.2(1)P, since the hydrostatic component is known, but the excess pore pressure is unknown, at the start (and end) of the calculation.

Figure C6.3 Footing with hydrostatic water pressure in granular soil over undrained clay

C6.5.2.1(2)

The use of buoyant weight is not recommended by the authors of this commentary because it tends to cause confusion. It is preferable to write calculations in terms of total weight and explicit water pressures. An exception occurs in the calculation of the N_γ term in the bearing capacity equation (Annex B), for which the use of buoyant unit weight of the soil is practically unavoidable.

C6.5.2.2 Analytical method

C6.5.2.2(2)

The calculation method for bearing capacity given in Annex B is recommended for general use. This approach, using bearing capacity factors, is accepted in many countries, including Britain. It is sometimes challenged, particularly in France, on the grounds that it is not reasonable to assume a constant value of ϕ' throughout the ground at all states of stress. This relates to the French preference for the empirical approach based on the results of pressuremeter tests, as noted under Annex C.

Figure C6.4 shows the values of bearing capacity factors derived from the formulae of Annex B. This annex is marked as 'informative' and is introduced by an application rule at 6.5.2.2(2). The designer is therefore permitted to use alternative calculations, and he may need to do this for layered materials. The method in Annex B gives an indication of the degree of accuracy and conservatism which is appropriate. The formulae are taken from DIN 4017 and are not believed to be markedly conservative. Appendix 1 compares these values for N_γ with values from other similar formulae.

Some examples of the use of Annex B are presented in E2 to E4.

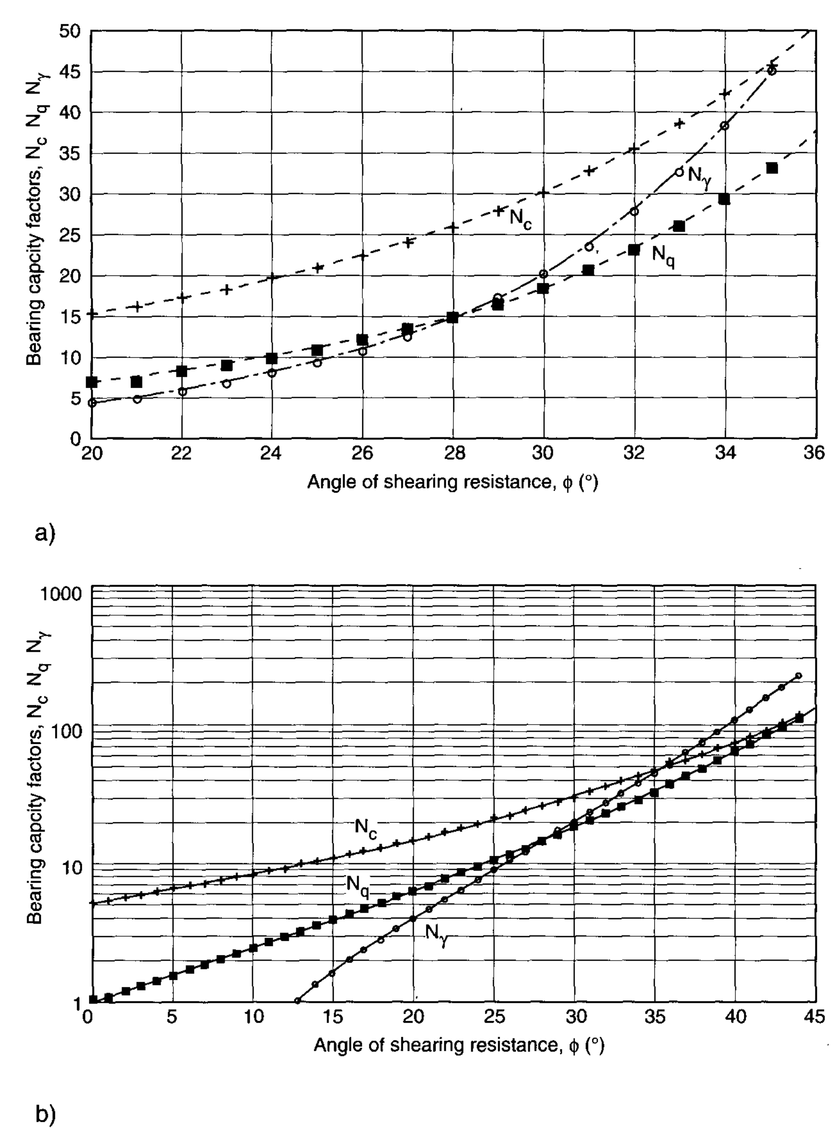


Figure C6.4 Bearing capacity factors derived from the formulae in Annex B

C6.5.2.2(3)P to (5)
The standard formulae for bearing capacity do not easily take account of soil layering or discontinuities. These paragraphs encourage caution for such cases, and (5) offer a simple calculation which may be very conservative in some cases.

C6.5.2.3 Semi-empirical method
Semi-empirical methods are those which rely on an established correlation between a test result and the ultimate bearing capacity. It is important not to extrapolate correlations beyond 'comparable experience' as defined in 1.4.2(1)P. Annex C provides an outline method for the use of pressuremeter tests.

C6.5.3 Failure by sliding
C6.5.3(2)P to (3)
It was noted in B2.1 that ultimate limit states may occur even when the ground has not reached the limits of its strength, that is, without the formation of a complete failure mechanism in the ground. Hence it is necessary to consider the *displacement appropriate to the limit state considered.* For this reason, E_{pd} may not necessarily be the limiting passive resistance of the ground. On the other hand, it is likely that the shear resistance, S_d will be mobilised at its maximum available value with relatively little movement though it could possibly reduce as large movements take place. Hence it may not be possible to obtain the maximum values of S_d and E_{pd} simultaneously.

Passive resistance to relatively shallow bases of retaining walls may easily be lost or much reduced by local excavations or erosion. This possibility is to be considered. For retaining walls, a more prescriptive approach is adopted in 8.3.2.1(2).

C6.5.3(4)P
Error
The reference to Inequality (6.5) is an error. It should be (6.2).

C6.5.3(8)
It should normally be assumed that the soil at an interface with concrete construction will be disturbed. Hence, critical state angles of shearing resistance are relevant to the interface, even if higher angles, taking advantage of the density of the soil, are used within the main body of soil. Thus the characteristic value of ϕ'_k, for the interface, should be a cautious estimate of the critical state angle of shearing resistance. This is used to derive ϕ'_d for the interface and hence δ_d.

C6.5.3(9)P
This paragraph is mainly relevant to sliding of bases on stiff clays or weak rocks, for which undrained strength could be relevant, especially for rapid loading cases. Equation 6.4 is a standard requirement based on undrained shear strength. The contact area is reduced in cases of eccentric loading. The relevant value of c_u is that available at the interface between the structure and the soil. It would be preferable to refer to this as 'adhesion'.

In some circumstances the vertical load from the foundation may be insufficient to give a large contact area. Consider, for example, a light precast concrete base pushed horizontally by a large force, as shown in Figure C6.5. In order to calculate the available horizontal resistance, the contact area must be derived as a function of the vertical force and vertical bearing capacity, with allowance for load inclination. Mortensen (1983) has shown that calculations of this type

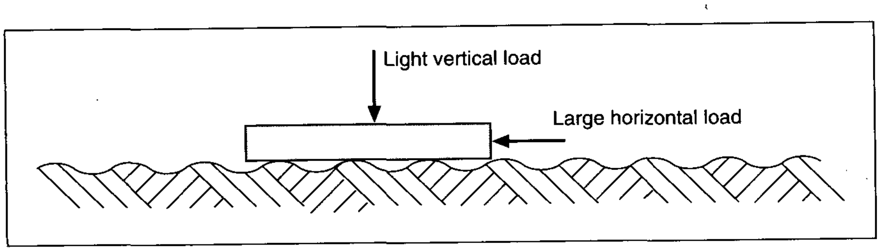


Figure C6.5 Light precast footing subjected to horizontal load

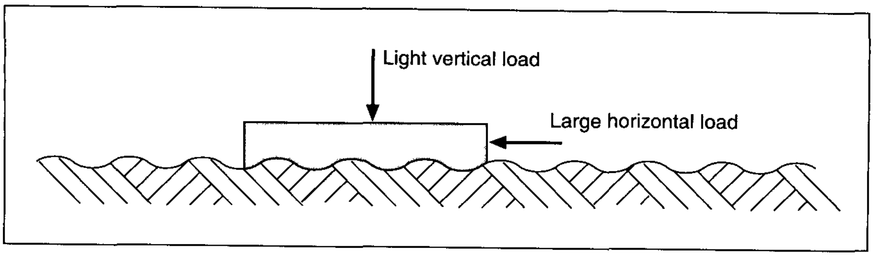


Figure C6.6 Cast in situ footing subject to rapid transient horizontal load

give a result which can be summarised as Inequality 6.5.

In some cases of rapid transient loading, for footings which fit intimately onto the ground surface, suction may ensure that no gap can form between footing and ground. In this case, Inequality 6.5 can be disregarded. An example is shown in Figure C6.6.

C6.5.4 Loads with large eccentricities

C6.5.4(1)P

EC7 does not place a specific limit on the degree of eccentricity of the load on a foundation. For example, it does not have a 'middle third rule'. This paragraph says that special precautions are required where the resultant force lies outside the middle two-thirds of a footing, as illustrated in Figure C6.7. Even this may be allowable where a relatively large footing is subject a comparatively small vertical load, but with high eccentricity. Where the line of action of the resultant load will be close to the edge of the footing, emphasis is placed on ensuring accuracy of construction and allowing a reasonable tolerance.

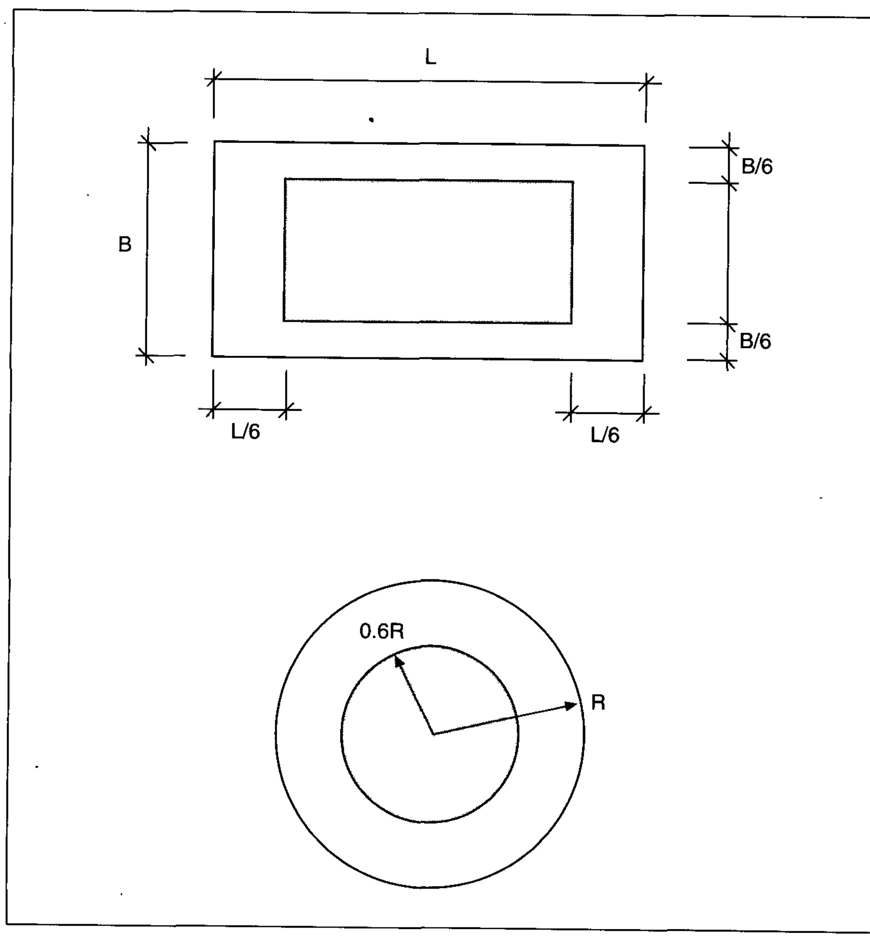


Figure C6.7 Definition of 'large eccentricities'

The authors of this commentary suggest that the middle third rule is a useful and conservative first check, on the basis of SLS forces and material properties. However, it is not a code requirement. Examples are presented in E4 and E5.

Error

The combination of application rule and principle in (2) and (3)P does not make sense.

C6.5.5 Structural failure due to foundation movement

The code recognises that an ultimate limit state may be caused in a supported structure by displacements in the ground, even if there is no plastic mechanism within the ground itself. It is difficult to give general advice for this situation and the code is not very successful in providing application rules. It is important to recognise, however, that ground movements may lead to states far more serious than normally accepted under the definition of serviceability limit state.

Common examples of severely damaging ground movements not caused by plastic mechanisms include:

a large settlements of soft clays due to loading beyond the reconsolidation pressure;
b severe settlements of clays caused by desiccation due to tree roots;
c swelling of clays due to the removal of trees or, in southern Europe, simply due to seasonal variations in precipitation and evaporation;
d settlement of loose fill due to inundation by water.

Neither the use of presumed bearing values nor the application of partial factors can deal with heave or more complex problems. These must be treated individually.

It is conceivable that quasi-elastic settlements of the ground could cause ultimate limit state failures of supported structures. However, examples of this are hard to find.

The suggestion in Paragraph (2) is that the use of presumed bearing pressures derived from comparable, preferably local, experience may be the best way to avoid settlement problems. The application of the boxed values of the partial factors probably achieves the same thing, at least for drained soil of reasonable density and stiffness. However, for clays which are loaded rapidly enough to remain undrained, the boxed values for c_u are fairly small and may be insufficient to prevent unacceptable settlement due to the approach of plastic failure. For soft clays, partial factors provide no protection against limit states due to drained settlement. This is discussed further in relation to serviceability limit states under 6.6.1 below.

It is always sensible to make an assessment of foundation settlement, either from experience or by calculation, for the worst conceivable loadings, to check that there is no danger of an ultimate limit state in the supported structure.

C6.6 Serviceability limit state design

The relationship of ground movement to ultimate and serviceability limit states is discussed in B2.1. This clause, under the heading of serviceability limit state, is equally applicable to ultimate limit states caused by ground movement remote from shear failure. The only exception to this is 6.6(2)P, since for ultimate limit states ULS design loads must be used.

In Paragraph 8.4(2), for retaining walls, it is stated that *the design methods and factors of safety required by this code for ultimate limit state design are often sufficient to prevent the occurrence of this type of limit state* (ie serviceability) *provided the soils involved are at least medium dense or firm, and adequate construction methods and sequences are adopted.* In general, the same is probably true for spread foundations on drained soils, if the boxed values are used for the partial factors on soil strength. This is an example of the approach discussed in 2.4.1(7)P. However, for undrained clays the boxed values are fairly small and may be insufficient to prevent unacceptable settlement, if there is a serviceability requirement for the undrained state.

The 6 paragraphs given here are relatively straightforward. The essential features to note are:

a An assessment shall always be made of the vertical and horizontal displacements. The word 'assess' does not necessarily imply calculation; an estimate based on appropriate experience would comply – see C1.5.2(1)P.
b Serviceability limit state design loads are to be used for this assessment. In general, these are equal to characteristic loads because the partial load

factors are unity (2.4.2(18)P). In some situations, it will be appropriate to use combination factors ψ, defined in EC1, 9.4.2, to obtain reduced values of the design loads. Similarly, the soil material properties are unfactored (EC7, 2.4.3(13)P).

C6.6.1 Settlement

C6.6.1(2)

The final two paragraphs should be read together. The assumption that a 20% increase in effective overburden stress may be considered negligible is not valid for very soft soils, especially if this takes the cumulative effective stress over the reconsolidation pressure.

C6.6.1(6)

See comments on Annex D.

C6.6.1(8)

Error

'Shall' should be 'should' in the first line of the final paragraph.

C6.6.2 Vibration analysis

The statements here are very basic. The reader must consult geotechnical text books for more information.

C6.7 Foundations on rock: additional design considerations

The code provides basic statements on foundations on rock. Design is dependent mainly on engineering geological observation of the state of the rock with the use of presumed bearing pressures based on comparable experience.

Following criticism, particularly from southern European countries, it is likely that the treatment of foundations on rock will be extended in future drafts of EC7.

C6.8 Structural design of spread foundations

As for the geotechnical calculations, structural calculations should in principle be carried out for all three cases A, B and C. However, in most situations it will be obvious that the loading of Case B governs the structural design of spread foundations. The geometry of the foundation may well be determined by Case C, however. It is important that the structural design is then carried out for the piece of structure which will actually be built. This implies that Case B loading must be applied to the geometry derived from Case C, which will often lead to larger bending moments in the footing than would result for the smaller geometry required for Case B.

This process is exactly equivalent to the use of BS 8110 for the structural design of foundations, in which the size of the foundations has been determined by other, geotechnical calculations. Some reduction of the bending moments might be justified by considering flexure of the foundations, but this is normally done only for rafts.

Examples are provided in E2 to E4.

Eurocode 2 Part 3 contains more information about structural design of concrete in the ground. However, this partly repeats information already in both EC2-1 and EC7-1, and it is now considered unlikely that it will be part of the final Euronorm (EN) version of EC2. Some of the material in Part 3 will probably be transferred to EC2-1.

C6.8(2)

Error

The cross-reference to 2.1(8)P is pointless.

Eurocode 2 Part 3 contains more detailed specifications and helpful formulae relating to linear distributions of contact pressure beneath footings of various shape.

C7 PILE FOUNDATIONS

See the general comments at the start of Section 6.

This section of Eurocode 7 is quite long and designers of piles will usually only need to refer to parts of it. Its contents can be summarised as follows.

7.1 to 7.4	Introductory;
7.5	Pile load tests;
7.6	Piles in compression;
7.7	Piles in tension;
7.8	Transversely loaded piles;
7.9	Structural design of piles;
7.10	Supervision of construction.

Clauses 7.1 to 7.5 will not be particularly controversial to British designers. Thus, for design of compression piles, the only clause which introduces unfamiliar requirements is 7.6.

As noted under 2.1(5), pile design in the United Kingdom is unlikely to lie in Geotechnical Category 1.

C7.1 General

C7.2 Limit states

In this list, the term 'overall stability' refers to the stability of the site or building as a whole, taking part of the piled structure into a general slope stability failure. It is not clear whether the term unacceptable vibrations refers to vibration due to vibratory loads on the piles in service or to vibration due to pile driving.

C7.3 Actions and design situations

C7.3.1 General

C7.3.1(1)P

Section 7 of the ENV was drafted before the text on Cases B and C was prepared in Subclause 2.4.2. There is therefore an incompatibility between these two sections. In principle, Section 7 is consistent with Case C, but note that 2.4.3(12)P states that the partial factors for soil strength parameters given in Section 2 are not to be used for piles. All three Cases A, B and C should be checked, though experience will quickly show that for most common situations Case C always governs the geotechnical design of axially loaded piles. (Exceptions occur for driven piles or bored, shaft controlled piles, when the dominant load is dead load. In this situation, using the boxed values of the partial factors, Case B governs by a small margin. This is an unintended anomaly which may be rectified in later versions of EC7.) Case B usually governs structural design. In situations of buoyant loads or transverse loads, it may be found that Cases A, B or C govern the geotechnical design. This is illustrated by E12.

C7.3.1(3)

The need for high characteristic values in problems involving downdrag is noted under 7.3.2.1(1)P.

When an interaction analysis is used to determine load distribution in a large group of piles, the effect of both hard spots and soft spots should be considered, represented by upper and lower characteristic values of strength and stiffness parameters. It can often be argued that the geotechnical design should not be changed to account for these, but there may be need to consider them in structural design. A relevant case study, but related to shallow foundations, was presented by Simpson (1992).

C7.3.2 Actions due to ground displacement

C7.3.2.1 General

C7.3.2.1(1)P

The treatment of downdrag and similar displacement-driven effects challenges definitions and understanding of the term 'action'. Reference to C2.4.2 and the clauses below is recommended.

Consideration of high characteristic values may be necessary in problems involving downdrag, where it is conservative to take an upper estimate of the magnitude of the downdrag. EC7 does not give values for the partial factors needed to derive design values from upper bound characteristic strengths. Reciprocals of the values in Table 2.1 are sometimes used.

C7.3.2.1(2)P

The designer is allowed to chose between two approaches, treating either forces or displacements as the basic action. The more economic approach may be chosen in each circumstance. An example of this is presented below under C7.3.2.2.

C7.3.2.2 Downdrag (negative skin friction)

Downdrag will usually be analysed by calculating the maximum force which could be generated by negative skin friction, following Paragraph (2), and treating this as an action (Paragraph (1)P). This implies that the force will be multiplied by the factors for actions given in Table 2.1. Normally downdrag will be regarded as a permanent action because its 'variation is always in the same direction (monotonic) until the action attains a certain limit value' (EC1, 1.5.3.3). In circumstances where variable loads of short duration are also applied to the pile, it may be justifiable to assume that the downdrag and variable loads cannot occur at the same time. However, the cumulative effect of repeated loading should also be considered.

Where ground movement is treated as the action, its design value must be assessed directly in accordance with EC7, 2.4.2(19, 20). In many cases, increasing the magnitude of this movement by a factor would, in any case, have only a small effect on the force transmitted to the pile. The interaction between moving ground and the pile may be analysed using finite elements or other simpler approaches.

In all cases of downdrag, the characteristic value of the shaft adhesion should be a cautious upper value. When ground movement is treated as an action, then in Case C the characteristic shaft adhesion must be divided by a factor $\gamma_m < 1$. Values which are reciprocals of the strength factors in Table 2.1 are suggested.

In E10, the two alternative approaches are considered on a fairly simple basis.

C7.3.2.3 Heave

C7.3.2.3(1)P

Only one specific instruction is given in this subclause: *the movement of the ground shall generally be treated as an action.* Unlike downdrag, heave movements are almost always small enough that a study of the interaction between heaving ground and pile is appropriate. This could be carried out by numerical analysis. Alternatively, a 'balance point' is found such that the pile is moving up more than the ground below the balance point but less than the ground above, and the whole system is in equilibrium.

This type of analysis is compared with results of a finite element analysis in E9.

C7.3.2.3(2)
EC7 has not been well developed for situations in which ground displacement is the action. In particular, no values are given for partial factors for situations where ground strength acts in a manner detrimental to the condition of the structure. An analysis generally consistent with 2.4.2 can be carried out for Case B by calculating the unfactored tension in the pile and multiplying this by 1.35 for ultimate limit state design. Lower bound values should be taken for any variable compressive forces at the head of the pile, possibly zero.

C7.3.2.4 Transverse loading
Loading of piles due to transverse ground movements is a difficult area normally requiring specialist analysis. It could well be categorised as Geotechnical Category 3. EC7 provides general requirements of the analysis.

C7.4 Design methods and design considerations

C7.4.1 Design methods

C7.4.1(1)P
This paragraph makes it clear that all pile design calculations must be related, directly or indirectly, to the results of static load tests. Where the design is based directly on static load tests, it must also be shown by calculations that the results are consistent with general experience. Pile designs may also be based directly either on analytical calculations or on dynamic load tests; however, in these cases, the calculation methods or interpretation of the dynamic load tests must have been validated against previous static load tests.

This paragraph guards against two errors. It is recognised that a purely theoretical calculation of pile capacity is likely to be unreliable, so calculation methods must be related to general experience of pile load testing. On the other hand, a small number of tests, perhaps only one, on a specific site might yield anomalous results, due either to some geological variation or poor testing technique. Thus blind reliance on tests, without backup calculation, is also not allowed.

C7.4.1(2)
Static load tests may be carried out on trial piles or working piles. Subclause 7.5.2.1 strongly implies that tests on compression piles need not necessarily be taken to failure, whilst those on tension piles always should be. Since Clause 7.6 requires an assessment of the ultimate bearing resistance of compression piles, it will be necessary to extrapolate the results of tests not taken to failure. The failure load should also be checked by calculation, in accordance with 7.6.3.3.

The observed performance of an existing piled foundation may also be used as a substitute for load tests. The paragraph suggests that this is particularly applicable to Geotechnical Category 1, where foundations simply become a copy of a design which has been successful locally. As noted in C2.1, calculations are usually not required for Geotechnical Category 1 designs, but pile design will generally not lie within Geotechnical Category 1 in the United Kingdom.

C7.4.2 Design considerations
This subclause contains some important lists of items which should be checked in the design of piles.

C7.4.2(2)P
Most conventional methods of pile design are adapted to conventional rates of testing and service loading. Only in unusual cases is it necessary to make specific allowances for rate effects.

C7.4.2(3)P
It is reasonable that designers should consider potential changes in groundwater regimes, but they should then decide whether to allow for them in the design. This may depend on whether their client is responsible for the changes of groundwater regime, or whether this is the responsibility of some other party. This paragraph, however, appears to require that any potential changes in groundwater regime are in fact taken into account in every design.

C7.5 Pile load tests

C7.5.1 General

C7.5.1(1)P
As noted under 7.4.1, it is not mandatory on every project to have pile load tests. Under some conditions, however, pile load tests are mandatory. These conditions are listed in this paragraph.

C7.5.1.2(2)
The second grammatical paragraph in this 'paragraph' is rather out of place. A separate section on special situations, such as cyclic loading, would perhaps be desirable in a future version of EC7.

C7.5.2 Static load tests

C7.5.2.1 Loading procedure

C7.5.2.1(1)P
It is clear from this paragraph that many pile load tests may not be taken to failure. Only for trial piles is it required that 'conclusions can be drawn' about ultimate failure load. This is not really consistent with the design methods given later (eg 7.6.3.2) which depend on ultimate resistance as measured in load tests. Paragraph 7.5.2.1(1)P states that the test procedure shall be such that the creep and rebound of a pile foundation can be assessed from the measurements on the pile. However, there is no specific mention of these items elsewhere in the chapter, except for settlement reducing piles in 7.6.3.1(4).

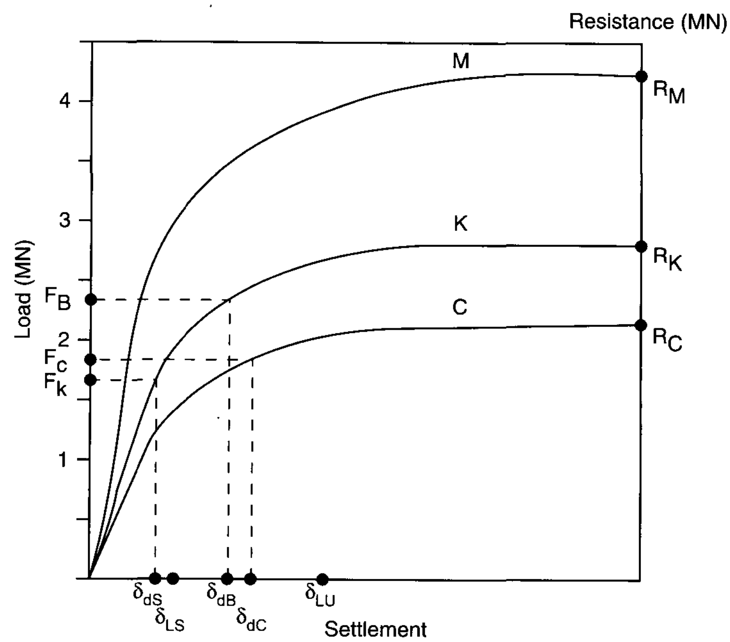
C7.5.2.1(2)
This paragraph breaks the CEN rules in that a reference is made to a document which is not a CEN or ISO publication. The text will probably therefore be changed in the final version of Eurocode 7.

The paragraph refers to the paper in the 'ASTM Geotechnical Testing Journal', (ASTM (1985)), 'Axial pile loading test, suggested method' (sic, actually 'Axial pile loading test – Part 1: static loading'). This was written before the 'Specification for piling and embedded retaining walls' (ICE (1996)), and also predates recent codes from other nations. Its requirements tend to be slightly less stringent than the ICE Specification. It also adds some background advisory information, including comments on various national practices.

C7.5.2.2 Trial piles

C7.5.2.3 Working piles

C7.5.2.3(1)P
This paragraph could lead to difficult negotiations between client, designer, and constructor since clients will usually be very unwilling to increase the number of working pile load tests during construction. However, it will often be possible to vary the number of working pile load tests within a preset maximum (even if the maximum is 1 and the minimum is 0!). There could be occasions when the designer has to inform other parties that unless the number of working pile tests is increased the design cannot comply with EC7. This type of situation has always existed, requiring negotiation and sometimes a strong will on the part of designers.



Curve M shows the measured load-settlement plot for a test on a single pile. By inspection, it is assessed that the shaft resistance of the pile is 2.6MN and the ultimate base resistance is 1.6MN.

Curve K is the characteristic load-settlement plot, obtained by dividing the force on Curve M by 1.5 (from Table 7.1). This is used without further factors for ULS Case B and for SLS design.

Curve C is derived from Curve K for ULS design to Case C. The shaft resistance is divided by 1.3 and the base resistance by 1.6 (Table 7.2).

The applied loads are as follows (in MN):

	Characteristic	ULS Case B	ULS Case C
Permanent	1.20	1.62	1.20
Variable	0.50	0.75	0.65
Total design loads	$F_k = 1.70$	$F_B = 2.37$	$F_C = 1.85$

From these design loads and the three curves, design settlements can be derived as shown on the figure: δ_{ds} for serviceability loads (= characteristic in this case), and δ_{dB} and δ_{dC} for ULS cases B and C.

The design requirement is then that the design settlements must not exceed the limiting values of the settlements, δ_{LS} for serviceability, and δ_{LU} for ultimate limit state. That is:

$$\delta_{dS} \leq \delta_{LS} \text{ and both } \delta_{dB} \text{ and } \delta_{dC} \leq \delta_{LU}$$

The values set for δ_{LS} and δ_{LU} depend on the supported structure (see 2.4.6). In some cases, there may be no limit on the settlement for ULS, in which case δ_{LU} is simply "large" and the ULS requirement becomes, in effect:

$$F_B \leq R_k \text{ and } F_c \leq R_C$$

where R_k and R_c are the ultimate resistances on the characteristic and Case C curves, as shown in the figure.

Figure C7.1 Use of measured, characteristic and factored load-settlement curve for a compression pile

C7.5.3 Dynamic load tests

Eurocode 7 allows design based on dynamic load tests, under the restricted circumstances described in (1)P. The rather cryptic requirement of (2)P is explained in (3).

C7.5.4 Load test report

C7.6 Piles in compression

C7.6.1 Limit state design

Paragraph (1)P reflects the more general Clause 7.2 but selects the specific limit states relevant to compression piles.

It is noted that an ultimate limit state might occur in a supported structure due to displacement of the piled foundation, even if the piled foundation has not itself reached its ultimate capacity. Paragraph (2) states that this problem should be addressed, when necessary, by using a factored load-settlement curve. Figure C7.1 shows an example of how a measured load-settlement curve could be transformed firstly to a characteristic curve, then to a design curve for assessment of ultimate limit states.

C7.6.2 Overall stability

C7.6.2(2)

Where there is a possibility of a failure surface intersecting the piles, a soil-structure interaction analysis will probably be needed, allowing for the bending resistance of the piles helping to resist the failure.

C7.6.3 Bearing resistance

C7.6.3.1 General

C7.6.3.1(1)P

Since the design load (or action) and design resistance already contain partial factors, no further factor of safety is required in Inequality 7.1.

C7.6.3.1(3)P

For piles in groups, only two requirements have been recognised by the code drafters: bearing resistance failure of piles individually and bearing resistance block failure. Other requirements, such as those proposed by AASHTO (1993 and 1992) for overall stability and settlement have not been recognised. However, (5)P, (7)P and (8) should also be considered. Paragraph (4) makes it clear that block failure is to be analysed as a single pile of large diameter.

C7.6.3.1(4)

The concept and use of 'creep load' is discussed in ASTM (1985).

C7.6.3.1(5)P

For ultimate bearing resistance, the only potential adverse effect of adjacent piles is that of the group effect in (3)P and (4). It is not clear to what further effect this paragraph refers.

C7.6.3.1(8)

The subclauses which follow deal essentially with the design of isolated piles. This paragraph makes it clear that the general procedure will require some interpretation when multiple piles act together to support a foundation. The study reported by Cooke et al (1981) is relevant here.

C7.6.3.2 Ultimate bearing resistance from pile load tests

C7.6.3.2(4)P

The basis of this paragraph is unclear. If it is considered that positive skin friction is always developed at ultimate failure, then the pile resistance measured in the test should be used without correction. However, if it is considered that ultimate failure may be accompanied by negative skin friction, then the resistance measured in the test should be modified by subtracting an amount approaching twice the skin friction in the potentially settling layer. The latter view seems to be taken up in (5).

C7.6.3.2(6)P

The use of this paragraph is illustrated in E7.

C7.6.3.2(7)

Paragraph 6(P) specifies minimum values for reduction factors ξ to be applied to measured ultimate resistances in deriving characteristic values. Paragraph (7) discusses variability in ground condition which is known or at least suspected. It leaves some discretion with the designer in deciding whether this is sufficiently accounted for by the factor ξ as specified in Table 7.1. As a simple conservative rule, measured ultimate resistances could first be modified to allow for any known variabilities, then the factor ξ taken from Table 7.1 could be applied to derive characteristic values.

C7.6.3.2(8)P and (9)

Usually, the load on a pile under test is only measured at the top of the pile. However, it is often possible to make a reasonable assessment of the separate components of base and shaft resistance. Paragraph (9) suggests that the ratio of the base and shaft components might be derived by calculation, though their sum is based on the measured results.

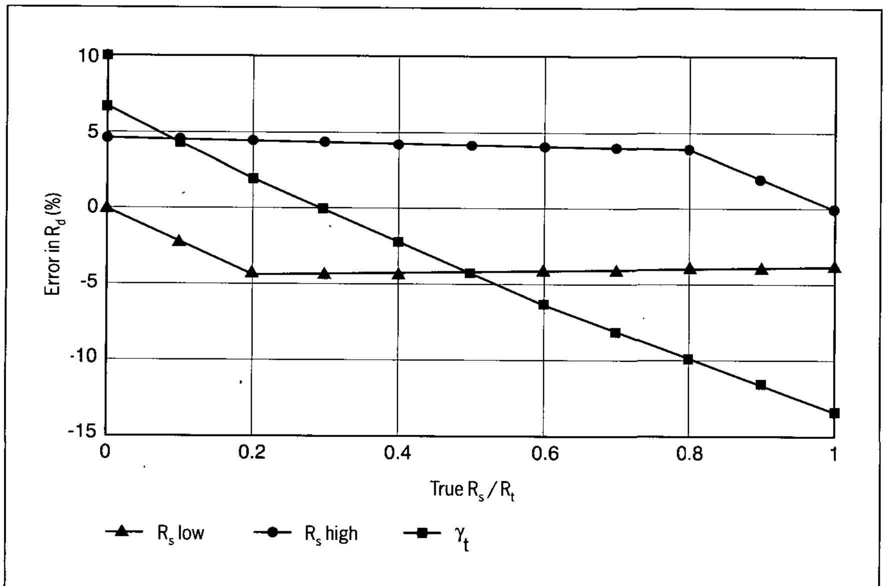


Figure C7.2 Effect of an inaccurate assessment of the share of resistance between shaft and base on calculated design resistance of a compression pile

The effect of an error in this estimate is shown in Figure C7.2. The error in the design resistance R_d is plotted against the true ratio of shaft resistance R_s to total resistance R_t, for situations where the ratio R_s / R_t has been wrongly estimated. Even with the estimated R_s / R_t in error by as much as $\pm$ 20%, the error in the design resistance is never more than 5%, so is of little importance.

Figure C7.2 also shows the 'error', or difference in design resistance, obtained by using the Paragraph (11), with a single partial factor $\gamma_t = 1.5$. In this case the design resistance of the pile may be underestimated by up to 13%, giving a less economic pile. For piles which are base-dominated (R_s / R_t small) a more economic result may be obtained using $\gamma_t = 1.5$ rather than $\gamma_b = 1.6$. This was not intended in the drafting of this clause and it is recommended that $\gamma_b = 1.6$ should be used in this case.

The use of Table 7.2 is illustrated further in E8.

C7.6.3.2(10)P

Table 7.2 allows somewhat lower factors to be used for shaft resistance than for the bases of bored piles, but for driven piles a constant factor is used. These factors refer to Case C loading and, when taken with the ξ factors in Table 7.1 give overall factors in the range 1.95 to 2.4 when a single pile has been tested and the loading is entirely permanent loads. The overall factor becomes larger when there is a significant variable load. When several piles are tested, however, the overall factor may drop as low as 1.69 on the mean test result or 1.43 on the lowest test result; higher values apply to the base components of bored piles.

For Case B calculations, the ξ factors in Table 7.1 should be applied, but the γ factors in Table 7.2 are replaced by 1.0.

C7.6.3.2(11)

Where it is not possible to distinguish base and shaft resistance, this paragraph provides an alternative way of applying the partial factors, using γ_t from Table 7.2. Except for cases where shaft resistance is very much less than base resistance, this approach will lead to a more conservative design than that of Paragraphs (8)P to (10)P. It is recommended that (11) should never be used to obtain a more economic design than would be achieved from a plausible application of (8)P to (10)P.

C7.6.3.3 Ultimate bearing resistance from ground test results

In this approach, design shaft and base resistance are obtained from characteristic shaft and base resistances by applying the same partial factors as used for the method based on load testing (Table 7.2). These factors are applied directly to shaft and base resistances, not to fundamental soil parameters. The reason for this is that the link between measured soil parameters and best estimates of shaft and base resistance is often fairly tenuous. Hence the engineer is asked to make the best estimate of shaft and base resistance, without any specific guidance from the code, and to divide the best estimate value by [1.5] in order to achieve a characteristic value. The precise wording of this approach in (4)P should be studied.

Sample calculations are given in E2 to E4.

C7.6.3.3(4)P to (6)

Although the method is based on the designer's assessment of base and shaft resistance, usually by calculation, it is related very strongly to results of static load tests. It is assumed that calculation rules have been derived by studying the results of load tests and an allowance of [1.5] is made for the likely variability of the test.

C7.6.3.4 Ultimate bearing resistance from pile driving formulae

Summary

Pile driving formulae may be used as the basis of pile design, provided the formulae have been very thoroughly calibrated in very similar ground conditions.

C7.6.3.5 Ultimate bearing resistance from wave equation analysis

Summary

Wave equation analysis may be used as the basis of pile design provided that it has been very well calibrated in very similar conditions.

C7.6.4 Settlement of pile foundations

This subclause requires that settlements should be 'assessed' (see C1.5.2). The assessment can probably be made from experience or from pile test results, and should not normally require extensive analysis. It will normally apply to the serviceability limit states, though it is noted that settlement of piles might cause ultimate limit states in the supported structure. In these cases, (2)P requires that the whole load-settlement curve should be downgraded from characteristic to design values, as discussed in C7.6.1.

C7.7 Piles in tension

The procedure for designing piles in tension is similar to that for piles in compression, except that the only design methods recognised are based on load tests or ground test results. The tensile resistance will normally be derived from the shaft resistance only. The partial factor applied will generally be greater than that used for compression piles; the ENV (and British NAD) gives a value of 1.6 in 7.7.2.2(4)P. Designers using Clause 7.7 should first be familiar with 7.6 for compression piles.

C7.7.1 General

C7.7.2 Ultimate tensile resistance

C7.7.2.1 General

C7.7.2.2 Ultimate tensile resistance from pile load tests

C7.7.2.3 Ultimate tensile resistance from ground test results

No value is given for the partial factor γ_m to be used in calculating tensile resistance from ground test results. Similarly, no factor ξ is specified relating calculations to characteristic values.

Paragraph (2)P could be taken to mean that **design** values for tensile resistance are to be assessed directly by the designer. However, it is recommended that the values for ξ and γ_m in 7.7.2.2 should be used as a guide. The factor ξ may be taken as the ratio between best estimate tensile resistance and characteristic value, as in 7.6.3.3(4)P.

C7.7.2.3(3)

'Annex G' should read 'Annex F'. This paragraph is out of place; Annex F refers primarily to structural design of piles.

An example of the use of Annex F is given in E11.

C7.7.3 Vertical displacement

C7.7.3(2)

In cases where very long tension piles are used, the elastic stretching of the piles themselves may exceed serviceability limits. It is sometimes necessary to increase reinforcement to prevent this. In some designs mild steel reinforcement, working at relatively low stresses, is adopted for this purpose.

C7.8 Transversely loaded piles

C7.8.1 General

C7.8.2 Ultimate transverse load resistance

C7.8.2.1 General

C7.8.2.1(2)

A standard approach, originally published by Broms, is given in Tomlinson (1994).

C7.8.2.2 Ultimate transverse load resistance from pile load test

C7.8.2.3 Ultimate transverse load resistance from ground test results and pile strength parameters

C7.8.3 Transverse displacement

C7.9 Structural design of piles

C7.9(1)P

Clause 2.4 requires calculations for all 3 cases A, B and C, as discussed earlier under C7.3.1(1)P. An example is given in E12.

C7.9(2)P

The calculation method presented in Annex F shows how the design tensile force in the pile might vary over the length, reducing towards the base of the pile. This is based on design soil strength, but makes no allowance for soil stiffness. In extreme cases, where the pile extends over most of its length through highly deformable ground but reaches very stiff ground, it might be necessary to assume that the full tensile force must be carried over the whole length of the pile. In general, this assumption might be too severe, though in practical design it could be preferable to provide the same reinforcement over the whole length.

C7.10 Supervision of construction

The requirements of this clause are generally consistent with 'Specification for piling and embedded retaining walls' (ICE (1996)).

C7.10(5)P

The requirement that records be kept for at least 5 years is not included in the ICE Specification, but the CDM Regulations (Health and Safety Commission (1994)) implicitly require that as-built drawings are retained throughout the life of the structure. These will normally contain a summary of the data recorded during construction, including ground stratification.

C8 RETAINING STRUCTURES

Summary

This section considers the design of all types of earth retaining structures, and includes a clause on anchorages. It relies on Sections 6 and 7 for the foundations of the structures (EC7, 8.2(4)) and on Section 9 for requirements of overall stability.

The section has some similarities with BS 8002, but also some differences. Appendix 3 lists the main items required for retaining wall calculations, comparing BS 8002 and Eurocode 7. In particular, Eurocode 7 mentions 'unplanned overdig' (8.3.2.1), but its requirements are slightly different from those of BS 8002 and they are applied only to ultimate limit state calculations. Eurocode 7 does not have a minimum surcharge to be applied to the retained ground.

Eurocode 7 applies partial safety factors, taken from Section 2, for ultimate limit state design but uses unit factors for serviceability limit states. This contrasts strongly with the approach of BS 8002. However, both codes point out that for structural design, even at ultimate limit state, it may be necessary to consider earth pressures greater than limiting active values where the supported soil is overconsolidated or compacted and the structure is fairly rigid.

See general comments at the start of Section C6.

C8.1 General

C8.1(1)P

Structures which retain water are included in this clause with regard to their strength and stability, but watertight design is not included.

C8.2 Limit states

C8.2(1)P

The final 3 items in the check list do not refer directly to the strength or stability of the wall. However, if flow of water or transport of soil particles is allowed to develop, instability and damage may ensue.

C8.2(4)

The bases of gravity walls are spread foundations, often subject to markedly eccentric loading. It was noted in C6.2 that no specific calculation is required for 'overturning' since this is regarded as a form of bearing capacity failure. The particular requirements of 6.5.4 for highly eccentric loading are important, however. See C6.5.4(1).

C8.3 Actions, geometrical data and design situations

C8.3.1 Actions

Besides listing relevant actions, 2.4.2 also requires the use of Cases A, B and C and the partial load factors to be applied. These are to be applied to retaining structures, with the special note in 2.4.2(17) about the application of Case B. The clauses of Section 8 refer directly to 'design values' rather than characteristic values. These design values should generally be derived from characteristic values in accordance with 2.4.2.

C8.3.1.1 Weight of backfill material

C8.3.1.2 Surcharges

Unlike BS 8002, Eurocode 7 does not impose a minimum design surcharge. The authors recommend that designers make their own assessment of the characteristic surcharge. In some cases, especially where the retaining walls are small, the surcharge may be appreciably less than the rather severe value required by BS 8002.

C8.3.1.3 Weight of water

C8.3.1.4 Wave forces

C8.3.1.5 Supporting forces

For the purpose of checking the stability of a retaining wall, the force in a dead-man anchor will generally be a result of the wall-ground interaction and so it is not an action, in that calculation. However, a prestressed anchor could, in principle, have a prestress force which is chosen by the designer, together with an additional component which results from the wall-ground interaction. In this case, for the purpose of the stability check, the prestress is an action, and the additional component is a reaction, not an action. In practice, many prestressing systems are fairly extensible, so the 'reaction' component is small compared with the prestress.

It was noted in C2.4.2 that the same force may be a reaction in some calculations and an action in others. For prestressed anchors, for example, the 'reaction' to wall-ground interaction, could be regarded as an action for the purpose of the structural design of the wall. In this case it might be either permanent or, if dependent on other variable loading such as traffic or tides, it could be variable.

EC2, Table 2.2 gives values for partial factors on prestress actions in prestressed concrete, but separate values for ground anchors are not provided in the Eurocode system.

The sequence of evaluation of the various forces involved in design of ground anchors is discussed in C8.8.2.

C8.3.1.6 Collision forces

C8.3.1.7 Temperature effects

C8.3.1.7(2)

The effect of temperature on prop loads has been considered in the recent CIRIA report 'Prop loads: guidance on design' (Twine and Roscoe (1997)).

C8.3.2 Geometrical data

In most cases, small variations in geometrical data are considered to be accommodated by the safety elements, mainly partial factors, included in the calculations (C2.4.5). However, because the design of retaining walls is extremely sensitive to ground levels and water levels, special requirements are included in this subclause. (In 6.5.4, a similar exception was made for loads with large eccentricities, for which a direct allowance for construction tolerance of a spread foundation was required.)

C8.3.2.1 Ground surfaces

C8.3.2.1(2)

This paragraph requires that the ground level of passive soil should be assumed to be slightly lower than the lowest the designer expects to occur. This is not intended to give the designer or constructor permission to over-excavâte in front of the wall. Rather, it is an allowance for the unforeseen activities of nature or humans who have no technical appreciation of the stability requirements of the wall.

The requirements of EC7 are similar to those of BS 8002, but not identical. BS 8002 requires a **minimum** allowance of 0.5 m, whereas Eurocode 7 sets 0.5 m as a **maximum**. This paragraph is an application rule, and so *It is permissible to use alternative rules different from the application rules given in this Eurocode, provided it is shown that the alternative rules accord with the relevant principles* (1.3(5)P). In 8.3.2.1, (2) is an application rule of the principle (1)P, so some discretion is left with the designer. An alternative wording of this paragraph has been proposed by Krebs Ovesen and Simpson as follows (the main changes being shown in bold type):

In ultimate limit state calculations in which the stability of a retaining wall depends on the passive resistance of the ground in front of the structure, the ground level of the passive soil should **be lowered below the nominal, expected level by an amount Δ_a. The value of Δ_a should be selected taking account of the degree of control to be exerted on site over the level of the surface. For situations with a normal degree of control, the following should be applied:**

a *for a cantilever wall, Δ_a should equal [10%] of its height, limited to a maximum of [0.5] m.*

b *for a supported wall, Δ_a should equal [10%] of the height beneath the lowest support, limited to a maximum of [0.5] m.*

Smaller values of Δ_a, including zero, may be used where the surface level is to be controlled reliably throughout the period in which it is operational.

Larger values of Δ_a should be used where the surface level is particularly uncertain.

It is recommended that this ground level reduction should only be disregarded with great caution, particularly where embedded walls rely heavily on relatively short penetrations into the restraining soil. Using the EC7 boxed values for soil strength factors, Simpson (1994) showed that for small penetrations the effect of this allowance was to give an equivalent factor of safety on passive pressure exceeding 2, whereas it drops to about 1.5 without this allowance. This is illustrated in Figure C8.1. Hence, if the ground level allowance is disregarded, the values of partial factors to be used in the calculations may need amendment.

The original purpose of this allowance was described by Simpson (1992). Embedded walls which have only small penetrations into soils with high shear strength (or high angles of shearing resistance) are very sensitive to any reduction in the ground level in front of the wall. Figure C8.2 compares two excavations in dense sand or gravel ($\phi_k' = 40°$). In excavation (a), the wall would be at failure (for ϕ_k') with a penetration of 1.25 m, but with a penetration of 2.0 m it would have reasonable factors of safety of 1.2 on $\tan\phi'$ or 2.0 on passive pressure (note that EC7 requires 1.25 on $\tan\phi'$). However, excavation (b) shows that if the excavation is taken only 0.5 m deeper by any accidental or natural process, the wall again reaches limiting stability. The dramatic effect of this small 'overdig' has led the drafters of both EC7 and BS 8002 to require that a direct allowance is made for it.

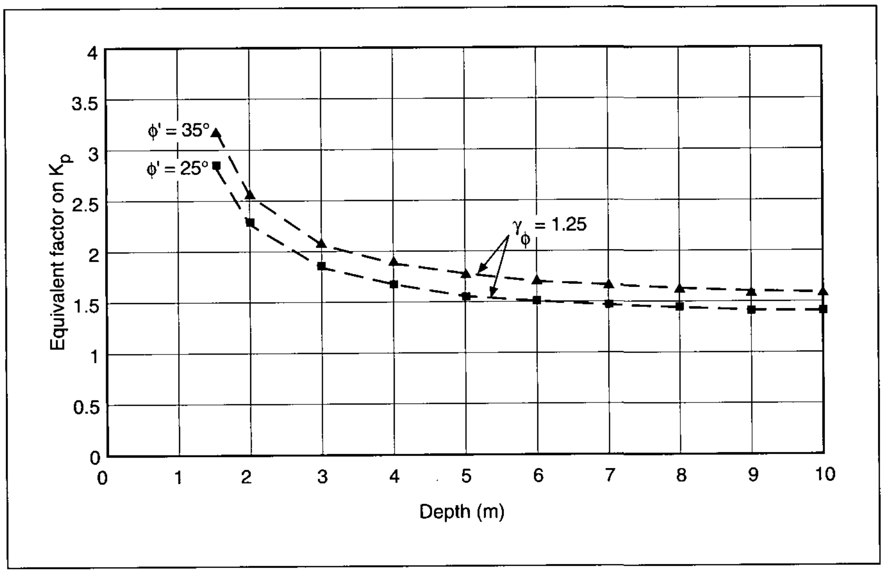


Figure C8.1 Overall factor of safety on K_p derived from boxed values with overdig allowance (after Simpson (1994))

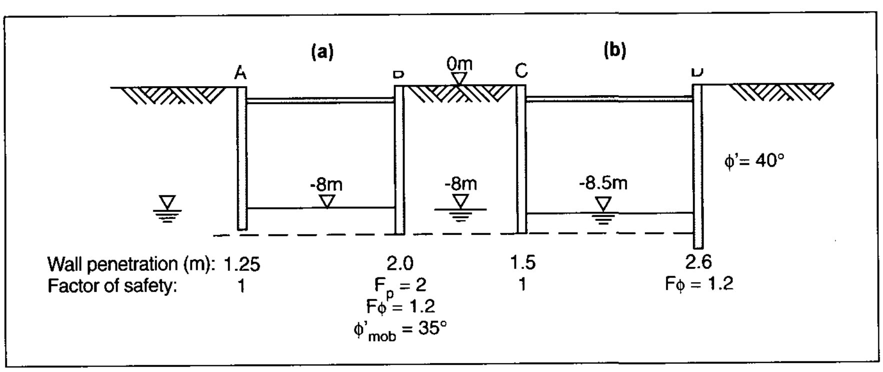


Figure C8.2 Effect of overdig on factor of safety of a wall with small penetration into dense granular material (after Simpson (1992))

Terzaghi (1954) recommended a specific allowance for 'overdig': the depth of penetration of sheet pile walls was to be increased by 20% to allow for 'the effects of unintentional excess dredging, unanticipated local scour, and the presence of pockets of exceptionally weak material'.

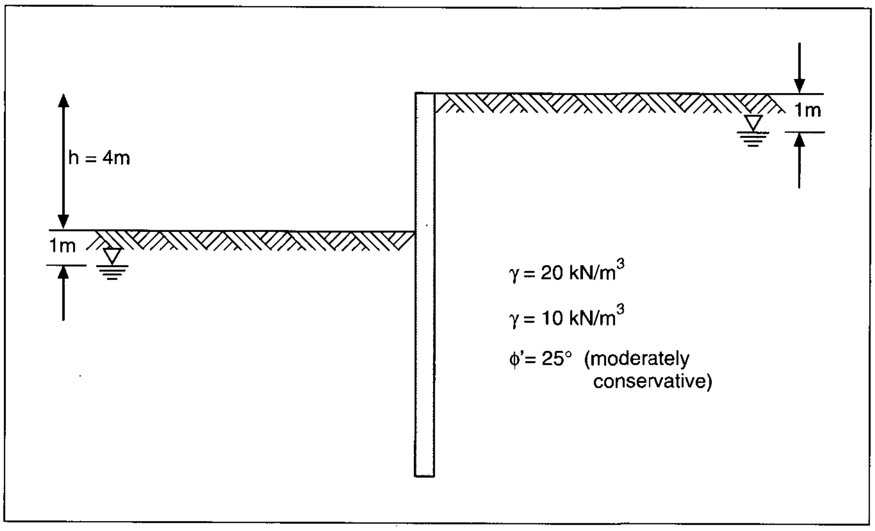


Figure C8.3 Cantilever wall example (CIRIA Report 104, Example B2)

Figure C8.3 shows a simple cantilever retaining wall reproduced from CIRIA Report 104 example B2, which is considered in some detail in E14. Some results for this wall, calculated using simple active and passive pressures, are shown in Table C8.1. It has been assumed that the 'moderately conservative' parameters of CIRIA 104 are equivalent to EC7's 'characteristic' values. In Columns 1 and 2, Eurocode 7 Case C designs are compared with and without overdig of 0.4 m, 10% of the retained height. It can be seen that the effect of overdig is to increase the length of the wall by about 20%, but, more importantly to increase the bending moment in the wall by about 60%. For comparison, CIRIA Report 104 requires that the bending movement is calculated initially using unit factors of safety, for which Columns 3 and 4 show that the effect of overdig is to increase wall length by 20% and bending moment by 70%. These results make it very clear that a fairly modest reduction in the level of the passive material requires a very much stronger wall of somewhat greater length. The requirements of Eurocode 7 for unplanned overdig should therefore be set aside only in exceptional circumstances and with great caution.

Table C8.1 The importance of 'overdig'

Case	EC7 Case C overdig Column 1	EC7 Case C no overdig Column 2	CIRIA F = 1 overdig Column 3	CIRIA F = 1 Column 4
$\gamma_\phi = F_s$	1.25	1.25	1.0	1.0
ϕ'_d	20.5°	20.5°	25°	25°
Overdig (m)	0.4	0	0.4	0
Length (m)	14.42	12.28	11.95	10.0
BM (kNm/m)	808	507	511 (= 303 × 1.7)	303
BM factor	1.0	1.0	1.5 ??	1.5
ULS BM (kNm/m)	808	507	767 ??	455

C8.3.2.2 Water levels

Information on design water levels and water pressures relevant to retaining wall design is dispersed throughout the code. Water levels (a geometrical parameter) are considered here and in 2.4.2(11), whereas water pressures are considered in 2.4.2(8)P to 2.4.2(10)P and in 8.5.6. The effect of drainage is considered in 8.4(5)P and (6)P. Paragraph 2.4.2(10)P makes it clear that a distinction can be made in the severity of assumptions used for ultimate and serviceability limit states. Subclause 8.5.6 considers water pressures from the point of view of practical soil mechanics. A practical example of normal and extreme water levels, not related to retaining walls, is presented in E12.

C8.3.3 Design situations

C8.3.3(1)P

The effect of future structures and surcharge loadings is to be 'considered' in the design. However, the extent to which they should be accommodated by designs will depend on the legal and contractual situation.

C8.4 Design and construction considerations

C8.4(1)P

Error

The cross reference to 2.1 is thought to be incorrect. It should probably be 2.4.1.

C8.5 Determination of earth and water pressures

C8.5.1 Design earth pressures

Many of the subclauses in this clause refer to states of earth pressure which are not at limiting active or passive values. These are generally not used in sizing the geometry of retaining structures. However, they are relevant to assessment of ground movement, generally serviceability limit states, and to structural design for both ultimate and serviceability limit states.

C8.5.1(4)

The parameter values referred to here are all **design** values, factored according to Table 2.1.

This paragraph allows the ratio δ / ϕ' to be set to 1.0 for concrete cast against the ground, provided that the **design** critical state angle of shearing resistance is used for ϕ'. However, it also notes that the vertical equilibrium of the wall and the direction of vertical movements of the wall and ground should be considered. In many situations, this makes it impossible for δ / ϕ' to equal 1.0 in all the soil on both sides of the wall. If a lower value is adopted, such as $2/3$, it is much more likely that equilibrium can be achieved. It may therefore be wise to use $2/3$ unless vertical equilibrium and displacements are checked very rigorously.

The paragraph discusses $k = \delta / \phi'$ for sand and gravel, but not for clay in a drained state. However, its recommendations could probably be applied in this case. A more detailed discussion is provided by BS 8002 (2.2.8).

Figure 8.4 of the code illustrates the type of situation in which vertical equilibrium is most important to selection of δ / ϕ'. The wall is being required to support vertical loads and to transfer these into the ground in friction. This may involve wall friction in a direction which is adverse for the calculation of active pressure; that is it causes an increase in the active pressure. Calculations for the horizontal earth pressures and vertical load capacity of the wall must use consistent values and directions for wall friction.

No mention is made here of the angle of friction across the 'virtual back' of a gravity retaining wall. For walls with significant heels, it is usually assumed that the force crossing the virtual back is parallel to the ground surface; that is, there is no friction across the virtual back if the ground surface is horizontal. An example is presented in E13.

The term 'precast concrete' is misused here. It should be taken to include any form of concrete not cast directly against the soil.

C8.5.2 At-rest values of earth pressure

C8.5.2(2)

Several formulae of the type given in Equation 8.1 are discussed by Simpson et al (1979). These formulae are thought to be reasonably accurate for over-consolidation ratios up to about 10.

C8.5.3 Limit values of earth pressure

C8.5.3(1)P

The graphs in Annex G are copied from CIRIA Report 104 and are based on Caquot, Kerisel and Absi (1973). Larger scale versions of these graphs, based on Kerisel and Absi (1990) may be found in BS 8002. Annex G also gives equations for coefficients of active and passive pressure based on plasticity solutions. These equations are quite complex, but are suitable for use in computer programs.

C8.5.3(2)P

The most common item in which use of simple active and passive earth pressures may be unconservative is the calculation of strut loads. For this reason, CIRIA Report 104, which uses simple active and passive earth pressures, advises a factor of safety of 2 on calculated strut loads. Eurocode 7 does not apply a factor in this way, but requires in this paragraph that the effect on the distribution of earth pressure of kinematic constraints such as struts should be considered. This may be done in several ways:

a by rules of thumb (eg Terzaghi and Peck (1967) or EAU (1980));
b from field observations (eg Twine & Roscoe (1997)); or
c by numerical analysis. This could involve finite element or other software which takes account of pressure redistribution, such as FREW, SPOOKS, WALLAP etc.

See also C8.6.1(6)P.

C8.5.3(3)P

This paragraph should be read together with 8.5.1(4).

C8.5.4 Intermediate values of earth pressure

C8.5.5 Compaction effects

Earth pressures caused by compaction are not normally considered when sizing the geometry of a wall, but they may affect the structural design. A method for calculating earth pressures was published by Ingold (1979).

C8.5.6 Water pressures

See earlier comments on 8.3.2.2.

Eurocode 7 does not have a minimum wall pressure, such as the '30z' rule of CP2 ('5z' in kPa). Instead, it requires a careful review of the water pressures which could occur in a range of circumstances. In this respect, Eurocode 7 is similar to BS 8002.

C8.6 Ultimate limit state design

This clause covers ultimate limit states of failure in the ground, which usually determine the geometry of retaining walls, and ultimate limit states of structural failure. Generally, the requirements of ultimate limit state design will form the basis of most of the calculations required.

C8.6.1 General

C8.6.1(4)P

This paragraph cross-refers to 2.1(8)P and (9). Compatibility of deformations is particularly important where there are brittle materials or structural members. These could include light sheet pile sections, subject to local buckling, struts which may buckle, and many forms of connections between struts, ties and wall. If large movements are needed to release high in situ earth pressures, reinforced concrete sections could become damaged and suffer a reduction of strength.

In other cases, where there is no brittle behaviour involved, there is no limit to the displacement allowed in ULS calculations. The only requirement is that equilibrium is attained. Design calculations should, however, also consider that large displacements may cause a ULS in an adjacent structure.

C8.6.1(6)P

It is important to consider redistribution of earth pressure, both as a means of reducing calculated bending moments, and critically to avoid under-estimation of strut loads. If allowance is not made for redistribution, the requirement of CIRIA Report 104 becomes relevant and a large factor is applied to calculated strut loads, ie 2.0. See also C8.5.3(2)P.

C8.6.2 Overall stability

C8.6.3 Foundation failure of gravity walls

An example of the design of a gravity wall is given in E13.

C8.6.4 Rotational failure of embedded walls

Examples of the design of embedded walls are given in E14 and E15.

C8.6.5 Vertical failure of embedded walls

C8.6.5(3)P

Usually prestressing forces are well controlled and unlikely to increase after construction, unless free anchor lengths are very short (note 8.8.2(7)). Individual tendons may temporarily have high forces during stressing, but it is unlikely that the forces averaged over a reasonable number of anchors will significantly exceed the intended value. Paragraph 8.3.1.5(1)P points out that forces caused by prestressing operations are regarded as actions. In calculations, the factors on actions taken from Table 2.1 should therefore be applied.

C8.6.5(4)P, (5), (6)P

See C8.5.1(4).

C8.6.6 Structural design of retaining structures

Paragraph 8.5.1(6) points out that the earth pressures relevant to ultimate and serviceability limit state may be fundamentally different. From the point of view of practical structural design, it is generally necessary that the loads on structural elements for ultimate limit state calculations are greater than those for serviceability limit state. It is therefore recommended that the structural design should be checked for pressures in the retained soil obtained by factoring the serviceability limit state earth pressures (see comments on 8.7.4). This recommendation effectively means that Case B is replaced by a requirement that the permanent load factor [1.35] is applied to serviceability earth pressures, even if these are greater than the characteristic values of limiting earth pressures used in the Case C ultimate limit state design check. The note in 2.4.2(17) on application of Case B still applies.

Note that this subclause refers only to ultimate limit state design of structures.

C8.6.6(3)P

This paragraph repeats the important sentiment of 2.1(9) and 8.6.1(4)P.

C8.6.6(4)

EC3-5 categorises steel sheet pile sections, showing which are sufficiently robust to allow development of plastic hinges, and which would buckle before this becomes possible. See also C8.6.1(4)P.

The possible use of a model factor γ_{Sd} is mentioned in 2.4.2(15). Neither Eurocode 7 nor the British NAD give values for this factor, and a value of 1.0 has been adopted in almost all trial calculations known to the authors. Possible future developments are noted in D2.2.

C8.6.6(5)P

Error

This Paragraph is a repeat of (3)P and should be deleted.

C8.6.7 Failure by pull-out of anchors

This subclause gives basic requirements. More details of calculation methods can be found in BS 8081. See also C8.8.

C8.7 Serviceability limit state

C8.7.1 General

C8.7.2 Displacements

The intention of this subclause is to require careful consideration of likely displacements without demanding unnecessary calculation or encouraging dubious attempts at calculation which might be misleading. Figure C8.4 shows a flow chart for the decisions required.

The draft of Eurocode 3 Part 5 (prENV 1993-5:1997) contains additional information on assessment of ground movements, particularly for sheet pile walls.

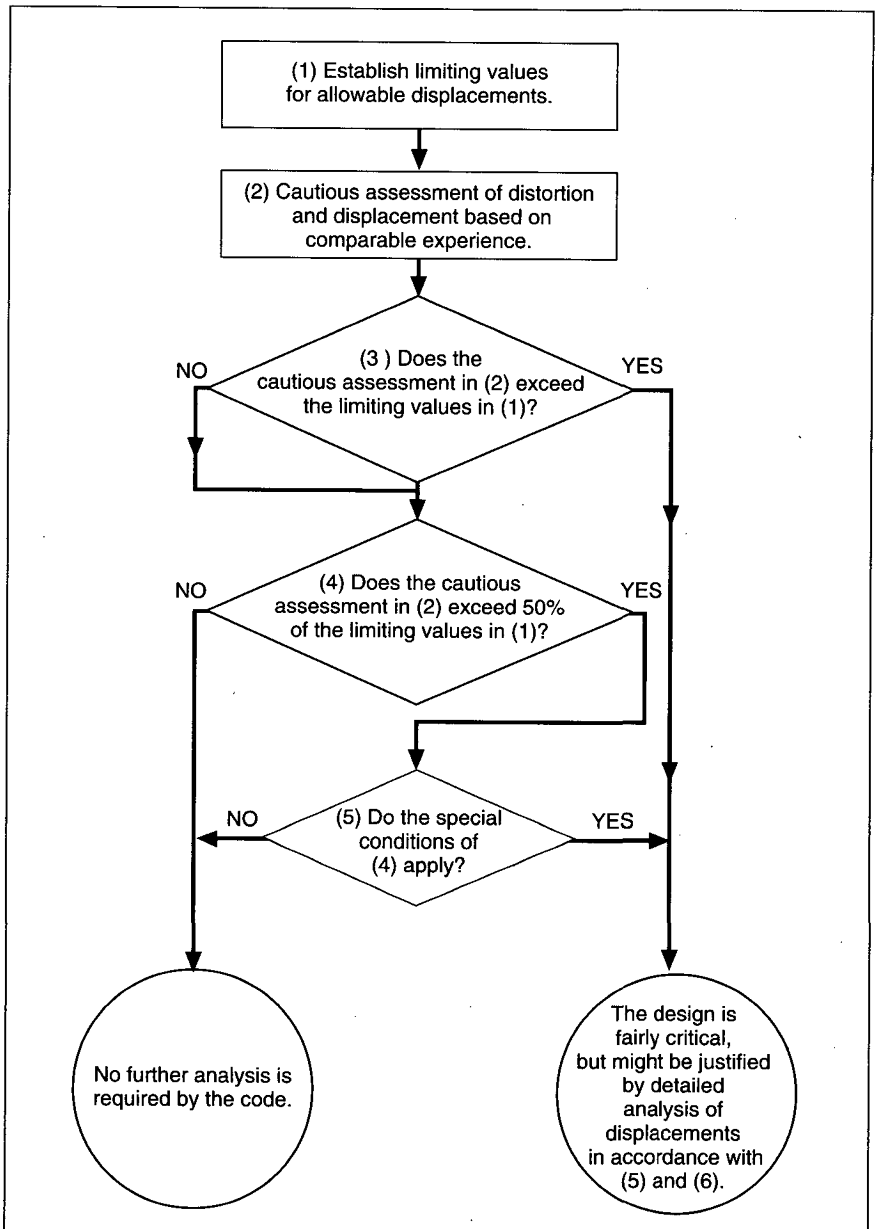


Figure C8.4 Flow chart for serviceability limit state displacements (Subclause 8.7.2)

C8.7.2(1)P

Error

The cross-reference to 2.4.5 should be 2.4.6.

C8.7.3 Vibrations

C8.7.4 Structural serviceability limit states

This is an important subclause, particularly if the comments in C8.6.6 are applied. However, EC7 gives little detail. The user is required to find characteristic values of stiffnesses both for the ground and for structural elements. For structural elements, EC1, 5(2) specifies that the characteristic values of stiffness parameters should be mean values. This is accepted for structural stiffnesses but is over-ridden in EC7 for ground stiffness by 2.4.3(5)P, which is applied to all soil and rock parameters, not only strength. This is discussed further in B4.12.

Because it is difficult to quantify stiffness and to perform soil-structure interaction calculations, $\frac{1}{2}(K_a + K_o)$ is sometimes adopted as the earth pressure coefficient for structural serviceability design of concrete gravity walls. The value of K_o should be derived as in 8.5.2(2).

Table C8.2 Use of partial factors in anchor design and use

Source of action or force	⇒ ⇒ ⇒ ⇒ ⇒ ⇒ ⇒ Magnitude of force ⇒ ⇒ ⇒ ⇒ ⇒ ⇒
From wall analysis [a]	
SLS design force	♦
Case B ULS design force	↓ ♦
Case C ULS design force	↓ ♦
ULS design force (greater of cases B and C)	↓ ♦ ↓
For anchor design and assessment testing	↓ ↓
Required minimum ULS design resistance	↓ ♦ ↓
Required minimum characteristic resistance	↓ I x 1.5 [b] ♦ ↓
Second check on characteristic resistance	• ↓ ↓
Required minimum assessment test result	↓ I ♦ (x 1.1) [c] ↓
Required minimum mean assessment test result	↓ I x 1.3 [c] ♦ ↓
Selected maximum assessment test load (> required mean test result)	↓ I ? [d] ♦ ↓
For anchor use	↓ ↓
Typical preload in acceptance test	I x 1.5 [e] ♦ ↓
Typical lock-off load	I ♦ (x 1.1) [f] ↓
Greater of these two	↓ ♦ ↓
Design of wall structure [g] **– anchors subject to acceptance tests only**	↓ ↓ ↓
Characteristic action for structural design	↓ ♦ ↓
ULS design action – short term	↓ I x 1.0 to1.5 [h] ♦ ↓
Design of wall structure [g] **– anchors subject to acceptance tests**	↓ ↓
Characteristic action for structural design	↓ ♦ ↓
ULS design action – short term	↓ I x 1.0 to1.5 [h] ♦
Design of wall structure [g] **– long term working state**	↓
Characteristic action for structural design	♦ ↓
ULS design action – long term	I x 1.35 [i] ♦
Check for major increase in load with time	I ?→?→?→?→♦

♦ Value of anchor force at indicated stage in the process

[a] SLS force may exceed ULS force in some cases. Case A should be treated as Cases B and C, when relevant

[b] For permanent anchors (EC7, 8.8.5(6)). Use 1.25 for temporary anchors

[c] EC7 Table 8.1, assuming more than 2 assessment tests

[d] Designer's or constructor's judgement, in order to achieve required mean result

[e] See BS 8081, 11.4.3. Use 1.25 for temporary anchors

[f] See BS 8081, 11.4.3

[g] Including bearing plates, walings and connections. prEN 1537:1996 contains information relevant to design of tendons

[h] EC1, Table 9.2 for variable actions. Reduced factors may be considered for short duration loading

[i] EC1, Table 9.2 for permanent actions

C8.8 Anchorages

This clause on anchorages is quite short and for British use should be supplemented by reference to BS 8081. Anchorages are to be designed for actions based on 2.4.2 and the earlier clauses of Section 8.

The approach taken to geotechnical design of ground anchorages is based entirely on load testing. Calculations of appropriate sizes, shapes, grout pressures etc. are not mentioned and are seen merely as tools used in the process of selecting an anchor for testing. For structural design of the anchorages, the user is directed to Eurocode 3 (EC7, 8.8.2(6)P).

Construction of ground anchorages is the topic of prEN 1537 'Ground anchors'. This also gives advice on the preliminary design of anchorages. In the terms of EC7, such preliminary design is the sizing of an anchor to be constructed for test purposes.

C8.8.1 General

The code says that this subclause refers to *any type of anchorage*, including soil nails, dead-man anchors, as well as pre-stressed anchors. However, most of the text is only relevant to pre-stressed anchorages. Dead-man anchors can be designed according to Section 8 as a whole, considering Clause 8.8 only where relevant.

C8.8.2 Anchorage design

Typically, calculations for a retaining wall or other type of structure will have been carried out at ultimate limit state, for Cases B and C, and A if relevant. From each of these, a required minimum capacity will have been calculated for the anchors. It is necessary to derive from these the target values for assessment and acceptance tests, together with an appropriate lock-off force for the anchors and design forces for structural elements such as tendons and walings. EC7 does not state clearly how these values are to be derived.

Table C8.2 illustrates the process involved in anchor design, from the analysis of the retaining wall to design of structural members. Approximate relative values of the forces and actions are indicated by a ♦ symbol.

It is recommended in this commentary that the minimum required anchor capacity, derived from wall or other calculations, should be treated as the **minimum ULS design resistance** of the anchors. This design resistance must equal the **characteristic resistance** divided by the factor γ_m (Equation 8.4). This leads to a minimum value for the characteristic resistance.

Calculations may also be performed for the serviceability limit state, yielding another minimum required resistance; the characteristic resistance of the anchors should also be not less than this value.

This approach to anchorage design differs from pile design in Section 7, in which piles are designed for Case B (factored loads, unfactored soil or unfactored load tests – $\gamma_m = 1$) and for Case C (unfactored (permanent) loads, factored soil or factored load tests). However, it is proposed here that anchors are designed for Case B and/or C, which effectively give factored anchor loads, **together with** factored load tests. The anchor loads derived from Case B or C are treated as required **design** anchor capacities.

It is considered that this approach generally leads to anchor designs similar to conventional practice, though possibly more conservative in some cases. An alternative might be to consider the required capacities derived from ULS wall calculations to be the required **characteristic** resistances of the anchors. This would be similar to the approach taken to pile design, but would be unconservative compared with conventional anchor design.

Assessment tests on anchors will normally prove the capacity of the grout/ground interface but will not fail the tendon/grout interface or the tendons themselves. Hence separate partial factors are quoted for the tendon/grout interface and the tendons themselves.

A worked example is presented in E16.

C8.8.3 Construction considerations

This subclause gives only a very bare outline of the specification needed for corrosion protection. Both BS 8081 and prEN 1537, Section 6 give guidance on corrosion protection for temporary and permanent anchors.

C8.8.4 Anchorage testing

C8.8.5 Assessment tests

C8.8.5(4)P

In prEN 1537:1996 'Ground Anchors', the creep limit is defined as *the maximum creep displacement rate permitted at a specific load level.* The creep displacement rate (k_s) is defined as:

$k_s = (s_2 - s_1) / \log_{10} (t_2 / t_1)$ (note that k_s has units of displacement)

where s_i = displacement at time t_i.

For ULS the creep limit load is defined as the load at which k_s is equal to 2 mm. prEN 1537 states that the limit of 2 mm will be used as one of the ULS failure criteria in an assessment (on-site suitability) test. The creep limit rate should not exceed 1 mm at proof load where investigation tests have been carried out (or 0.8 mm where no investigation tests have been carried out).

The measurement of the creep displacement rate may be carried out using a maintained load test, as defined in prEN 1537 which recommends time intervals for measurement of anchor head displacement of 1, 2, 3, 5, 10, 15, 20, 30, 45, 60 minutes. For fine grained soils, longer time intervals may be needed.

C8.8.6 Acceptance tests

C8.8.7 Supervision of construction and monitoring

Comment

A subclause should be added requiring that records be kept of ground conditions encountered.

C9 EMBANKMENTS AND SLOPES

This section covers two main topics: site stability (including slopes) and embankments. It might have been better to place a section on site stability at an earlier point in the code, since reference is made to this section from Sections 6, 7 and 8. In principle, embankments and slopes should be designed for Cases A, B and C, as with other items. However, Case B is rarely found to be critical, and 9.5.1(5)P states that *Case A ... may generally be omitted.* The section is therefore directed almost entirely to Case C. The partial factor on $\tan\phi'$ in Table 2.1 could be the only factor of safety against failure in many of the situations relevant to this section. The value of 1.25 is somewhat less than conventionally used for major slope stability checks. For example, BS 6031 requires an equivalent overall factor of safety of between 1.3 and 1.4. However, noting that higher partial factors are applied to c' and c_u it is considered that the factors in Table 2.1 are adequate provided that the characteristic soil strengths are 'cautious estimates' (2.4.3(5)P).

C9.1 Scope

C9.1(1)P

It is strange that dykes and dams are excluded from Section 9, though they are not excluded in 1.1. It is recommended that dykes and dams should not be designed to ENV 1997-1:1995.

C9.2 Limit states

C9.2(1)P

Note that deformations in the ground may cause either 'structural damage' (ultimate limit states in the structure) or 'loss of serviceability' (serviceability limit states). In principle this is correct, but it is difficult to make a clear distinction in practice. Hence Paragraphs 9.5.2(1)P and 9.6(1)P are almost identical.

C9.3 Actions and design situations

C9.3(2)P

The list should also include the effects of vegetation, both in strengthening slopes and in causing desiccation which may lead to water-filled cracks.

C9.3(3)P

It is stated that the failure of drains, filters and seals shall be considered. This does not necessarily mean that the design must accommodate failure, provided that failure can be shown to be avoidable or sufficiently unlikely. For ultimate limit states, this means that it must be made extremely unlikely. The principles developed under 8.3.2.2 and 8.4(5)P and (6) also apply here.

C9.4 Design and construction considerations

C9.5 Ultimate limit state design

C9.5.1 Loss of overall stability

C9.5.1(3)P

As noted at the start of this section, it will usually be obvious by inspection that Case C is critical, and calculations for Cases A and B should not be necessary. (See also 9.5.1(5)P.) This conclusion assumes that the weight of the ground is treated as a 'single source', as discussed in 2.4.2(17). The statement in 2.4.2 appears to be restricted to retaining walls, but could also be applied here. This is clarified by 9.5.1(6).

C9.5.1(4)
The implication of this application rule is that the following methods of slope stability are acceptable:

a Bishop (1955) methods with either horizontal or inclined interslice forces;
b Janbu (1957) with inclined interslice forces;
c Morgernstern and Price (1965);
d the method of Spencer (1967).

The following methods are not acceptable:

e Fellenius (1927), also known as the Swedish circle method;
f Janbu (1957) with horizontal interslice forces.

C9.5.2 Deformations

C9.5.2(1)P
This paragraph, still under the heading ultimate limit state design, refers to cases where deformation may cause ultimate limit states in supported structures. Compare 9.6(1)P for serviceability limit states.

C9.5.2(3)
The main method available for limiting the deformation of a slope is by limiting the mobilised shear strength. Where slopes support structures which are very sensitive to movement, it might be necessary to use partial factors applied to shear strength which are slightly higher than those given in Table 2.1. See the note at the start of this section.

C9.5.3 Superficial erosion, internal erosion and hydraulic uplift

C9.5.4 Rock slides

C9.5.5 Rock falls

C9.5.6 Creep

C9.6 Serviceability limit state design

C9.7 Monitoring

ANNEX A
Checklist for construction supervision and performance monitoring

Annex A supplements Section 4.

ANNEX B
A sample analytical method for bearing resistance calculation

The calculation method for bearing capacity given in Annex B is recommended for general use. It is a development of the original work of Brinch Hansen (1970), taking advantage of more recent work, particularly that of Smoltczyk and his co-workers in Stuttgart. This has led to a revision of the formula for N_γ, providing a less conservative value. A comparison of various formulae for N_γ derived by different researchers over the past decades is presented in Appendix 1. Formulae for N_c and N_q are well established.

The values of bearing capacity factors implied by the equations given in Annex B are shown graphically in Figure C6.4.

Formulae for depth correction factors, d, have been omitted from Annex B because they are regarded as unreliable. Although they have the potential advantage of providing continuity between the design of a footing and that of a pile, their practical relevance is minor and they can be neglected, without undue conservatism, in the design of spread foundations.

Some examples of the use of Annex B are presented in E2 to E4.

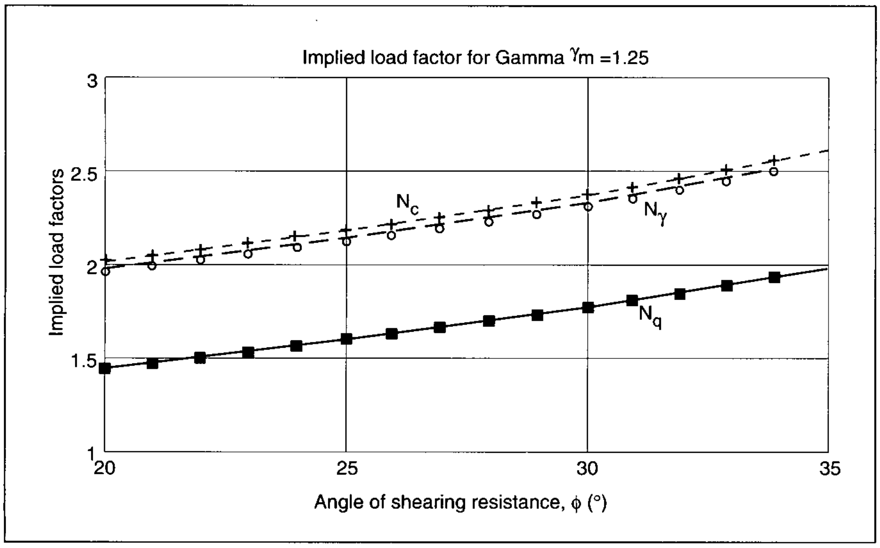


Figure CB.1 Overall factors of safety on bearing resistance implied by EC7 partial factors ($\gamma_\phi = 1.25$, $\gamma_{c'} = 1.6$)

Traditional design uses an overall factor of safety on bearing capacity in the range 2 to 3 (BS 8004, 2.2.2.3.3.4 notes this range, but only for 'cohesive' soils). Figure CB.1 shows the overall factors on bearing capacity in **drained** soils derived from the equations in Annex B, together with the boxed values of partial factors (EC7, Table 2.1), also adopted in the British NAD (Table 1). The boxed values imply overall factors in the range 2 to 2.5 for foundations dependent mainly on N_c (ie 'cohesion') or N_γ. Where bearing capacity is strongly dependent on the benefit of overburden pressure above the foundation depth, N_q becomes important. In this case, the overall factor of safety could fall below 2.

For **undrained** soils, EC7 and the British NAD lead to an overall factor of safety of only 1.4 (ie the factor on c_u). However, this is only relevant to very short term loading, and should not be confused with BS 8004's requirement, which is intended for long term loading of clay soils. To comply with EC7, long term situations must be checked in the drained state, for both ULS and SLS requirements.

ANNEX C
A sample semi-empirical method for bearing resistance evaluation

Annex C presents the basic principles of evaluation of bearing resistance from Ménard pressuremeter tests. Insufficient detail is given to make calculations possible, though more will be included in EC7 Part 3 (ENV 1997-3), when it is published. For more information, reference should be made to Ménard (1975), Baguelin et al (1978) and the brief discussion of Mair and Wood (1987, section 6.7).

The semi-empirical use of the pressuremeter, popular in France, is based on results from Ménard pressuremeters simply inserted into bored holes. This approach has not found favour in the UK. Instead, attempts have been made to derive fundamental parameters from pressuremeter results, and so self-boring pressuremeters have been favoured (Mair and Wood (1987)). These parameter values can then be used, in combination with other information about the nature of the soils, in standard bearing capacity or settlement calculations etc.

ANNEX D
Sample methods for settlement evaluation

D.1 Stress–strain method

The method described here is a standard approach to settlement calculation. Stresses are derived from elasticity theory (such as the Boussinesq equations), and strains are then calculated for various layers in the ground according to the Young's modulus at each point. Vertical strains are integrated to find displacement. Computer programs such as VDISP in the *Oasys* GEO suite perform this calculation.

The stress distribution used for this calculation is an approximation since it is derived taking no account of the distribution of stiffness in the ground. However, this is generally considered to be adequate provided stiffness is constant or increases with depth (Gibson (1974)).

An example of this method is presented in E5.

D.2 Adjusted elasticity method

The method described here is based on elasticity theory and would be accurate if the Young's modulus E_m were constant with depth. However, this method could be quite unreliable if Young's modulus varies significantly with depth, especially if used to predict the settlement of a large variety of sizes and shapes of foundations. In general, the authors of this commentary consider the stress-strain method to be preferable.

An example of this method is presented in E5.

D.3 Settlements without drainage

D.4 Settlements caused by consolidation

The prediction of settlements due to undrained behaviour and subsequent consolidation is helpfully discussed by Burland, Broms and De Mello (1978).

D.5 Time–settlement behaviour

ANNEX E
A sample method for deriving presumed bearing resistance for spread foundations on rock

Annex E supplements EC7, 6.7.

ANNEX F
A sample calculation model for the tensile resistance of individual or grouped piles

An example of the application of this annex is presented in E11.

ANNEX G
Sample procedures to determine limit values of earth pressure

This annex gives two methods of deriving coefficients of active and passive pressure: charts and formulae. There is some confusion in the annex because the charts refer to K_a and K_p, whereas the formulae use K_γ and K_q.

The charts are taken directly from CIRIA Report 104, and are based on the work of Caquot, Kerisel and Absi (1973). Rather clearer charts, to a larger scale and based on more recent work of Kerisel and Absi (1990) may be found in BS 8002. Both the charts and formulae relate to the components of forces normal to the wall surface (ie horizontal components if the wall surface is vertical). This contrasts with the published work of Kerisel et al whose factors relate to the resultant forces inclined at angle δ to the wall surface.

The terms used in the formulae will be unfamiliar to British users in two respects.

a The coefficients applied to the self weight of the soil K_γ and the effect of surcharges K_q are treated separately. However, these two values are identical for vertical walls.

b The formulae do not directly distinguish active and passive coefficients. Rather, the equations are formulated so that active or passive values may be derived by changing the signs of some of the parameters.

The numerical procedure provided in the annex is based on plasticity solutions. Results obtained from the formulae are shown for horizontal ground and vertical walls in Figures CG.1 and CG.2, for active and passive coefficients, respectively. Figures CG.3 and CG.4 provide a comparison between the EC7 and the Kerisel and Absi results. Close agreement is found except for large angles of shearing resistance using high values of δ / ϕ'.

Error

In Equation G.14, the final symbol should be θ, not ϕ:

$K_\gamma = K_n \cos\beta \cos(\beta - \theta)$

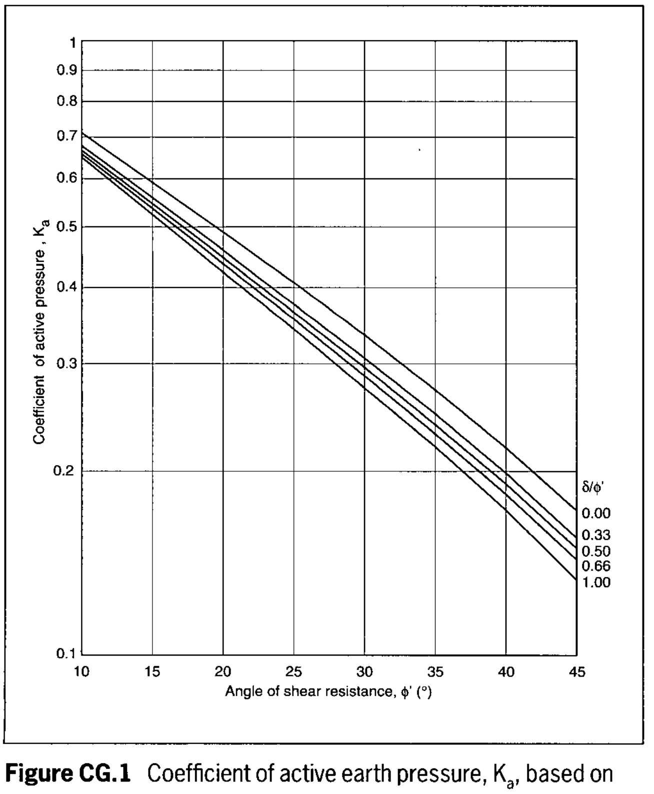


Figure CG.1 Coefficient of active earth pressure, K_a, based on EC7 Equation G12

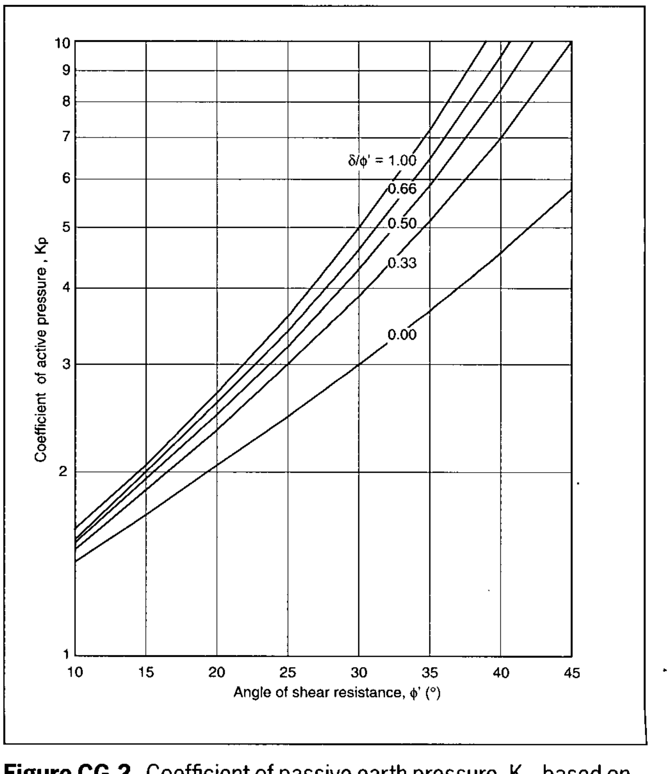


Figure CG.2 Coefficient of passive earth pressure, K_p, based on EC7 Equation G12

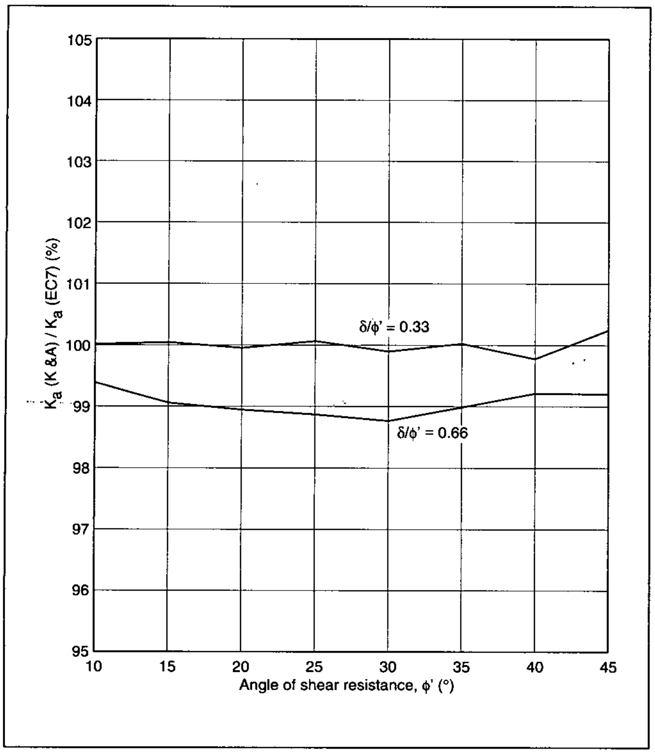


Figure CG.3 Ratio between coefficients of active earth pressure from Kerisel and Absi and from Equation G12

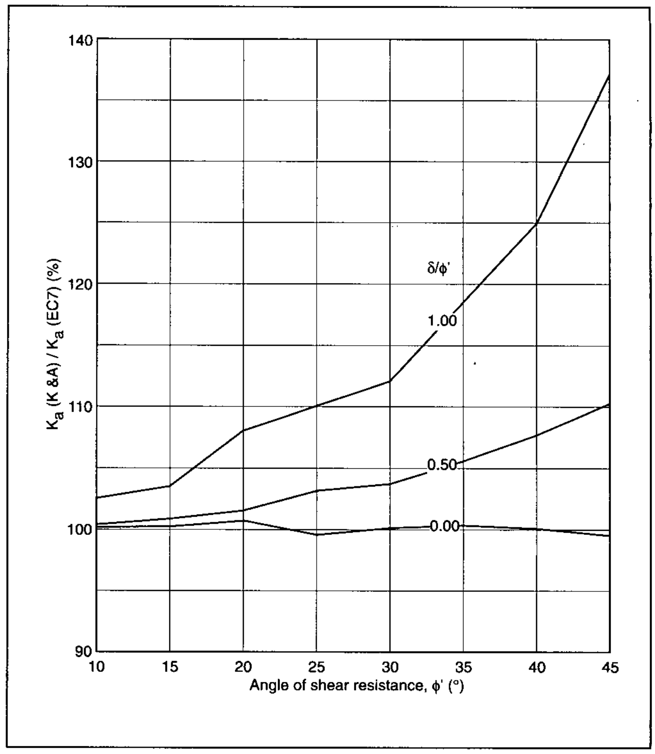


Figure CG.4 Ratio between coefficients of passive earth pressure from Kerisel and Absi and from Equation G12

APPENDIX 1
Bearing capacity factor N_γ for shallow foundations

The stability of shallow foundations, including both mudmats and bases of gravity structures, is conventionally checked using bearing capacity factors N_q, N_c and N_γ. The values of N_q and N_c are established by theory and there is no dispute about these. However, N_γ is established empirically and its value has been the subject of much debate over many years. It is particularly critical to the design of large shallow foundations subject to a component of horizontal loading.

Brinch Hansen (1970) published the following formula for N_γ which has been widely used:

$$N_\gamma = 1.5\,(N_q - 1)\tan\phi' \qquad \text{(C1.1)}$$

Caquot and Kerisel (1953) published the more optimistic formula used by the American Petroleum Institute documents (API, 1993):

$$N_\gamma = 2.0\,(N_q + 1)\tan\phi' \qquad \text{(C1.2)}$$

Recent research in Germany and elsewhere has led to the adoption of the following formula in DIN 4017 and in EC7:

$$N_\gamma = 2.0\,(N_q - 1)\tan\phi' \qquad \text{(C1.3)}$$

Results obtained from these formulae are shown in Figure Cappl.1.

In the range of practical interest, values of N_γ calculated from Eq C1.3 (EC7) exceed those of C1.1 (Brinch Hansen) by 33% but are 10 to 15% less than those of C1.2. For comparison, it is noted that for offshore structures DnV (1992) uses Eq C1.1 generally, but with C1.2 for localised stresses, whilst API-RP 2A-WSD (API (1993)) uses only Eq C1.2. The use of C1.3 in EC7 lies between these two.

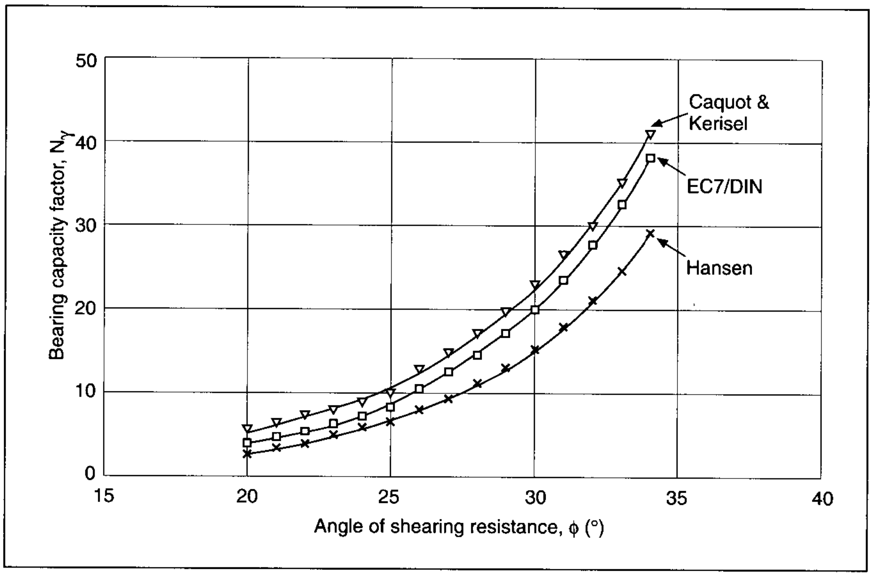


Figure Cappl.1 Values of bearing capacity factor, N_γ, calculated using the formulae of Brinch Hansen (1970), Caquot and Kerisel (1953) and EC7/DIN

APPENDIX 2
Errors in EC7

Location	Error
6.5.3(4)	(6.5) should be (6.2)
6.5.4(1)	The combination of application rule and principle in (2) and (3)P does not make sense
6.6.1(8)	'Shall' should be 'should' in the first line of the final paragraph
6.8(2)	The cross-reference to 2.1(8)P is pointless
7.7.2.3(3)	This paragraph should refer to Annex F, not Annex G. It is out of place; Annex F is about structural design of piles. The paragraph therefore should be moved to 7.9.
8.4(1)P	The cross reference to 2.1 is thought to be incorrect. It should probably be 2.4.1.
8.5.2(2)	In the second paragraph, 'horizon' should be 'horizontal'
8.6.6(5)P	This Paragraph is a repeat of (3)P and should be deleted
8.7.2(1)P	The reference to 2.4.5 should be 2.4.6
8.8.5(4)	The line beneath the table should read 'The characteristic anchorage resistance, R_{ak}, ...'
9.4(2)	Line 11: 'settlements' should be 'settlement' Line 14: 'press ure' should be 'pressure'
9.4(4)	'scrubs' should be 'shrubs'
Annex G	In Equation G.14, the final symbol should be θ, not ϕ: $K_\gamma = K_n \cos\beta \cos(\beta - \theta)$

APPENDIX 3
Design to BS 8002 and Eurocode 7

Initial assessment of soil properties
BS 8002
'Representative values'.

EC7
'Characteristic values'.

Comment
In practice, Representative and Characteristic values will probably be indistinguishable. They are also similar to the 'moderately conservative' values of CIRIA Report 104. See B4.

Design values of soil strength
BS 8002
Strength (c' and $\tan\phi'$ or c_u) divided by Mobilisation factor M.
$M = 1.2$ for c' and $\tan\phi'$, and 1.5 for c_u.

EC7
Strength divided by partial safety factor γ_m (Table 2.1).

Factor on:		**tan.ϕ'**	**c'**	**c_u**	**Permanent loads (γ_G)**	**Variable loads (γ_Q)**
BS 8002	M =	1.2	1.2	1.5	1.0	1.0
EC7 Case B	γ_m =	[1.0]	[1.0]	[1.0]	[1.35] unfavourable	[1.0] favourable
					[1.5] unfavourable	[0.0] favourable
EC7 Case C	γ_m =	[1.25]	[1.6]	[1.4]	[1.0]	[1.3] unfavourable
						[0.0] favourable

Comment
Characteristic or representative values are divided by factors γ_m or M to obtain **design** values. These are the values entered into equilibrium and other calculations. There are no overall factors of safety, but load factors are applied to external loads in some cases.

Designs to EC7 must comply with **both** Cases B and C. In selecting the γ_G factor, 'the ground' should be treated as a 'single source', so all earth pressures and water pressures are multiplied by the same factor (generally the 'unfavourable' value). In cases where this seems unreasonable, the alternative of applying the same factor to bending moments and shear forces is allowed.

EC7 values are shown in brackets because they may be varied nationally.

BS 8002 has a further requirement that the design strength shall not be greater than that given by the representative **critical state** value of ϕ' (ϕ'_{crit}). In practice, this requirement will rarely govern the value of ϕ'_d, but it may limit the strength at low stress if $c'_{rep} > 0$.

In use, the M and γ_m factors are similar, but their justifications are, in principle, different. Eurocodes see γ values as allowances for probabilistic uncertainties, ensuring that ultimate limit states will not occur. BS 8002 uses M to ensure that in the working state materials are not overstressed, so deformations should be tolerable, in common cases at least; that is, serviceability limit states will not occur.

In reality, the factors probably fulfil both roles. Their main justification is that they have been shown by experience, or by comparison with previous designs, to produce suitable structures. That is, structures which rarely exhibit either ultimate or serviceability limit states.

Design angle of wall friction δ

BS 8002

$\delta_{des} = \tan^{-1}(0.75 \tan\phi'_{des})$ $^2/_3\phi_{rep}$, or ϕ'_{crit}, whichever is smaller. Restricted to 20° for relatively smooth walls. Vertical equilibrium is also to be considered.

EC7

δ_{des} may be up to ϕ'_{crit}, depending on the roughness of the wall, relative vertical movements and the need to preserve vertical equilibrium. For steel or pre-cast concrete walls, δ_{des} should not exceed $^2/_3\,\phi'_{crit}$. EC7, 8.5.1(4), 6.5.3(8).

Comment

See C6.5.3(8) and C8.5.1(4).

Wall adhesion

BS 8002

$c_{w,des} = {}^3/_4\, c_{u,des} = {}^1/_2\, c_{u,rep}$.
Effective wall adhesion, c'_w is not mentioned.

EC7

Wall adhesion is given the symbol a. Both a_u and a′ are mentioned, but guidance is not specific.

Comment

The BS 8002 rules could be used with EC7. Use $c'_w = a' = 0$ at all times.

Water pressures

BS 8002

'... the most onerous that is considered reasonably possible.' (3.2.2.3)

EC7

ULS: ' the most unfavourable values which could occur in extreme circumstances.'

SLS: ' the most unfavourable values which could occur in normal circumstances.'

(EC7, 2.4.2(10); see C8.3.2.2)

Surcharges

BS 8002

A **minimum design surcharge** of 10 kPa is required. When larger surcharges are actually expected, they should be used in the calculations with load factors specified by other codes for structural design.

EC7

There is no minimum surcharge. Variable loads are multiplied by factors of 1.3 to 1.5 for ultimate limit state design.

Comment

The BS 8002 requirement of a minimum surcharge of 10 kPa is severe, especially for small walls.

In both cases, surcharges should be applied at the worst locations that could occur.

Unplanned overdig

For walls which rely on passive soil for support, both codes require that the surface of the passive soil be assumed lower than it is expected to be.

BS 8002

Allow for over-dig of 10% of the height of cantilever walls, or 10% of the height below the lowest prop for propped walls, with a **minimum** of 0.5 m.

EC7
For ultimate limit state design, allow for over-dig of 10% of the height of cantilever walls, or 10% of the height below the lowest prop for propped walls, with a **maximum** of 0.5 m.

Comment
See C8.3.2.1(2)

Bearing resistance
BS 8002
Refers to BS 8004 for allowable bearing pressures. This is severe when used in conjunction with factored strength. Furthermore, BS 8004 does not treat eccentric and inclined loads adequately; these are major features of wall bases.

EC7
Calculations using bearing capacity factors N_γ, N_c and N_q, with the same set of design soil strengths.

Equilibrium calculation
Check equilibrium with no requirement for an 'overall' factor of safety. It is often helpful to calculate the value of a 'spare' overall factor of safety in some form, the only code requirement being that it is greater than 1. If it is too large, the design may be uneconomic.

Structural strength
BS 8002
Derive bending moments and internal forces directly from the equilibrium calculations and use these as both ultimate **and serviceability** values. In principle, check ultimate strength of the structure for earth pressures derived in the same way, but with the possibility of greater (factored) surcharges if these are required by other codes.

Also check the possibility of larger earth pressures in the working state of the structure (due to compaction, pre-consolidation, etc), and, if relevant, treat these as **serviceability** values.

EC7
Derive bending moments and internal forces directly from the equilibrium calculations and use these as **ultimate** limit state values.

Serviceability is to be checked using unfactored characteristic values of soil properties and loads. (In a formal sense, a factor of 1.0 is applied.) Overdig is not included in this calculation.

Also check the possibility of larger earth pressures in the working state of the structure (due to compaction, pre-consolidation etc), and, if relevant, treat these as **serviceability** values.

Comment
For simple design to BS 8110, the BS 8002 requirement will mean that further factors of about 1.4 must be applied to the moments and shears. This is more severe than traditional design or EC7.

Note that compaction, pre-consolidation pressures etc are not included in the equilibrium calculations, but only in the structural strength calculations. It is assumed that if these high stresses push the wall towards instability they will be relieved with relatively little movement.

Movement
Both BS 8002 and EC7 state that **calculations** of displacement will not normally be necessary for typical walls, built on good ground with the required factors of safety or mobilisation. However, a rough assessment of movements should always be made.

Part D The way ahead

CONTENTS

D1 THE USE OF EUROCODE 7 OUTSIDE THE UK

D1.1 Introduction

Considerable interest in Eurocode 7 is being expressed in many parts of the world; for example, 26 countries were represented at a Seminar held by the Institution of Structural Engineers in 1996 (Orr (1996)). Attitudes to the Eurocode, at least in the Member States of the EU, may be differentiated roughly by geographic location. The Scandinavian countries have, by and large, more readily accepted the use of partial factor design, reflecting perhaps the influences of Brinch Hansen in the region. Indeed, in Denmark, their national codes, embodying partial factor design, are well advanced. Other 'northern' European countries such as France, Germany and the UK, having in place comprehensive, well-tried sets of geotechnical codes and standards based heavily on empiricism, have been less ready to embrace partial factors. The southern European countries generally have less comprehensive codes and rely to a greater extent on legislation and government to implement safety regulations.

Elsewhere in the World, particular interest in EC7 has been shown in Japan, South Africa and Israel; some views on their opinions are given later.

In parts of Europe other than the EU member states, there is involvement in EC7 development (in Norway, the Czech Republic and Slovakia). These countries are also developing their own NADs to permit implementation of the ENV.

Any UK company or designer wishing to work in any of these and the Member State countries must understand the requirements of the local NAD.

D1.2 Other European NADs

As noted in A1.5, the future status of NADs is uncertain. The present situation, which relates to the 1995 edition of EC7, is described here.

The purpose and use of National Application Documents (NADs) was introduced in A1.5 and A1.6, and the British NAD was discussed in A2.4. For the members of the EU, each State prepares its own National Application Document (NAD) for each Eurocode. These will generally be written in the national language and, together with the national translation of the Eurocode, they will govern design of structures to be constructed in that country, irrespective of the nationality or location of the designer. In most cases, English translations of NADs are being prepared. Some of the southern European countries have not yet produced NADs because these documents have to be written by government departments and must acquire legal status; this takes time.

At the time of writing (early 1998), completed NADs (in English) were available only from the UK, Ireland, Germany and Finland; these are discussed in more detail below. Some countries have used their NADs to introduce significant modifications and additions to EC7. The extent to which this will be allowed in the final publication of the EN is uncertain, but it would seem that 'National Annexes' may be permitted in which normative material that does not conflict with EC7 may appear, together with other informative material.

Progress with NADs for the EU Member States and other European countries that are participating as observers on CEN/TC250/SC7 is indicated in Table D3.1, from which a few general observations can be made:

- **a** NADs are likely to contain substantial amounts of material introducing 'local' requirements, usually by reference to national codes and standards;
- **b** several countries report difficulties where government legislation will be required for the adoption of the Eurocodes and NADs;
- **c** there is some difficulty in applying Cases B and C, especially to the design of retaining structures;
- **d** while usage of EC7 is very low, as is the case in the UK, those who have used it are generally favourably disposed towards it.

D1.2.1 The German NAD

DIN (1996) have published the proceedings of a seminar in which applications of Eurocode 7 were demonstrated by extensive worked examples. This reflects the state of German thinking about EC7 in 1996.

Features of this NAD are summarised in Table D3.2. As can be seen, there is extensive reference to DIN documents. (Note: the designation 'V' refers to pre-standards, for experimental application.) These documents have been included as Appendices in the NAD and embrace the concept of partial safety factors. The NAD introduces the role of a 'Geotechnical Expert' for Geotechnical Categories 2 and 3. Clause 8.2 of DIN V 1054-100 states:

Before construction the geotechnical expert shall be responsible for planning and directing the necessary investigations, both in the field and in the laboratory. He shall give decisive parameters and assess the results obtained as related to the design and execution of the proposed works. He shall also assess the effects such results may have on other work involved, and notify the appropriate persons (designer, other experts etc).

During construction the geotechnical expert has to compare the real ground conditions with those expected, to assess the correspondence of design and execution from the geotechnical point of view and to carry out the measurements and evaluations which are necessary for this purpose.

This text does not define the nature of the geotechnical expert but rather what he does. In EC7 there is an overall requirement (EC7, 1.4(1)P) that personnel with 'appropriate qualifications and experience' shall perform the tasks. Any further attempt to define the qualifications of personnel should perhaps be a matter for legislation and contract: see also C1.4.

The German NAD significantly modifies the application of Cases A, B and C and corresponding γ factors by reference to DIN V 1054-100; Tables D3.3 and D3.4 (taken from the DIN) show the values of partial safety factors for actions and for soil resistances respectively that replace the boxed values of EC7, Table 2.1. Comparing Table D3.3 with EC7, Table 2.1, it can be seen that Load Case 1 factors are those in Table 2.1, but with additions such as $\gamma = 1.2$ for permanent earth pressures at rest. The NAD does not explain the significance of Load Case 2. An explanation of the thinking behind the adoption of the different German values can be found in Gudehus and Weißenbach (1996).

Comparing Table D3.4 with Table 2.1 shows that factors on soil strength, for Case 1C with Load Case C, are identical. However, for Case 1B it seems that factors > 1.0 on soil resistance will lead to a more conservative design than for Table 2.1.

D1.2.2 The Finnish NAD

Features of this NAD are summarised in Table D3.5. The following points are noteworthy:

a a desire to make the use of Geotechnical Categories (see C2) normative;
b changes to γ values;
c references to Finnish codes and standards, particularly for site investigation and retaining structures;
d emphasis on frost susceptibility.

D1.2.3 The Irish NAD

The Irish NAD is intended to enable EC7 application in an 'experimental way' and makes very few amendments to EC7; Table D3.6 shows the main features. The user is alerted to uncertainty about the legal and contractual significance of EC7 clauses covering the Geotechnical Design Report (2.8), the Ground Investigation Report (3.4) and Supervision, Monitoring and Maintenance (4.1).

D1.3 Other overseas usage of Eurocode 7

D1.3.1 South Africa

In 1995, the South African Institution of Civil Engineers decided to adopt EC7 as a standard for limit state design in geotechnics; it would be used in parallel with existing design methods for a trial period of three years. Thus far, there has been little use made of EC7. One reason for this is that the South African Bureau of Standards adopted for its structural codes a partial load factor for dead weight of 1.2 compared with the EC7 boxed value of 1.35. Another important reason is that many South African geotechnical engineers rely upon partial saturation when designing structures such as temporary excavations; this gives them added difficulties when considering characteristic values of strength. It is not the material property variability that they must consider but rather the probability of the occurrence of an external event affecting the saturation of the soil. It would appear that South African engineers may have no difficulties in using EC7 for the design of spread and piled foundations but baulk at its use for geotechnical structures such as slopes and embankments, for which they would prefer to take account more directly of the variability of the input parameters.

D1.3.2 Israel

Israel has been attracted to the concepts of EC7, including the separation of Principles from Application Rules, and has decided to use EC7 as the basis for a new Israeli code. There will be no NAD, but substantial textual changes to reflect geological and climatic differences (eg desiccated, expansive soils).

D1.3.3 Japan

There is considerable interest in EC7 in Japan. The following remarks are a synopsis of a comprehensive set of essays prepared for the authors by some leading Japanese designers, code writers and academics.

Partial factor design

In Japan, the major design codes are traditionally developed by different sectors of the government. Thus there are different design codes for roads, ports and harbours, buildings and energy facilities etc. In the not-too-distant future they hope to be able to produce and promote internationally a unified Japanese model code. Understandably, design thinking is dominated by seismicity and displacement based considerations are not yet amenable to partial factor approaches. Nevertheless, there is consistent support for partial factor design because it is considered:

a to be more rational. As technology develops in the future, it will be possible to reflect improvements in design methods by altering the partial factors without losing the 'safety balance' of the whole structure, as could happen with the global factor approach;

b to provide more flexibility to account for new changes in construction methods, developments in site exploration and advances in analytical tools. One of the impediments to developing new construction methods is the inflexibility of the global safety factor approach.

In 1996 a new design standard for foundations of railway structures was published. This was based on limit state design, but applying partial factors to loads and ground resistances, rather than to ground properties. Partial geotechnical factors are applied to parts of the ground resistance: for example, there are different factors for the shaft and toe resistance of piles. It is considered that this approach gives greater freedom to change partial resistance factors to reflect changes in safety levels arising from advances in pile design and construction methods.

The Japanese Specification for Highway Bridges (SHB) is based on Allowable Stress Design (ASD). It is considered that one of the greatest disadvantages of ASD is that all of the uncertainties are taken into account in one global safety factor and that this safety factor cannot be varied depending on the uncertainties. Consequently, the next SHB will introduce partial safety factors.

Characteristic value

The definition of 'characteristic value' in EC7 is supported by some and it is recognised that it gives designers 'total' freedom (of choice), or 'biggest uncertainty'. There is also support for the characteristic value to be the 'average' (possibly a 'statistical average') value.

Others feel that the partial factor and the characteristic value should be considered as a pair and that EC7 should clearly define the derivation of the characteristic values applicable to the partial factors shown in Table 2.1.

Cases B and C

There is no consensus on Cases B and C. Some think the definitions are ambiguous; others favour only one case (B), with the Case C strength factor adopted for determining the resistance in the ultimate limit state design check and with $\gamma_F = 1.0$ for soil self weight. If Cases B and C remain unaltered, *conservative designs together with troublesome calculations may be unavoidable. An attempt needs to be made to find a better alternative.*

There is general agreement that $\gamma_F = 1.35$ is an 'extraordinarily large value for dead load'.

D1.3.4 Hong Kong

The Hong Kong 'Guide to retaining wall design' (GEO (1993)) uses the concepts of EC7 and quotes its text extensively. One significant difference from the boxed values of EC7 is the choice of 1.2 for the factor on both $\tan\phi'$ and c'.

D2 FUTURE DEVELOPMENT OF EUROCODE 7

D2.1 The Eurocode system

In early 1998, mandates are being issued for a further round of development of Eurocodes. It seems likely that Eurocode 1 will be divided into two documents: the first on Basis of Design, and the second on loading. The Basis of Design document will probably have separate sections for different types of structures – buildings, bridges, towers etc – possibly with separate versions of EC1 Table 9.2, giving differing values for load factors for each type of structure.

D2.2 Eurocode 7 Part 1

During 1997 and 1998, a Working Group set up by CEN/TC250/SC7 has been meeting to try to resolve some of the known problems with the ENV, and to review comments received. In early 1998, a mandate is about to be issued to CEN by the European Community which will require the formation of a Project Team, contracted to prepare EC7-1 for EN status. Their work on the text is to be completed by mid-2000, and the EN is to be published as soon as possible thereafter, subject to time required for translation into three official EU languages. The members of the Project Team will probably be C Bauduin (Belgium), G Bosco (Italy), R Driscoll (UK) and U Schmoltczyk (Germany, convenor).

The significance of publication as a Euronorm was discussed in A2.7.

Much discussion in the Working Group has centred around the scheme of the partial factors to be adopted, together with the use of Cases A, B and C. Various schemes are being proposed to reduce Cases B and C to a single case. There is a strong lobby for the use of 'model factors', placed at various locations within the calculations. These would allow a degree of differentiation between the safety margins associated with different types of structural elements, but it is unclear whether they will be associated with specific calculation methods or formulae. The outcome of these discussions is as yet uncertain; any changes will need to be consistent with Basis of Design.

At early 1998, no specific changes in the definition of characteristic values seem likely, though SC7 might try to issue a supporting document including some examples. In some countries there is considerable interest in attempts to produce viable statistical methods of deriving characteristic values, requiring use of fairly advanced statistical processes, as discussed in B4.9. As noted in A2.5, Parts 2 and 3 of EC7 lead the user to 'derived values' of parameters, so it is likely that this concept will require definition and adoption in Part 1. Its definition is not yet clear, however.

It is likely that new sections on 'Site stability' and 'Ground anchorages' will be introduced, removing some of the material from the current Sections 9 and 8, respectively.

Material which relates to design calculations and which has been introduced into CEN documents produced since 1994 will probably be moved into EC7-1. This includes methods for calculation of settlement in EC7 Parts 2 and 3, based, in most cases, on the results of specific types of soil test, such as pressuremeter or penetration tests. Material may also be moved from documents produced by TC288, notably in relation to design of ground anchors from prEN 1537.

D2.3 Eurocode 7 Parts 2 and 3

The European Community decided in Autumn 1997 that mandates for the completion to EN status of Parts 2 and 3 will be issued, probably in about 1999.

The nature and status of Parts 2 and 3 was discussed in A2.5 and A2.7. It is possible that the two parts will be consolidated into a single document. Some of their material will probably have been removed to Part 1, as noted above.

D2.4 National variation of Eurocode 7

As this commentary is drafted (early 1998), there is considerable pressure on drafters of Eurocodes to eliminate national variations. It appears likely that boxed values, as such, will not be allowed in the Euronorms. However, it is likely that for EC7 some form of national appendices will be retained, at least giving national values for safety factors. In July 1997, CEN/TC250/SC7 passed a resolution requiring ... *that the partial factors remain boxed in ... Basis of Design.* It is foreseen that development of EC7 to the status of a Euronorm will be impossibly difficult if this facility is not available.

It was noted in A1.5 that some of the available NADs for EC7 have been quite extensive, and have added to or amended the rules of the Eurocode. It is not clear to what extent this will be allowed for the final versions of Eurocodes, though the following resolution was passed by CEN/TC250 in September 1996:

CEN/TC250 accepts that EN 1997-1 may need to concentrate on the fundamentals of geotechnical design and may be supplemented by National Standards.

D3 RESEARCH AND DEVELOPMENT NEEDS

Careful study of EC7-1 raises many questions, and research needs could be found in many of its clauses. Most of these needs, however, are not special to EC7 but merely reflect the levels of knowledge and uncertainty which characterise geotechnical design. A selection of research needs which the use of EC7 will particularly emphasise is provided here.

D3.1 Application of partial factors

It was noted in D2.2 that an acceptable scheme of applying partial factors is still not agreed. Although this debate might temporarily be curtailed by the production of the Euronorm, it will doubtless recur until broad agreement is found. Engineers with practical knowledge of geotechnical design, together with a broad understanding of the purposes of factors of safety and the options for their implementation should be involved in this debate.

The continuing study of both successful geotechnical designs and failures could form the basis of useful research in this area. In particular, a clear understanding is needed of the calculated factors of safety of structures **as constructed**, noting that the constructed structure often incorporates elements of safety which are not included in the minimum design calculated to a code. In this study, both ULS and SLS failures should be considered, together with the relationship of these to factors of safety.

D3.2 Serviceability and deformations

The limit state approach requires more explicit consideration of serviceability limit states than has been the case previously. In part, this implies that the geotechnical profession must improve its ability to calculate deformations. EC7 therefore provides strong encouragement for continuing research into the deformation properties of soils, and the numerical methods needed to use these.

It should not be inferred, however, that EC7 encourages heavy numerical analysis where it is not needed. Geotechnical understanding of deformations is based mainly on case histories, leading either to simple empirical rules or to more complex back-analysis. The collection, categorisation and simple interpretation of case histories remains of paramount importance.

The relationship between deformation and mobilised strength requires further understanding. This is expressed in the 'mobilisation factor' of BS 8002.

D3.3 Statistics and probability methods

Across Europe, there is considerable interest in the application of statistical methods to geotechnical analysis. The training of British engineers is probably inferior to that of their European counterparts in this respect.

It is considered likely that statistical methods, well applied, could add to the geotechnical profession's understanding of uncertainty and safety in design. At worst, it is important that geotechnical engineers ensure that their work is not damaged by spurious, but plausible, uses of statistics which they are unable to challenge through lack of knowledge. The development of a research programme in this area is therefore to be encouraged.

Topics to be considered could include:

a statistical variations of common material properties, including study of their standard deviations and the significance of extreme values;

b the use of statistical methods in deriving characteristic values for material parameters;

c the relationship of factors of safety to probability of failure, and the use of reliability indices, β.

D3.4 Economy of design

The original purpose of the Eurocodes was to facilitate trade and fair competition in Europe. Studies will be needed to check whether this is being achieved, both before the Eurocodes become influential and as they become more dominant. Emphasis on the economy of design achieved in the various nations will be of particular importance.

Table D3.1 Progress with European NADs and comments on EC7-1 (ENV 1997)

Country	Position with ENV, NAD	Specific issues/problems
Austria	NAD published in 1996	Problems with errors in German translation. The 8 Austrian standards do not use γ factors but are being converted to EC7 design philosophy. Use Case B for structures, Case C for geometry. Same γ on c′ and tanϕ′
Belgium	Completion expected by end 1997. NAD may include specific design method from CPT results	For characteristic value will propose in NAD 'representative mean value' with table of pessimistic default values. Prefer low γ_F and high γ_m. Support Cases A, B, C. Want γ_{model}. As for Switzerland, those who use EC7 indicate 'interesting and easy', while those who do not indicate 'difficult'
Czech Republic	Available in Czech and on sale. Legal problems, with no Govt ministry taking overall responsibility. Likely to adopt lower factor on pile resistance (ENV Tables 7.1, 7.2 and 7.3) and reduced γ_G in Table 2.1 (eg from 1.5 to 1.4, 1.35 to 1.2 and 1.3 to 1.2)	Have used LSD for many years; taking characteristic values from existing standards, leading to reduced γ on loads. Does EC1 Table 9.2 apply only to buildings? Term 'Geotechnical design' not understood
Denmark	NAD published. Updating Danish code to EC7 (by end 1997?) – the 'Danish Eurocode'!	Some problems with Case B for retaining walls – leave out Case B?
Finland	Translation and NAD published; use is 'rare'. Available in English (see Table D3.2 for details of NAD)	Too many Cases; sometimes conservative; problems of interpretation with Chapter 7. How to deal with variable water levels?
France	ENV & NAD published; NAD is 'short'; legal problem: *all public works contracts must refer to French standards only*	Cannot calibrate to existing practice because of ambiguity of characteristic value definition; therefore new definition of characteristic value required, eg 'nominal value'. Unify Cases B, C? AFNOR mirror committee meets every 3 months; a testing committee of 20 people meets 10 times per year!
Germany	Commentary published; calculation examples to be published. See Table D3.3 for details of NAD	Over 500 calibrations point to need for 'safety classes' with γ on tanϕ varying from 1.2 to 1.3. Want other specific changes including omission of 5% in characteristic value definition
Greece	Translation complete; draft NAD in circulation but Govt approval not received. NAD does not change γ values and includes 50 pages of calculation methods	No use of EC7 except for Athens Metro, with mixing-up of Cases B, C? Greeks writing a guide book. EC1 limited to buildings
Holland	No translation yet. NAD ready but 'legal problems'; includes calculation methods. Some use by international contractors	Trial calculations indicate problems with sheet pile wall design
Ireland	NAD published but need EN to encourage use. In NAD, Paragraph 2.4.6(7): Total settlements limited to 25 mm not 50 mm. Adopts [boxed values] used in ENV. Paragraph 6.6.1(2) modified to read: *Normally, this depth may be taken as the greater of 1.5 times the width of the footing or the depth at which the effective vertical stress due to the foundation load amounts to 20% of the effective overburden stress*	Concern about Cases B, C for retaining walls
Italy	Translation of EC7-1 completed; distribution during 1997? Not known who will write NAD. No interest in Italy?	Some problems achieving Ministry Public Works recognition
Norway	Translation complete; draft NAD complete during 1997?	Problem with characteristic value definition; like 'National Annexes' concept
Portugal	Translation and NAD complete. Boxed γ's retained	EC7 not sufficient on rocks. National Regulations being developed
Slovakia	Translating EC7; completion and NAD expected during 1997?	Only small differences between national standards and EC7, usually concerning 'execution'. γ's similar. Term 'Geotechnical design' not understood
Spain	Translation published. NAD published – very short and boxed γ's retained	Emphasise importance of geological models as opposed to calculations. Cannot use EC7 to design. Important ground, eg rocks, partially-saturated soil not sufficiently covered
Sweden	Will complete translation in 1997. NAD will have only one case, not three cases A, B, C. Will have safety levels	Still testing Eurocode and NAD: results showing ~15% increase in costs of shallow foundations
Switzerland	NAD in German (not published); rejects section on Anchors	EC7 'OK' for those who have tried, not 'OK' from those who haven't

Table D3.2 Details of the German National Application Document

Features
Makes extensive reference to the following DIN documents: ● DIN V 1054-100 Calculation of partial safety factors used in earthworks and foundation engineering ● DIN V 4017-100 Calculation of design capacity of shallow foundations using the concept of partial safety factors ● DIN V 4019-100 Settlement calculations using the concept of partial safety factors ● DIN V 4084-100 Calculation of slope and terrain rupture using the concept of partial safety factors ● DIN V 4085-100 Calculation of earth pressures using the concept of partial safety factors ● DIN 4126-100 Design of diaphragm walls using the concept of partial safety factors
Uses three combinations of actions and three safety classes, grouped into three load cases
Replaces EC7, Table 2.1 with Table 3, taken from DIN V 1054-100
EC7, 3.2: introduces the use of a 'geotechnical expert' for Geotechnical Category 3 and (usually) GC 2 situations
Supplements EC7, 3.2.3: 'Design investigations' are introduced
EC7, 4.2.2: requires a construction log for GC 2 and 3
EC7, 6.3: a national standard on the interaction of stiff structures and ground is envisaged
EC7, 6.4: in Germany, ground is regarded as frost-free below 0.8 m
EC7, 7.4.1: 'analytical calculation methods' are excluded
EC7, 7.7.2.1: reference is made to Arbeitskreis 'Baugruben' (EB 62) (Excavation Working Group) for the interaction of grouped piles in tension
EC7, 8.3.2.2: additional safety margins apply
EC7, 8.5.1: the only differentiation between ULS design earth pressure and SLS design earth pressure is in the use of different γ values
EC7, 8.5.4: DIN V 4085-100 gives more detailed specifications

Table D3.3 Partial safety factors for actions

ULS	Action	Symbol	Load Case 1	Load Case 2	Load Case 3
1A	Permanent, unfavourable	γ_{Gsup}	1.00	1.00	1.00
	Permanent, favourable	γ_{Ginf}	0.90	0.90	0.95
	Liquid Pressure	γ_F	1.00	1.00	1.00
	Variable, unfavourable	γ_{Qsup}	1.50	1.00	1.00
1B	Permanent, unfavourable	γ_{Gsup}	1.35	1.20	1.00
	Permanent, favourable	γ_{Ginf}	1.00	1.00	1.00
	Liquid Pressure	γ_F	1.35	1.20	1.00
	Variable, unfavourable	γ_{Qsup}	1.50	1.30	1.00
	Perm. Transverse Pressure	γ_H	1.35	1.20	1.00
	Perm. Shaft Resistance	γ_M	1.35	1.20	1.00
	Perm. Earth Pressure	γ_{Eg}	1.35	1.20	1.00
	Variable Earth Pr, unfav	γ_{Eq}	1.50	1.30	1.00
	Earth Pr. at rest, perm	γ_{E0g}	1.20	1.10	1.00
	Earth Pr. at rest, var, unfav	γ_{E0q}	1.35	1.20	1.00
1C	Permanent	γ_G	1.00	1.00	1.00
	Liquid Pressure	γ_F	1.00	1.00	1.00
	Variable, unfavourable	γ_{Qsup}	1.30	1.20	1.00
	Perm. Transverse Press.	γ_H	1.00	1.00	1.00
	Perm. Shaft Resistance	γ_M	1.00	1.00	1.00
	Perm. Earth Pressure				
	Variable Earth Pressure				
2	Permanent	–	1.00	1.00	1.00
	Variable		1.00	1.00	1.00

Table D3.4 Partial safety factors for soil resistances

ULS	Soil Resistance	Symbol	Load Case 1	Load Case 2	Load Case 3
1B	Passive Earth Pressure	γ_{Ep}	1.40	1.30	1.20
	Bearing Capacity	γ_S	1.40	1.30	1.20
	Sliding Capacity	γ_{St}	1.50	1.35	1.20
	Piles, axial	γ_P	1.40	1.20	1.10
	Injection anchors	γ_A	1.10	1.10	1.10
	Soil nails	γ_N	1.20	1.10	1.05
	Flexible reinforcement	γ_B	1.40	1.30	1.20
1C	$\tan\phi$	γ_ϕ	1.25	1.15	1.10
	c'	γ_c	1.60	1.50	1.40
	c_u	γ_{cu}	1.40	1.30	1.20
	Piles, axial	γ_P	1.60	1.40	1.20
	Injection anchors	γ_A	1.30	1.20	1.10
	Soil nails	γ_N	1.30	1.20	1.10
	Flexible reinforcement	γ_B	1.40	1.30	1.20

Table D3.5 Details of the Finnish National Application Document

Features
NAD + SFS-ENV 1997-1:1994 presents alternatives to the Collection of Finnish Construction Regulations (B1, B3)
Adds specific recommendations for highway structures
Paragraph 2.1(5): use of Geotechnical Categories (GCs) may become a Principle, not an Application Rule
Paragraph 2.1(2)P: attention to specific Finnish geological and climatic conditions
Paragraph 2.1(5): further definition of the GCs for Finnish use
Paragraph 2.4.2(10)P: for stability analysis, assume 50-year worst case ground water level; for settlement analysis, use average maximum and minimum values
Paragraph 2.4.2(14)P–Table 2.1: Case C γ values changed as follows: c_u – 1.55, q_u – 1.6; furthermore, γ_ϕ and γ_c can be decreased by up to 10% for transient loading where risk of material damage is minor, and **shall be** increased by 10% where risk of injury or material damage is large. Similarly, γ_{cu} and γ_{qu} can be decreased by up to 15% and shall be increased by 20%
Paragraph 2.4.3(5)P and (6): the definition of characteristic value is changed somewhat and the reference to the use of statistical methods is removed
Paragraph 2.4.6(7): additional limiting values of rotation are quoted for structures of different materials
Paragraph 3.2.3(9)P: calls for closer spacing of exploration points than does the ENV
Calls up Finnish site investigation, soil and rock classification and testing documentation
Clause 3.3: introduces Weight Sounding tests
Paragraph 3.4.1(1)P: the presentation system in publication SGY 201 is used
Paragraph 3.4.1(2): 'radon' and 'frost susceptibility' added
Section 5: several constraints and amendments applied to the use and properties of fill materials
Clause 6.2: several additions concerning frost heave and its avoidance
Paragraph 6.6.1(2): additional text on compressible and organic soils and ground water level change
Paragraph 7.3.2.2(2): Addition: negative skin friction and transient actions need not be considered simultaneously
Paragraph 7.4.2(4)P: Add:'Handling and transport of piles'
Paragraph 7.4.2(5): Pile design classified as GC 2 or 3
Paragraph 7.6.3.2(6)P, Table 7.1: replaced by an extensive set of factors according to number of loaded piles, whether tests are static or dynamic
Paragraph 7.6.3.3(4)P: factor value changed to 1.6
Paragraph 7.6.3.3(9)P: introduces a plugging coefficient
Paragraph 7.6.3.4(1)P: specifies minimum values for the product $\xi \times \gamma_t$
Paragraph 7.7.2.3(3): Addition: *Characteristic tensile resistance of tensile piles ... assessed from characteristic compression resistance of ... shaft ÷ 2 (long-term) or ÷ 1.6 (short-term)*
Paragraph 7.9(5): delete comment on established practice
Paragraph 7.10(5)P: Storage period for pile records and other documents at discretion of builder/client; as-built drawings stored for service life
Section 8: extensive reference to RIL 181 and RIL 194
Paragraph 8.3.2.1(2): Additional requirement: ... *a level surface load of at least $q = 10$ kN/m² should be assumed in allowance of over-fill ... etc*
Paragraph 8.3.2.2(1)P: specific guidance on determination of design water level, with γ values related to duration of period of water level observation
Paragraph 8.8.5(6)P: changes γ value from 1.25 to 1.3 for temporary anchorages
Paragraph 9.2(1)P: warns of frost melting
Paragraph 9.3(3)P: ... *most unfavourable value occurring during the service time, or the value recurring once during 50 years, is selected as the characteristic value ...*
Annexes: ... *little experience ... of ... some ... in Finnish conditions ... should calibrate ... to the Finnish practice ... (without) sufficient information and earlier experience ...*

Table D3.6 Details of the Irish National Application Document

Features
Paragraph 2.4.2(15): For ease of application, a table of öd values is provided for Cases A, B and C
Paragraph 2.4.6(7): total settlements for normal strip and pad footings are limited to 25 mm not the 50 mm in EC7
Paragraphs 3.1(3), 3.2.3(6)P and 3.3.2(2)P: Reference Standards: Relevant documents are BS 1377 and BS 5930
Paragraphs 5.3.2(1)P and 5.3.4(2): Proctor density to be derived using Method 3.4 of BS 1377-4
Paragraph 6.4(2)P: width of foundation should not be less than that specified in Part E of Technical Guidance Document A of the Building Regulations
Paragraph 6.6.1(2), fourth sentence, amended to read: *Normally, this depth may be taken as the greater of 1.5 times the width of the footing or the depth at which the effective vertical stress due to the foundation load amounts to 20% of the effective overburden stress*
Paragraph 7.6.3.3(4): a clarifying statement is made, similar to the U.K. NAD
Paragraphs 8.3.2.1(1) and 8.3.2.1(2) are both to be satisfied
Paragraph 8.6.6(4) (and 2.4.2(15)): a model factor of unity to be used for Cases A, B and C

Eurocode 7: a commentary

Part E Worked examples

CONTENTS

E1 INTRODUCTION

Part E of the commentary consists of worked examples, to which reference is made in the other parts. The examples have been developed to illustrate specific points about the application of EC7, and this also governs the amount of detail presented. All examples follow the rules of EC7 and are presented in a step by step approach demonstrating the checks that are required in carrying out a conforming design. Some include more detailed explanation than would normally be found in design calculations, whilst in other cases detail has been omitted so as not to obscure the points being illustrated.

The design examples cover a range of common geotechnical structures and are divided into spread footings, piles, walls and slopes as shown in the Table E1.1.

Further examples of the application of EC7 may be found in Frank (1994), DIN (1996), Orr and O'Brien (1996), Simpson (1996) and Carder (1998).

Table E1.1 Design examples

Example no	Structure type	Design example title	Main EC7 clauses	
E2	Spread footing	ULS design of a pad footing on cohesionless ground	2.4.2 Annex B	Section 6 6.5.1 and .2
E3	Spread footing	ULS design of a pad footing - short term loading on stiff clay	2.4.2 Annex B	Section 6 6.5.1 and .2
E4	Spread footing	Design of spread foundation for a tower	2.4.2 Annex B	Section 6 6.5.1 to .4
E5	Spread footing	Application of Annex D: SLS settlement check	Annex D	
E6	Pile	Design of a compression pile	2.4.2	Section 7 7.6.3.3
E7	Pile	Characteristic capacity of compression piles derived from pile load tests		Section 7 7.6.3.2
E8	Pile	Derivation of base and shaft components from a compression pile load test	2.4.2	Section 7 7.6.3
E9	Pile	Input to structural design of a pile in heaving ground	2.4.2	Section 7 7.3.2.3
E10	Pile	Piles subject to downdrag		7.3.2
E11	Pile	Input to structural design of a tension pile	Annex F	Section 7
E12	Pile	Examples for piles in tension due to buoyancy effects of an underground station	2.4.2	Section 7 7.7.2.2
E13	Wall	Design of a concrete stem wall	2.4.2 Annex G	Section 8 8.5.1 8.5.4 and .5 8.7.2
E14	Wall	Design of a cantilever sheet pile wall	Annex G	Section 8 8.3.2
E15	Walls	Design of a propped embedded wall	Annex G	Section 8 8.6.1
E16	Ground anchor	Design of a ground anchor		Section 8 8.6.2 and .5 8.8.2 and .5
E17	Slope	Design of a slope in drained ground	2.4.2	Section 9 9.5
E18		ULS check on a simple, potentially buoyant structure		

E2 ULS DESIGN OF A PAD FOOTING ON COHESIONLESS GROUND

E2.1 Introduction to E2 to E4

Design examples E2 to E4 look at ULS stability for spread footings. E2 considers the drained bearing capacity of a footing on a medium dense sand and gravel layer. E3 uses the same load case as the first example but has a firm to stiff clay as the foundation stratum. In E4, the stability of a tall light structure with wind loading is considered. All three examples make use of the equations for bearing capacity in Annex B and the clauses in Section 6 of EC7.

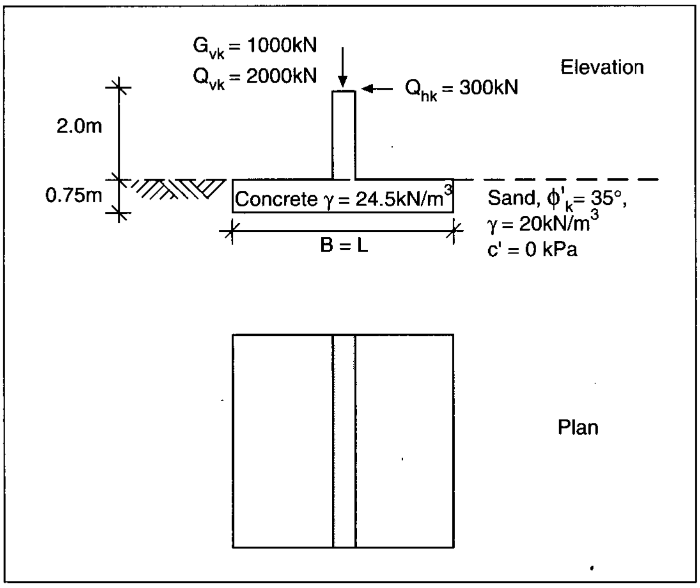


Figure E2.1 Spread footing loads and geometry

E2.2 Data and load factors for E2

The pad footing shown in Figure E2.1 is to be designed for a structure with large live load. The foundation stratum is medium dense sand and gravel with characteristic shear strength parameters of $\phi' = 35°$ and $c' = 0$ kPa. The characteristic loading conditions are shown in Table E2.1.

Table E2.1 Characteristic actions and partial factors

	Action (kN)	Case B: Partial factor (γ) Unfavourable	Case B: Favourable	Case C: Partial factors (γ) Unfavourable	Case C: Favourable
Permanent:					
vertical	$G_{vk} = 1000$	$\gamma_G = 1.35$	1.0	1.0	1.0
Variable:					
vertical	$Q_{vk} = 2000$	$\gamma_Q = 1.5$	0	1.3	0
horizontal	$Q_{hk} = 300$	$\gamma_Q = 1.5$	0	1.3	0

The vertical and horizontal components of the variable load derive from a single source, hence they are factored together. A moment results from the horizontal load which is applied to the structure at 2 m above the top of the pad footing.

In addition to the actions that are applied to the structure, the vertical weight of the pad footing should also be included in the bearing capacity calculation. Unit weights of the ground and concrete are 20 kN/m³ and 24.5 kN/m³ respectively. The water table is at depth and can be ignored for this example.

E2.3 ULS calculations

In order to satisfy EC7, Cases B and C must both be satisfied. For Case B the required dimension of the 0.75 m deep footing was calculated to be 2.5 m square, while for Case C the required dimension was 3.0 m square. These dimensions were found by iteration. As Case C governed sizing of the footing in this instance, the calculations for the final iteration of this case will be presented below.

E2.3.1 Case C

Design vertical action at footing base:

Vertical action $(V_d) = \gamma_G (G_{vk} + W_{vk}) + \gamma_Q Q_{vk}$ (W_{vk} = Weight of pad)

$= 1.0 \times (1000 + 24.5 \times 3.0 \times 3.0 \times 0.75) + 1.3 \times 2000$

$= 1000 + 165 + 2600$

$= 3765$ kN.

Similarly the design horizontal and moment action effects at footing base:

Horizontal action $(H_d) = \gamma_G G_{hk} + \gamma_Q Q_{hk}$

$= 1.0 \times 0 + 1.3 \times 300$

$= 390$ kN

Moment action $(M_d) = \gamma_G G_{mk} + \gamma_Q Q_{mk}$

$= 1.0 \times 0 + 1.3 \times (2 + 0.75) \times 300$

$= 1073$ kN.m.

The expressions for bearing capacity factors and associated shape factors are given in Annex B of the Eurocode, and discussed below. For $\phi_k' = 35°$ and $\gamma_\phi = 1.25$ (from Table 2.1), the design value $\phi_d' = \tan^{-1}((\tan 35°) / 1.25) = 29.3°$. The associated design bearing capacity factors for N_q and N_γ are 16.9 and 17.8 respectively (N_c is not used as $c' = 0$).

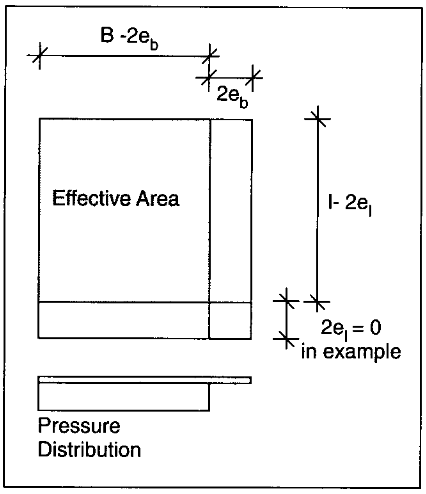


Figure E2.2 Effective area for eccentric loading

Effect of moment loading

The effect of a moment loading on the footing is to move the centroid of the loaded area of the pad base away from the centre of the footing. To account for this in the calculation of allowable bearing capacity, it is assumed that the resistance of the ground to the applied actions is centred on the centre of gravity of the applied load. As shown in Figure E2.2, the zone of unused base has a width of $2e_b$ where e_b is the load eccentricity. There is no eccentricity in the perpendicular direction, so $e_l = 0$. Hence the Effective Area (A′) in this case is $L(B - 2e_b)$.

$e_b = M_d / V_d$

$= 1073 / 3765$ (kN.m/kN)

$= 0.285$ m.

Hence $B' = B - 2e_b$

$= 3 - 2 \times 0.285 \qquad = 2.43$ m

$L' = L - 2e_l \qquad = L$.

ULS design vertical bearing resistance

The ULS design vertical bearing resistance is a combination of three components, namely the cohesive strength of the ground, the vertical effective stress in the ground at the formation level of the spread footing and the buoyant unit weight of the ground beneath the spread footing. These three components are combined with factors depending on the size of the footing and the load combinations applied to the footing as shown below:

R / A' = Cohesion + Effective stress + Density

$= N_c c' s_c i_c + q' N_q s_q i_q + 0.5\gamma' B' N_\gamma s_\gamma i_\gamma$

where R = design bearing resistance

N = bearing capacity factor

s = spread footing shape factor

i = load inclination factor

q' = effective stress at formation level

γ' = buoyant weight of ground beneath footing.

Bearing capacity factors

The following equations for the bearing capacity factors are taken from EC7, Annex B:

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

$N_\gamma = 2(N_q - 1) \tan\phi'$ when $\delta \geq \phi' / 2$ (rough base)

$N_c = (N_q - 1) \cot\phi'$.

Effect of shape

The above expressions for bearing capacity factors (N_q etc) are for a strip footing. Where the spread footing is not a strip, the change in ratio of dimensions must be accounted for using shape factors s_q, s_γ and s_c (s_q and s_c are greater than or equal to unity while s_γ is less than or equal to unity). This is done by considering the ratio of effective width to effective length, ie:

$s_q = 1 + (B' / L') \times \sin(\phi_d')$
$= 1 + (2.43 / 3.0) \times \sin 29.3 \qquad = 1.45.$
$s_\gamma = 1 - 0.3(B' / L')$
$= 1 - 0.3(2.43 / 3.0) \qquad = 0.76.$

s_c is not needed because c' is equal to zero in this example.

Effect of loading inclination

Horizontal loads change the inclination of the resultant load that is distributed over the effective area of the footing. Expressions for inclination factors are given for the situation where the horizontal load is parallel to B′ and where it is parallel to L′. In this case, the horizontal load is parallel to B′.

$i_q = (1 - 0.7\,H / (V + A' \times c' \times \cot\phi_d'))^3$
$= (1 - 0.7 \times 390 / 3765)^3 \qquad = 0.80.$
$i_\gamma = (1 - H / (V + A' \times c' \times \cot\phi_d'))^3$
$= (1 - 390 / 3765)^3 \qquad = 0.72.$

i_c is not needed because c' is equal to zero in this example.

The ULS design bearing pressure for Case C is

$R / A' = N_c\, c'\, s_c\, i_c + q'\, N_q\, s_q\, i_q + 0.5\gamma'\, B'\, N_\gamma\, s_\gamma\, i_\gamma$ (EC7, Eq B2)
$= 0 + (0.75 \times 20) \times 16.9 \times 1.45 \times 0.8 + 0.5 \times 20 \times 2.43 \times 17.8 \times 0.76 \times 0.72$
$= 0 + 283 + 236$
$= 530\ \text{kN/m}^2.$

Note that in this case, the water table is at depth and hence $q' = q$ and $\gamma' = \gamma$.

The required ULS design bearing resistance is

$Q / A' = 3765 / (3.0 \times 2.43)$
$= 516\ \text{kN/m}^2 < 530\ \text{kN/m}^2$, hence OK.

E2.3.2 Case B

As stated previously Case B is not critical for sizing of the spread footing in this example. This is demonstrated by checking Case B using the same method as Case C, with the appropriate partial factors on load and ground shear strength.

E2.3.3 Overall factor of safety

For comparison, if the partial factors on load and shear strength are set to unity then the ULS bearing resistance is equal to 8950 kN for the 3 m square footing. This is equivalent to a factor of safety on applied characteristic load of 2.8.

E2.3.4 ULS structural design

Once the size of the footing is fixed it is necessary to consider the applied forces to allow for the structural design of the footing. In Table E2.1 above it is clear that the partial factors applied to actions are greater in Case B than in Case C. For structural design Case B governs. Hence for structural design the design actions are (Case B):

Vertical action $(V_d) = \gamma_G\,(G_{vk} + W_{vk}) + \gamma_Q\, Q_{vk}$
$= 1.35 \times 1000 + 1.35 \times 4.5 \times 3.0 \times 3.0 \times 0.75 + 1.5 \times 2000$
$= 1350 + 223 + 3000$
$= 4573\ \text{kN}.$

Similarly the design horizontal and moment action effects at footing base are:

Horizontal action (H_d) $= \gamma_G\, G_{hk} + \gamma_Q\, Q_{hk}$
$= 1.35 \times 0 + 1.5 \times 300$
$= 450$ kN

Moment action (M_d) $= \gamma_G\, G_{mk} + \gamma_Q\, Q_{mk}$
$= 1.35 \times 0 + 1.5 \times (2 + 0.75) \times 300$
$= 1238$ kN.m.

These actions should then be used in design of the footing using the appropriate EC2 for reinforced concrete design, assuming a linear distribution of bearing pressure beneath the footing (EC7, 6.8(2)).

E2.4 Settlement assessment

Assessment of the settlement for this footing is given in E5.

E3 ULS DESIGN OF A PAD FOOTING – SHORT TERM LOADING ON STIFF CLAY

E3.1 Data

The same loads are applied in this example as the footing in E2, but in this case the pad footing is on a stiff overconsolidated clay. The characteristic strength of the clay, c_{uk}, is 75 kPa and a check on undrained conditions only is carried out in this example, as the critical loading is short-term.

E3.2 Bearing capacity calculations

Using the formulae in Annex B.2 for undrained conditions, the length of side of the footing for Case B was 3.75 m while for Case C is was 4.0 m. Hence, as for E2, Case C is critical and B = L = 4.0 m. This may be checked as follows.
For Case C, the design vertical action at footing base:

Vertical action $(V_d) = \gamma_G (G_{vk} + W_{vk}) + \gamma_Q Q_{vk}$ (W_{vk} = weight of pad)
$= 1.0 \times (1000 + 24.5 \times 4.0 \times 4.0 \times 0.75) + 1.3 \times 2000$
$= 1000 + 294 + 2600$
$= 3894$ kN.

The design horizontal and moment actions are as in E2, 390 kN and 1073 kNm, respectively.

In Case C the design strength for the clay incorporates the partial factor γ_{cu} of 1.4 to give:

$c_{ud} = c_{uk} / \gamma_{cu}$
$= 75 / 1.4$
$= 53.5$ kN/m^2.

The allowable bearing resistance is $R / A' = (2 + \pi)\, s_c\, i_c\, c_{ud} + q$ (EC7, Eq B1)
where R = design bearing resistance
A' = effective area
$s_c = 1 + 0.2\,(B' / L')$
$i_c = 0.5\,(1 + (1 - H / A' c_{ud})^{1/2})$
q = total overburden pressure at base level.

The effective width B' is equal to 3.45 m giving the shape factor (s_c) a value of 1.17.

The design horizontal load is equal to 390 kN and the effective area is equal to 13.8 m^2 giving the inclination factor a value of 0.84.

Hence the ULS design vertical bearing resistance is:

$R / A' = (2 + p)\, s_c\, i_c\, c_u + q$
$= 5.14 \times 1.17 \times 0.84 \times 53.5 + 0.75 \times 20$
$= 285$ kN/m^2.

The required ULS design vertical bearing resistance is:

$V_d / A' = 3894 / (4.0 \times 3.45)$
$= 282$ kN/m^2 < 285 kN/m^2, hence OK.

In Paragraph 6.5.3(9)P the horizontal shear action at foundation level is limited to $A'c_u$. The undrained shear strength of the ground c_u used here is that available at the soil structure interface; 'adhesion' would be a better term, as used in 8.5.1(4). (If this is taken to be equal to c_u in the body of the clay, then the formula for the inclination factor can only be evaluated for $S_d \leq A'c_u$.)

In Paragraph 6.5.3(9)P a second limit is placed on the horizontal shear where water may penetrate to the base of the footing and prevent suctions from developing (see Figure E3.1). In this case, the ratio S_d / V_d is limited to 0.4. For this example the ratio of H_d / V_d is $390 / 3894 = 0.1 < 0.4$, hence OK.

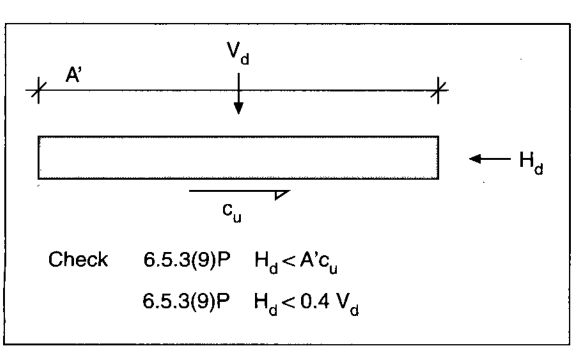


Figure E3.1 Limit on horizontal loads

E3.2.1 Overall factor of safety

If the partial factors on load and shear strength are set to unity then the ultimate bearing resistance is equal to 5852 kN; the characteristic vertical load is 3294 kN. This is equivalent to a factor of safety on applied characteristic load of 1.78.

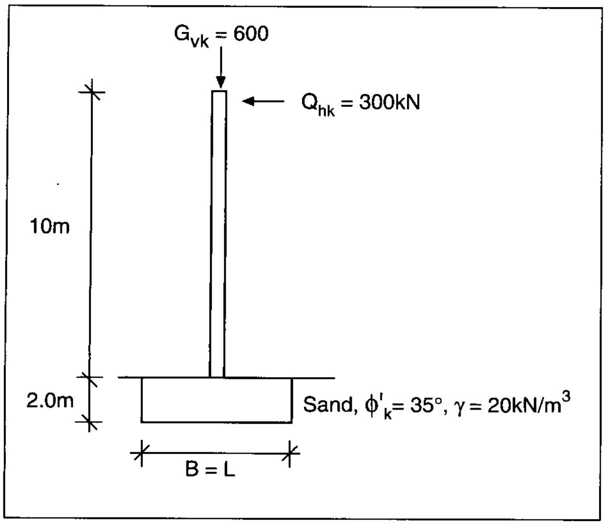


Figure E4.1 Loading for a tall, lightweight structure

E4 ULS DESIGN OF SPREAD FOUNDATION FOR A TOWER

E4.1 Data and load factors

This example considers the bearing capacity of a tall lightweight structure which is subjected to significant horizontal loading (eg a windmill or chimney). The characteristic actions and partial factors are shown in Figure E4.1 and Table E4.1.

A moment results from the horizontal load which is applied to the structure at 10 m above the top of the pad footing. The pad footing is 2 m deep and is situated on a dry medium dense sand and gravel layer with $\phi' = 35°$ and $c' = 0$ kPa.

Table E4.1 Characteristic actions and partial factors

	Action (kN)	Case B: Partial factor (γ) Unfavourable	Favourable	Case C: Partial factors (γ) Unfavourable	Favourable
Permanent:					
vertical	$G_{vk} = 600$	$\gamma_G = 1.35$	1.0	1.0	1.0
Variable:					
vertical	$Q_{vk} = 0$	$\gamma_Q = 1.5$	0	1.3	0
horizontal	$Q_{hk} = 300$	$\gamma_Q = 1.5$	0	1.3	0

E4.2 Calculations

The required size of footing was calculated for both cases B and C. For Case B, it is necessary to consider the permanent vertical actions both as favourable and unfavourable. In this example, Case B with the permanent vertical actions favourable was found to be the critical case with a required footing dimension of 5.6 m square. The required dimension for Case C was 5.4 m square. Hence, Case B was marginally critical for sizing of the pad footing (ie critical for geotechnical stability). This possibility was discussed in B5.4.

For Case B with favourable permanent vertical load, the design actions are:

Vertical $V_d = (600 + 5.6 \times 5.6 \times 2 \times 24.5) \times 1.00$
$= 2137$ kN

Horizontal $H_d = 300 \times 1.5$
$= 450$ kN

Moment $M_d = 300 \times (10 + 2) \times 1.5$
$= 5400$ kNm.

The calculations showed that for such a high, light structure the moment induced eccentricity resulted in the resultant force passing close to the edges of the footing:

$e_b = M_d / V_d$
$= 5400 / 2137 = 2.53$ m

and $B' = B - 2e_b$
$= 5.6 - 2 \times 2.53$
$= 0.54$ m.

Using the equations in Annex B and as used in E2 the design bearing resistance R is:

$$R / A' = N_c c' s_c i_c + q' N_q s_q i_q + 0.5\gamma' B' N_\gamma s_\gamma i_\gamma \qquad \text{(EC7, Eq B2)}$$
$$= 0 + (2 \times 20) \times 33.3 \times 1.05 \times 062 + 0.5 \times 20 \times 0.54 \times 45.2 \times 0.97 \times 0.49$$
$$= 0 + 871 + 116$$
$$= 987 \text{ kN} / \text{m}^2.$$

The required ULS design vertical bearing resistance is:
$Vd / A' = 2137 / (5.6 \times 0.54)$
$= 707\ \text{kN/m}^2 < 987\ \text{kN/m}^2$, hence OK.

In this situation a small increase in applied horizontal force to the structure (and hence an increase in moment) would lead to overturning of the foundation and hence in this situation it is vital to size the footing to allow for unforeseen situations. EC7, 6.5.4 requires that in situations such as these, where the design load passes outside the middle ⅔ of the footing, an allowance must be made for construction tolerances. This will typically require that the footing be extended a further 0.1 m, increasing B′ to about 0.64 m and the footing dimensions to 5.8 m square. (In situations such as this a piled foundation solution may result in a lower cost foundation.)

The resulting bearing pressure for the factored loads of Case B is 707 kPa. For the serviceability loading it is less than 200 kPa, the precise value depending on the assumed distribution of bearing pressure.

The factor of safety against bearing capacity failure using characteristic loads and soil shear strengths is 10.3. This appears to be large and is mainly a result of the large moment loading and its influence on the magnitude of B′ in addition to increases in the bearing capacity factors and inclination factors. For SLS considerations the potential of the footing to rock under wind loading should be considered, as in E5.4.

E5 APPLICATIONS OF ANNEX D – SLS SETTLEMENT CHECK

E5.1 Data

Considering the load case for the pad footing in E2, it is necessary to calculate the settlement of the footing for SLS conditions.

The ground conditions at the footing are medium-dense sand and gravel to a depth of 7 m with an average SPT value of 20. This sand and gravel layer overlies a 13 m depth of overconsolidated clay. The characteristic Young's moduli of these two layers are $E' = 50$ MPa for the sand and gravel and $E' = 15$ MPa increasing to 32 MPa at 13 m below the clay surface for the clay. At 20 m depth there is a rigid rock stratum.

Paragraph 2.4.2(18)P states that partial factors of unity shall be used for all permanent and variable actions for verification of the serviceability limit state. Hence, the design actions are:

Vertical $V_d = 3165$ kN
(Horizontal $H_d = 300$ kN)
(Moment $M_d = 825$ kN.m).

Annex D proposes two methods (and two variants). Both methods are presented below. Only movements under vertical loading will be considered in E5.2 and E5.3. Rocking will be checked in E5.4.

E5.2 Method D.1

Isotropic elasticity is assumed to apply to this situation. The settlement of the loaded area may be calculated by hand or by using proprietary software.

Fadum (1948) presents relationships for the change in vertical stress beneath an uniformly distributed load in the form:

$\Delta\sigma_z = \Sigma\, qI_\sigma$

where q is the uniformly distributed load

I_σ is the influence factor depending of geometry of loaded area and position of soil element relative to loaded area. Stress changes are calculated for a set of loaded areas all of which have a corner above the point being investigated.

For a point some 2 m beneath the centre of the loaded area of dimensions 3 m by 3 m, the change in stress for the imposition of a UDL of 352 kN/m^2 is:

$\Delta\sigma_z = \Sigma\, qI_\sigma$.

The Equivalent Loaded Area is 4 rectangles with dimensions 1.5 m by 1.5 m arranged symmetrically above the point at 2 m depth (superposition is assumed). The value of factor I_σ for one of these areas is 0.136 (Fadum (1948))

$= 4 \times 352 \times 0.136$

$= 192$ kN/m^2.

The strain in this element will be influenced by both the change in vertical and in horizontal stresses. For this element the change in horizontal stress is calculated to be approximately 6% of the change in vertical stress (in this instance). The vertical strain (ε_z) can then be calculated using the equation:

$\varepsilon_z = \Delta\sigma_z / E' - \nu'\Delta\sigma_x / E' - \nu'\Delta\sigma_y / E'$ (x and y are horizontal directions)

$= \Delta\sigma_z / E'\,(1 - 2 \times 0.3 \times 0.06)$ (ν' is 0.3 in this case)

$= 192 / 50\,000\,(1 - 0.04)$

$= 0.37\%$.

To obtain the full settlement of the loaded area, it is necessary to integrate along a vertical line the strain magnitude in each element of soil; the smaller the elements the more accurate the solution.

This process of integration is most easily carried out on a computer. The results of a settlement analysis obtained using the *Oasys* program VDISP are shown on Figure E5.1. The maximum settlement is 24.3 mm, situated in the centre of the loaded area, reducing to an average of 15 mm along each edge. To compare the strain calculated by the program and that calculated above, the displacement at the centre line of the loaded area at 1.9 m depth was 17.0 mm while at 2.1 m depth it was 16.3 mm. Hence the computed strain was (17.01 – 16.34) / 200 × 100% = 0.34% and this compares reasonably with the hand calculated strain above. This settlement calculation ignores the stiffness of the structure.

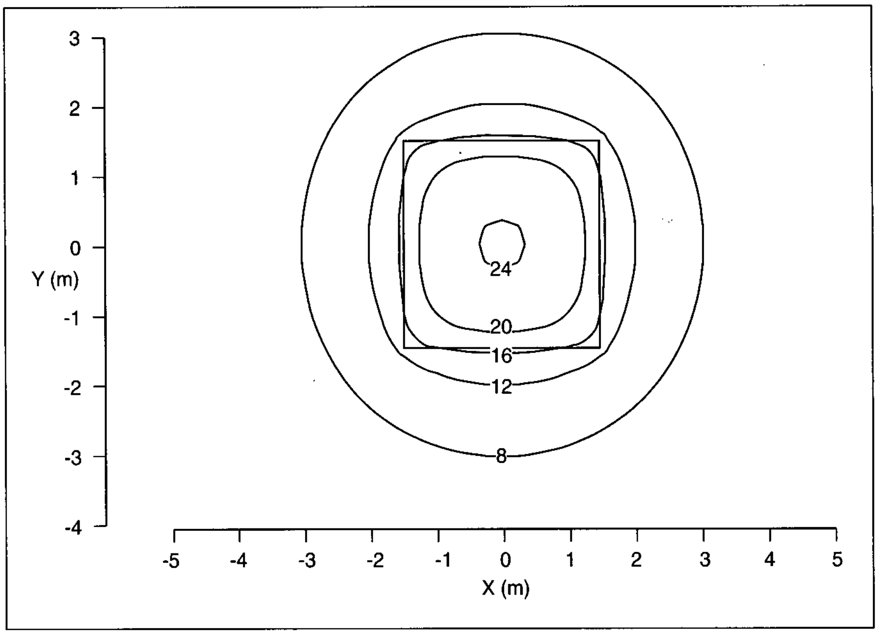


Figure E5.1 Settlement analysis for spread footings (settlement in mm)

E5.3 Method D.2

In part 2 of Annex D, the 'Adjusted Elasticity Method' is presented. This method allows the results of field measurements or soil tests to be used to provide a back calculated stiffness for the ground which can then be used elsewhere on a particular site. A simple form of equation is presented (Eq. D.1), a variation of which is presented below (settlement for a rigid rectangular element):

$\rho_z \quad = q\,(BL)^{1/2}\,(1 - \nu^2) / (\beta_z E)$

where β_z is a factor depending on the ratio L / B (increasing from 1.06 to 1.4 for L / B = 1 to 10).

Hence, for the loaded foundation above with $q = 352$ kN/m^2 and B = 3.0 m, L = 3.0 m and E = 50 000 kN/m^2, the plate settlement is:

$\rho_z \quad = 352\,(3.0 \times 3.0)^{1/2}\,(1 - 0.3^2) / (1.06 \times 50\,000)$

$\quad = 19.9$ mm.

This uniform settlement is similar to the average settlement calculated using the computer program. The inclusion of the clay layer in the VDISP calculation will cause larger settlement at depth as compared to the assumption in the Method D.2 calculation which assumes uniform ground properties.

E5.4 Calculation of the effect of moment loading

Moment loading could be incorporated in Method D.1 by dividing the vertical load into strips that would result in the correct moment being applied about the centre of the footing. The result on moment loading would not significantly alter the settlement at the centre point of the footing. In Method D.2, equations similar to those for settlement calculation exist for moment loading (Poulos and Davis (1974)) as shown above for E4.

From the data in E4, the design loadings for SLS conditions at the base of the 5.6 m square base are:

$V_d = 2137$ kN
$H_d = 300$ kN
$M_d = 3600$ kN.

The resulting load eccentricity is 1.68 m (3600 / 2137), well outside the middle third of the foundation. The middle third check is useful as it helps the designer to visualise if part of the foundation will detach during loading; it is not, however, an EC7 criterion (see C6.5.4(1)P). Where the load eccentricity lies outside the middle third of the foundation base, it is possible that an elastic calculation of rocking will underestimate movements. In this example the fact that the resulting load is outside the middle third of the footing will be considered in an approximate fashion during the calculation of footing rotation.

Poulos and Davis (1974) state that the rotation of the foundation can be calculated using the following equation:

$$\phi = M\,(1 - \nu^2)\,I_\theta / (b^2 \times l \times E)$$

where ϕ = rotation in radians
M = moment in the direction of dimension 'B'
ν = Poisson's ratio of ground
I_θ = geometrical factor
b, l = dimensions of footing, positive contact area
E = Young's modulus of ground.

The eccentricity of the characteristic load e_b is 1.68 m and hence the effective width B′ is (5.6 – 2 × 1.68) 2.23 m. This effective width is the dimension which when loaded by a uniformly distributed load results in the appropriate vertical reaction to the applied vertical and moment loading. When, as in this case, the eccentricity is outside the middle third, the positive contact width will be 1.5 × B′ or 3.35 m (clearly when the eccentricity is inside the middle third the positive contact width is B). The calculation of footing rotation can then be carried out using b = 3.35 m and l = 5.6 m. The assumed ground conditions are E = 50 000 kN/m^2 and ν = 0.3 and the rotation of the footing is:

$$\phi = \pm M\,(1 - \nu^2)\,I_\theta / (b^2 \times l \times E)$$
$$= \pm 3600 \times (1 - 0.3^2) \times 3.4 / (3.35^2 \times 5.6 \times 50{,}000)$$
$$= \pm 0.0035 \text{ rad.}$$

The actual elastic rocking δ is :

$$\delta = \pm \phi B / 2$$
$$= \pm 0.0035 \times 5.6 / 2$$
$$= \pm 9.9 \text{ mm.}$$

The rocking movement at the top of the tower will be ± 42 mm and this rocking movement must be checked against the requirements of the project. If it is not acceptable, a further increase in foundation size, or a change of foundation type, will be required.

E6 DESIGN OF A COMPRESSION PILE

E6.1 Data

A structure is to be supported on bored piles. Each pile supports a single column with vertical characteristic loading of $G_k = 1000$ kN (permanent) and $Q_k = 300$ kN (variable). Based on this load information and the soil properties shown in Figure E6.1 it is necessary to calculate the required toe level of a 0.6 m diameter bored pile.

The following mean relationships for pile resistance will be used, using the subscript 'μ' to denote mean, or most probable, ground properties and resistances. The relationships must be based on the results of a large number of pile tests in similar ground conditions, as per 7.4.1(1)P.

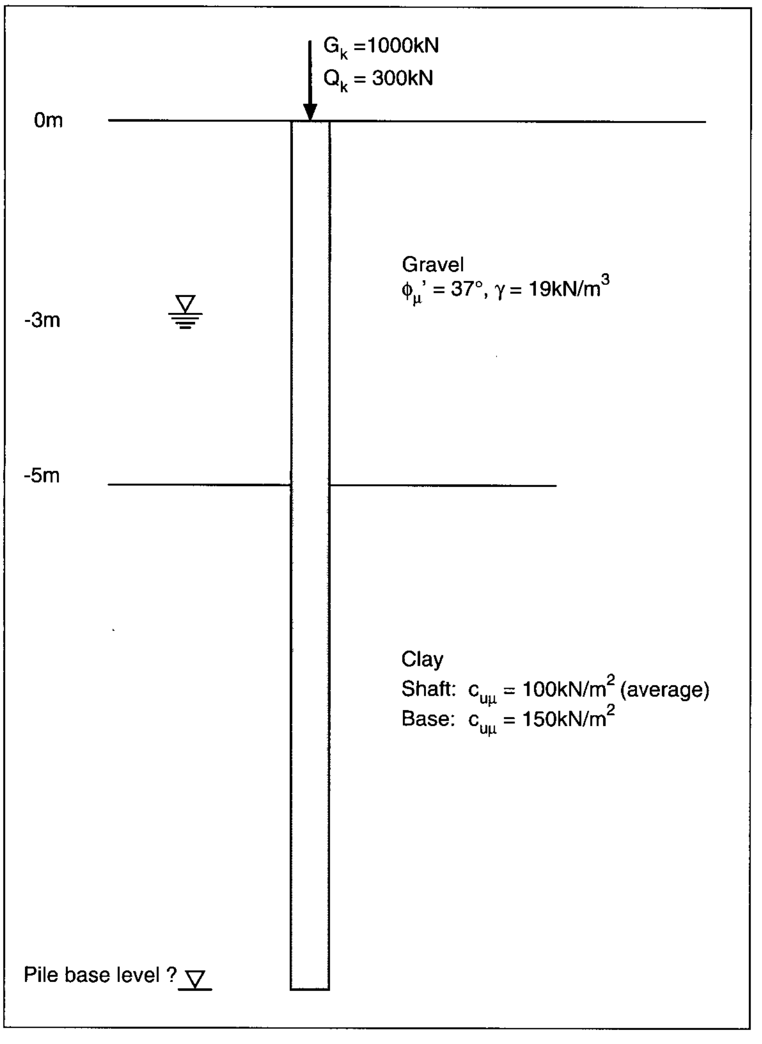


Figure E6.1 Compression pile with characteristic actions and mean ground properties

Shaft resistance in gravel:

$q_{si\mu} = Ks\ \sigma_v'\tan\delta_\mu$
$= 0.7\ \sigma_v'\tan\phi_\mu'$.

Shaft resistance in clay:

$q_{si\mu} = 0.6\ c_{u\mu}$.

Base resistance in clay:

$q_{b\mu} = 9\ c_{u\mu}$.

The calculations and notation follow 7.6.3.3.

E6.2 Calculations

Before calculating the required pile length, it is necessary to calculate the characteristic resistance of the pile elements (incremental lengths of shaft and base). This is done by using the mean pile information above and Paragraph 7.6.3.3(4)P, where the ratio between measured resistance q_m and characteristic resistance q_k is required to be at least 1.5 (ie $q_m / q_k \geq 1.5$). The derivation of the characteristic property from the mean property is analogous to Paragraph 7.6.3.2(6)P used for obtaining the characteristic bearing resistance from pile load tests.

Hence, the characteristic resistances are:

characteristic shaft resistance in gravel:

$q_{sik} = 0.7\ \sigma_v'\tan\phi_\mu' / 1.5$
$= 0.35\ \sigma_v'$

characteristic shaft resistance in clay:

$q_{sik} = 0.6\ c_{u\mu} / 1.5$
$= 0.4\ c_{u\mu}$

characteristic base resistance in clay:

$q_{bk} = 9\ c_{u\mu} / 1.5$
$= 6\ c_{u\mu}$.

These are also the design resistances for ULS Case B as per Table 2.1 where all partial factors on ground properties are equal to unity.

For the ULS design, Cases B and C (Table 2.1) must be checked. The combination of partial factors on design pile resistance and design actions are taken from EC7, Tables 2.1 and 7.2, as shown in Table E6.1.

Table E6.1 Combined use of partial factors from EC7, Tables 2.1 and 7.2.

	Actions (Table 2.1)			Pile Resistance (bored)	
	Permanent	Variable	Design load (V_d)	γ_b	γ_s
Case B	1.35	1.5	1800 kN	1.0 (Table 2.1)	1.0 (Table 2.1)
Case C	1.0	1.3	1390 kN	1.6 (Table 7.2)	1.3 (Table 7.2)

Hence, for derivation of the design resistance using Case C:
design shaft resistance in gravel:

$q_{sid} = q_{sik} / \gamma_s$
$= 0.35\,\sigma_v' / 1.3$
$= 0.27\,\sigma_v'$

design shaft resistance in clay:

$q_{sid} = q_{sik} / \gamma_s$
$= 0.4\,c_{u\mu} / 1.3$
$= 0.31\,c_{u\mu}$

design base resistance in clay:

$q_{bd} = q_{bk} / \gamma_b$
$= 6\,c_{u\mu} / 1.6$
$= 3.75\,c_{u\mu}$.

The required pile length for the two load cases can then be calculated using the following equations, with the appropriate design resistances:
design base resistance:

$R_{bd} = q_{bd} \cdot A_b$ (A_b = base area = πr^2)

design shaft resistance:

$R_{sd} = \Sigma\, q_{sik} \cdot A_{si}$ (A_{si} = shaft area for length 'i' of shaft = $\pi D.L_i$).

The calculations in Table E6.2 show that Case C is critical in this case for the sizing of the pile.

Table E6.2 Calculations for compression pile

	Case B	Case C
Length of shaft in gravel (m)	5	5
Average σ_v' in gravel (kN/m^2)	43.5	43.5
R_{sk} gravel (kN)	144	144
R_{sd} gravel (kN)	144	110
R_{bk} clay (kN)	254	254
R_{bd} clay (kN)	254	159
Design action V_d (kN)	1800	1390
Required R_{sd} clay (kN)	1800 – 144 – 254 = 1402	1390 – 110 – 159 = 1121
Required R_{sk} clay (kN)	1402 × 1.0 = 1402 – not critical	1121 × 1.3 = 1457 – critical
Hence L shaft clay (m)	Not critical	19.4
Pile base level (m Datum)	Not critical	– 24.4

The SLS design case (pile settlement) has not been considered here. This would be required in a complete pile design.

E7 CHARACTERISTIC CAPACITY OF COMPRESSION PILES DERIVED FROM LOAD TESTS

For a series of load tests, the characteristic capacity (ie ultimate resistance) is calculated using Paragraph 7.6.3.2.6(P). It depends on both the measured capacities of the trial piles and the number of piles tested. This example considers results from a test series using three test piles.

E7.1 Calculations

Firstly, suppose that the measured capacities of the three test piles are identical, and equal to 1500 kN. The characteristic capacity for a working pile, calculated after each trial pile result became available, is shown in Table E7.1. The results demonstrate the advantage gained from additional tests, where all pile tests gave the same resistance. The percentage increase in characteristic capacity (R_{ck}) obtained by carrying out the second trial pile was 11%. A further 4% increase in characteristic capacity was available after the third pile test. Carrying out more than three pile tests will not increase the characteristic value, assuming that the measured pile resistance remains constant from test to test.

Table E7.1 Characteristic capacities derived from 1, 2 or 3 load tests

Pile test series 1			
Pile test no	**1**	**2**	**3**
Measured capacity (R_{cm})	1500 kN	1500 kN	1500 kN
Factor ξ on mean R_{cm}	1.5	1.35	1.3
Factor ξ on minimum R_{cm}	1.5	1.25	1.1
Characteristic capacity (R_{ck})	1000 kN	1111 kN	1153 kN
Mean R_{cm} / R_{ck}	1.5	1.35	1.3
Min R_{cm} / R_{ck}	1.5	1.35	1.3

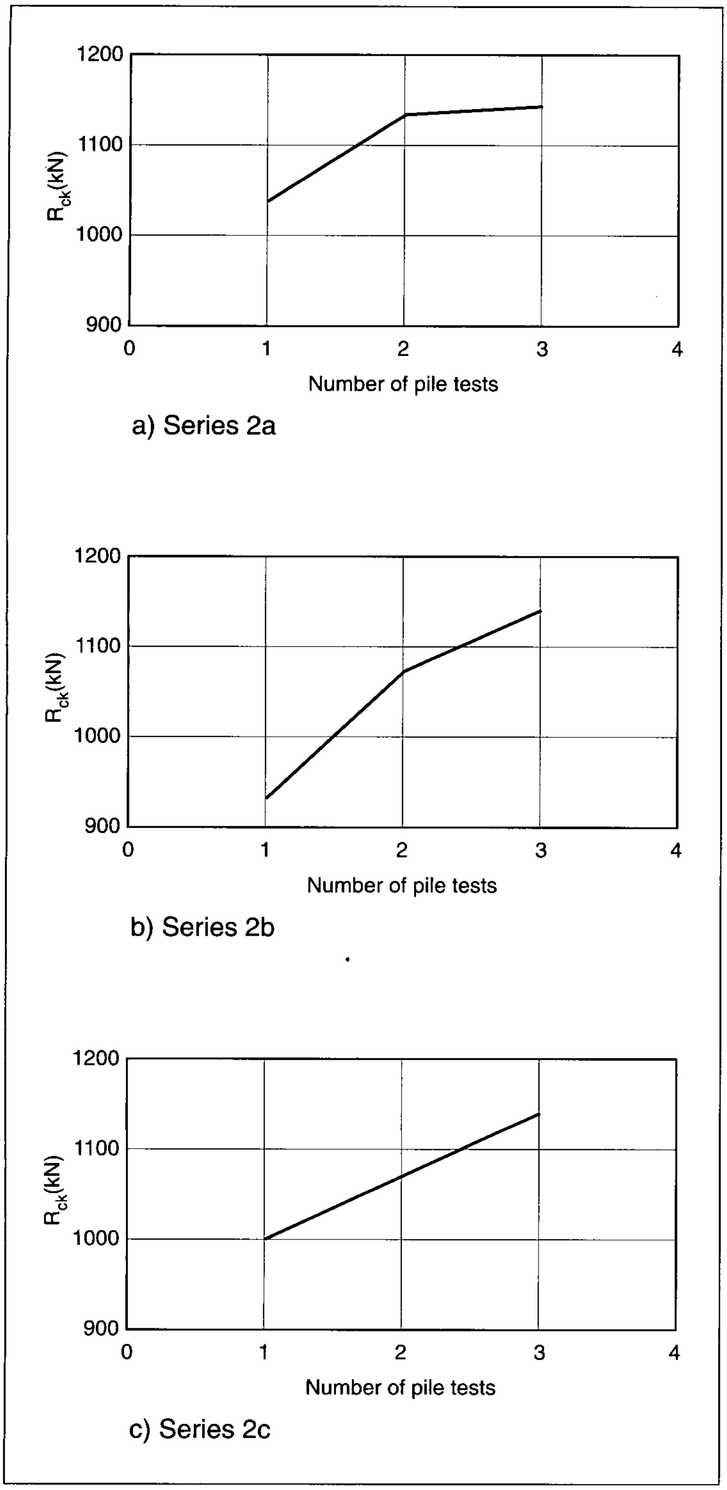


Figure E7.1 Pile test series

E7.2 Comparisons between sequences of pile test results

Suppose that, in a second series of pile tests, three piles were tested and the measured ultimate capacities were 1550 kN, 1500 kN and 1400 kN. The analysis of the pile tests results would depend in the sequence in which the results became available, as shown in Tables E7.2a, b & c, and Figures E7.1a, b & c.

Table E7.2a Pile test series 2a

Pile test no	**1**	**2**	**3**
R_{cm}	1550 kN	1500 kN	1400 kN
Mean R_{cm}	1550 kN	1525 kN	1483 kN
ξ on mean R_{cm}	1.5	1.35	1.3
ξ on min R_{cm}	1.5	1.25	1.1
R_{ck}	1033 kN	1130 kN	1141 kN
Critical	Mean + Min	Mean	Mean

Table E7.2b Pile test series 2b

Pile test no	**1**	**2**	**3**
R_{cm}	1400 kN	1500 kN	1550 kN
ξ on mean R_{cm}	1.5	1.35	1.3
ξ on min R_{cm}	1.5	1.25	1.1
R_{ck}	933 kN	1074 kN	1141 kN
Critical	Mean + Min	Mean	Mean

Table E7.2c Pile test series 2c

Pile test no	**1**	**2**	**3**
R_{cm}	1500 kN	1400 kN	1550 kN
ξ on mean R_{cm}	1.5	1.35	1.3
ξ on min R_{cm}	1.5	1.25	1.1
R_{ck}	1000 kN	1074 kN	1141 kN
Critical	Mean + Min	Mean	Mean

The tables and graphs show that when there is a reasonable variation between the capacity measured in the three tests (10% variation) the characteristic capacity always increases, even if the strongest pile is tested first and the weakest pile tested last (Series 2a). Hence, where a large number of piles are to be constructed there may be significant savings available if more than one pile trial test is carried out.

If the variation in ultimate resistance between piles is large (more than 20%) then it is possible that the characteristic resistance will decrease as the number of pile tests increases (assuming a strong pile was tested first). In this case it is necessary that the designer investigates the wide variation in pile capacities prior to proceeding to construction. When this occurs the minimum measured capacity (rather than the mean) will control the characteristic capacity.

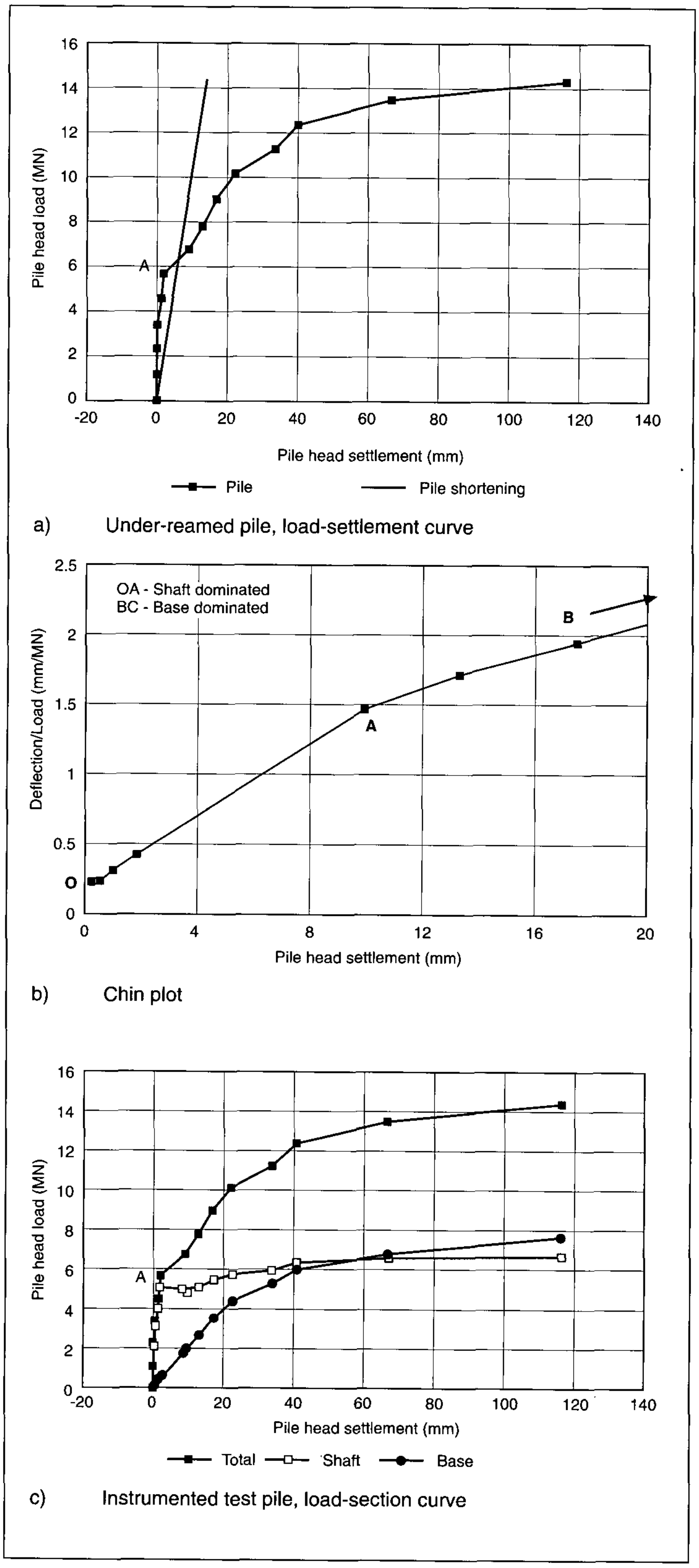


Figure E8.1 Design of a compression pile, assessing shaft and base capacity from loading test

E8 DERIVATION OF BASE AND SHAFT COMPONENTS FROM A COMPRESSION PILE LOAD TEST

E8.1 Introduction and data

The ULS design pile load capacity is obtained from a single pile load test using three methods of interpreting the pile load tests to failure.

Method A considers the maximum pile shaft capacity by inspection;

Method B uses Chin (1972) to derive both ultimate shaft and base capacities; and

Method C uses the results of instrumentation which distinguishes between components of shaft and base resistance.

The pile has a 1.02 m diameter shaft and a 2.06 m diameter base. The total length of the pile is 22 m.

E8.2 Method A calculations

Figure E8.1a shows the load-settlement points measured in the pile test, for which the ultimate resistance was 14.2 MN at a settlement/shaft diameter ratio of $\delta / D = 10\%$. The pile shortening line is also shown on Figure E8.1a. This line is the elastic compression of the pile assuming that the total imposed load results in compression of the whole length of the pile shaft. The intersection of this pile shortening line with the pile load-deflection curve is at approximately 6.2 MN ($\delta = 5$ mm) and it is this load that may be assumed to approximately equal to the shaft friction of the pile. By inspection, it is therefore considered that the shaft resistance is about 6.2 MN, so the base resistance is inferred to be 8.0 MN.

E8.3 Method B calculations

Figure E8.1b shows the same data on a 'Chin plot'. From this it could be inferred that the shaft capacity is about 8.3 MN and the total ultimate capacity (at very large displacement) 16 MN, giving a base capacity of 7.7 MN.

E8.4 Method C calculations

During the loading cycle, the test pile was instrumented at locations along the shaft and base. The load cells showed a shaft resistance of 6.7 MN and a base resistance of 7.5 MN at $\delta / D = 10\%$. The results for the shaft and base resistances are shown in Figure E8.1c.

In order to calculate the design resistance of the pile from the measured resistance in a pile load test, Paragraphs 7.6.3.2(6)P and 7.6.3.2(10)P must be used.

Paragraph 7.6.3.2(6)P considers the characteristic resistance of the pile based on both the load test results and the number of load tests carried out (see E7):

$$R_{ck} = R_{cm} / \xi$$

where R_{ck} = the characteristic, k, resistance of the compression, c, pile test(s)
R_{cm} = the measured, m, resistance from the compression pile tests (only one test was carried out here)
ξ = a factor depending on the number of pile load tests and, where there is more than one test, whether the minimum R_{cm} or average R_{cm} is being considered (Table 7.1).

Paragraph 7.6.3.2(10) allows the design resistance to be derived from the characteristic resistance:

$R_{cd} = R_{ck} / \gamma$.

The partial factors (γ) are the equivalent of the ULS Case C factors presented in Paragraph 2.4.2(14)P for use with ground properties. Values of γ are provided for total pile capacity, shaft capacity and base capacity (Table 7.2). The value of γ is different for different types of pile installation (conventional bored, continuous flight auger or driven). This example uses bored piles.

The design capacity of the pile is now calculated for Method C.

For a single pile load test the value of ξ is 1.5 (R_{cm} for average and minimum conditions are obviously the same). The measured pile resistance was 14.2 MN, hence the characteristic resistances are:

$R_{ck} = R_{cm} / \xi$
$R_{ck} = 14.2 / 1.5$
$= 9.47$ MN total resistance
$R_{sk} = 6.7 / 1.5$
$= 4.47$ MN (subscript 's' for shaft)
$R_{bk} = 7.5 / 1.5$
$= 5.0$ MN (subscript 'b' for base).

The designer may then chose between two approaches:

either $R_{cd} = R_{ck} / \gamma_t$
$= 9.47 / 1.5$
$= 6.31$ MN ($\gamma_t = 1.5$ on total resistance)
or $R_{sd} = R_{sd} + R_{bd}$
$= 3.44 + 3.13$
$= 6.57$ MN on combined shaft and base
where $R_{sd} = 4.47 / 1.3$
$= 3.44$ MN ($\gamma_s = 1.3$ on shaft)
$+ R_{bd} = 5.0 / 1.6$
$= 3.13$ MN ($\gamma_b = 1.6$ on base).

In this instance the capacity derived by resolving the total resistance into shaft and base components is some 4% larger than the resistance of the pile when no distinction is made between shaft and base, providing a more economic design which is in compliance with the code.

E8.5 Method comparisons

The results for the three Methods are summarised in Table E8.1. In the table, the displacement that is appropriate to the total resistance in each Method is stated. It is clear that the displacements for Methods A and C are the same (the measured resistance) while Method B is for a larger displacement. In order to calculate the design pile resistance, the shaft and base resistance is expressed as a fraction of the total resistance. This breakdown between shaft and base resistance is then used with the measured ultimate compression resistance to obtain the design resistance of the pile.

Table E8.1 Method comparison

	Method A: By inspection	**Method B: Chin method**	**Method C: By measurement**
Shaft resistance (MN)	6.2	8.3	6.7
Base resistance (MN)	8.0	7.7	7.5
Total resistance (MN)	14.2 at $\delta = 10\%$ D	16.0 δ = large	14.2 at $\delta = 10\%$ D
Ratio shaft/total resistance	0.437	0.519	0.472
Ratio base/total resistance	0.563	0.481	0.528
Measured ultimate compression resistance, R_{cm} (MN)	14.2	14.2	14.2
Characteristic ultimate compression resistance, R_{ck} (MN) $R_{ck} = R_{cm} / 1.5$ (Table 7.1)	9.47	9.47	9.47
Characteristic shaft resistance, R_{sk} (MN)	$9.47 \times 0.437 = 4.14$	$9.47 \times 0.519 = 4.91$	$9.47 \times 0.472 = 4.47$
Characteristic base resistance, R_{bk} (MN)	$9.47 \times 0.563 = 5.33$	$9.47 \times 0.481 = 4.56$	$9.47 \times 0.528 = 5.00$
Case C design shaft resistance, R_{sd} (MN)	$4.14 / 1.3 = 3.18$	$4.91 / 1.3 = 3.78$	$4.47 / 1.3 = 3.43$
Case C design base resistance, R_{bd} (MN)	$5.33 / 1.6 = 3.33$	$4.56 / 1.6 = 2.85$	$5.00 / 1.6 = 3.13$
Case C design total resistance, R_{cd} (MN)	6.51	6.63	6.56

In this example, the division between shaft and base resistance was varied between the three Methods. Nevertheless, the resulting design total resistances for ULS Case C differed by less than 2%, due partially to the even distribution between base and shaft resistances.

It is important to note that although there is some uncertainty in the assessment of the measured **ratio** of shaft to base resistance, the calculation uses the measured **total** resistance, which is clearly determined in the test to be 14.2 MN.

E9 INPUT TO STRUCTURAL DESIGN OF A PILE IN HEAVING GROUND

E9.1 Purpose

This example is used to illustrate the value of considering ground-structure interaction in order to reduce design forces in a structure.

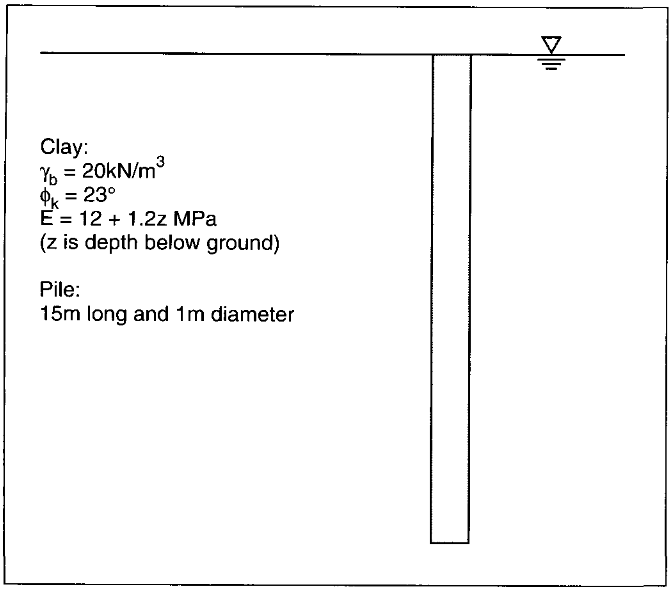


Figure E9.1 Pile in heaving ground

E9.2 Data

EC7, 7.3.2.3 considers the reaction of a pile when the ground that supports it swells. It states that the movement of the ground shall generally be treated as an action. (This is an 'indirect action' as defined in EC1, 4.1(1)). The example given below considers the ULS state for structural design of the pile to withstand ground heave prior to placement of permanent vertical load on the pile. The pile and ground conditions are summarised in Figure E9.1. No consideration of the allowable displacement or calculation of crack width in the pile section is presented.

In this example Case B is critical. The partial factor for Case B is 1.35 and this can be applied to the force calculated using characteristic ground strength. The resulting design force (tension) will be greater than that calculated for Case C where there is a factor of 1.25 applied to $\tan\phi'$. (Note that in this case $\tan\phi'_d = \tan\phi'_k / \gamma_\phi > \tan\phi_k'$ as strong ground is detrimental to the pile integrity. EC7 does not give a value for the partial factor for this case, but $\gamma_\phi = 1 / 1.25 = 0.8$ is assumed here.)

Strictly, it might be argued that if ground displacement is treated as the action, the factor of 1.35 should be applied to displacement, and not to the resulting forces, which are action effects. In this case, it could be found that the design action effects from Case B would be smaller, and Case C becomes critical. In the approach shown here, the partial load factor is applied to the action effect. Further discussion of this question is presented in E10.

Two calculations are presented for the maximum mobilised tensile force in the pile.

a The first calculation considers the maximum forces that could be mobilised in the pile if the pile were considered to be infinitely stiff. This calculation provides an upper limit to the design tensile force in the pile.

b The second calculation incorporates the stiffness of the pile and assumes that the pile cracks. The stiffness of the pile is calculated using only the reinforcement in the pile.

In both cases, allowance is made for the weight of the pile.

E9.3 Method 1

In this upper limit calculation of the ultimate pile tensile force the pile skin friction is assumed to be fully mobilised along the pile shaft. This is conservative, as mobilised skin friction is known to be related to the shear displacement between pile and surrounding ground. The level at which the cumulative skin friction, starting from the pile base, equals the cumulative skin friction, starting at the pile head, is called the balance point and this is the point of maximum tensile stress in the pile. Typically, the balance point will be at about two thirds the depth of the pile shaft. The calculated stress distribution, including the self-weight of the pile, is shown in Figure E9.2. The maximum tensile stress due to heaving ground around the pile is 1411 kPa, equivalent to a tensile force of 1108 kN in the pile.

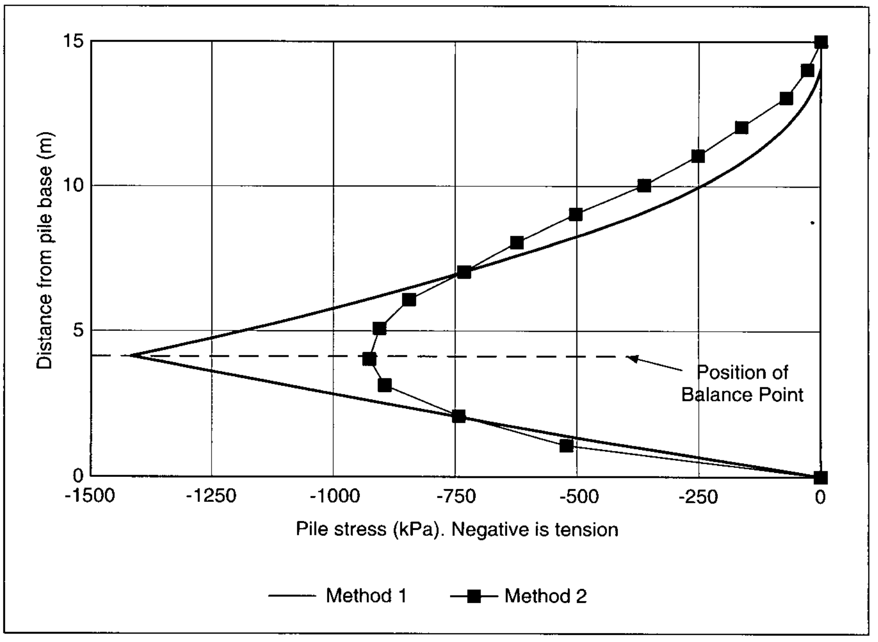


Figure E9.2 Calculated ULS design tensile force

E9.4 Method 2

In this calculation, both the stiffness of the ground and the stiffness of the pile are considered (in contrast to Method 1). This is most easily carried out using a finite element program (or similar) which will iterate to a solution with compatible stresses and strains. The cumulative stress in the pile shaft derived is shown in Figure E9.2.

The maximum tensile stress in the pile shaft is 922 kPa, which is 35% less than the value calculated in Method 1. The stress of 922 kPa is equivalent to a force of characteristic tensile force (V_k) of 724 kN, which should be used for design of the pile reinforcement.

The required area of tensile steel can then be calculated using this characteristic force and criteria in EC2.

The design tensile force is:

$$V_d = V_k \times \gamma_G$$
$$= 724 \times 1.35$$
$$= 978 \text{ kN.}$$

The characteristic yield strength (f_{yk}) of the steel used is 460 N/mm^2 and the design yield strength is:

$$f_{yd} = f_{yk} / \gamma_m$$
$$= 460 / 1.15$$
$$= 400 \text{ N/mm}^2.$$

Hence the required area of steel (A_s) is:

$$A_s = V_d / f_{yd}$$
$$= 978 / 0.4$$
$$= 2445 \text{ mm}^2.$$

The stiffness of the pile has been calculated assuming that the longitudinal reinforcement in the pile is 8T20 bars (area 2513 mm^2), which amounts to 0.32% reinforcement of the pile section. It is assumed that the pile concrete is cracked and hence only the steel provides the longitudinal stiffness of the pile. (This is considered appropriate for a ULS calculation. In the working condition the concrete would probably add to the stiffness of the pile.)

The above example demonstrates the need (in this case, benefit) to consider the stiffness and flexibility of the structure and the ground when calculating forces for structural design.

E10 PILE SUBJECT TO DOWNDRAG

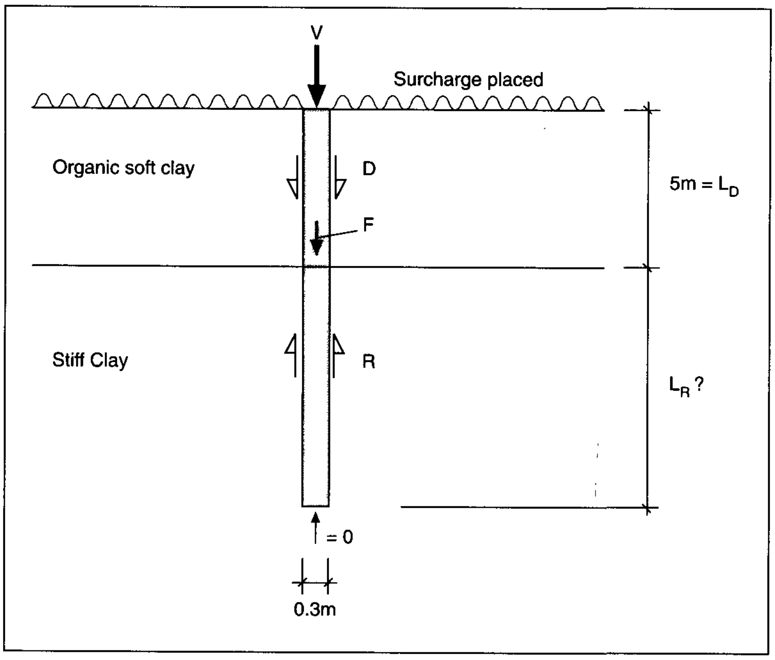


Figure E10.1 Pile subject to downdrag

E10.1 Data

Figure E10.1 shows a 300 mm diameter concrete pile driven or bored through 5 m of soft organic clay into a stiffer bearing stratum. At the time the pile comes into use, a surcharge is to be placed at ground level which is sufficient to mobilise limiting negative skin friction between the pile and the soft clay. Assessment of the interaction between the shaft and the soft clay should take account of the relative movement of the pile and soil, and this may lead to the adoption of values for the negative skin friction which vary with depth. However, for simplicity it will be assumed that the characteristic negative skin friction is 20 kPa, constant with depth. Similarly, it will be assumed that the characteristic shaft resistance in the bearing stratum is 50 kPa, constant with depth. Base resistance is negligible.

The pile is to support a vertical characteristic permanent action of 300 kN. The necessary penetration L_R into the bearing stratum is to be calculated, together with the maximum compression force in the pile for structural design, F_d.

E10.2 Approach

Checks are required for Cases B and C. This requires some interpretation of EC7 Section 7 which was written for Case C alone. The concepts embodied in Table 2.1 will be followed, applying Case B directly and replacing the factors on soil strength in Case C by factors from Table 7.2 (EC7, 2.4.3(12)P).

EC7, 7.3.2.1 allows the designer to adopt either the ground displacement or the downdrag forces on the pile as the action (see C7.3.2.1(2)). The designer must show that the pile conforms to EC7 by one of these methods, but he is free to adopt whichever of these gives the more economic design. Treating settlement as an action is a more complicated approach, requiring consideration of the ground-pile interaction. However, it could often be used to show that the downdrag forces are less than the maximum determined by the limiting shaft adhesion, but this possibility is not addressed in this simple example.

E10.3 Characteristic values of forces

Characteristic applied load, $= V_k$
$= 300$ kN.

Characteristic downdrag force, $= D_k$
$= \pi D L_D q_{Dk}$
$= \pi \times 0.3 \times 5 \times 20$
$= 94.2$ kN.

Characteristic shaft resistance, $= R_k$
$= \pi \times 0.3 \times 50 \times L_R$
$= 47.1 \times L_R$ kN.

E10.4 Case C1 – downdrag force (D) taken as action

Partial factors for actions:

Vertical load, V: $\gamma_G = 1.0$ (Table 2.1)
Downdrag, D: $\gamma_G = 1.0$ (Table 2.1).

Downdrag is classified as a 'permanent' action because its *variation is always in the same direction (monotonic) until the action attains a certain limit value* (EC1, 1.5.3.3).

Partial factors for resistances:

Shaft resistance, R: $\gamma_s = 1.3$ (Table 7.2).

Total design vertical load $= F_d = V_d + D_d$

$= V_k \times \gamma_G + D_k \times \gamma_G$

$= 300 \times 1.0 + 94.2 \times 1.0$

$= 394.2$ kN.

Design shaft resistance $= R_d$

$= R_k / \gamma_s$

$= 47.1 \times L_R / 1.3.$

Require $R_d \geq F_d$.

Hence $47.1 \times L_R / 1.3 \geq 394.2$ kN.

So $L_R \geq 10.88$ m.

Design force for concrete shaft $= F_d = 394.2$ kN.

E10.5 Case C2 – settlement taken as action

Partial factors for actions:

Vertical load, V: $\gamma_G = 1.0$ (Table 2.1)

Settlement: $\gamma_G = 1.0$ (Table 2.1).

Partial factors for resistances:

Shaft resistance, R: $\gamma_s = 1.3$ (Table 7.2).

When downdrag force was taken as an action, in Case C1, a factor of 1.0 was applied to it. However, when settlement is taken to be the action, its effect is transferred to the pile using the soil strength, which therefore acts in an unfavourable manner. EC7, 2.4.2(11) says that a partial factor less than 1.0 must be applied in such cases, but a more precise value is not given. It could be taken to be $1 / \gamma_s$ from Table 7.2 or $1 / \gamma_{cu}$ from Table 2.1. This would give $1 / 1.3 = 0.769$ or $1 / 1.4 = 0.714$, respectively. A compromise value of $\gamma_D = 0.75$ will be used here.

As noted above, a careful analysis of ground-pile interaction could be applied to find the downdrag force, but this has not been done in this simple example. Instead, the assumption that the displacement would mobilise all the available shaft adhesion in the soft clay is adopted. The characteristic value of this is 94.2 kN, as shown above.

Then $D_d = 94.2 / \gamma_D$

$= 94.2 / 0.75$

$= 125.7$ kN.

Total design vertical load $= F_d = V_d + D_d$

$= Vk \times \gamma_G + D_k \times \gamma_G$

$= 300 \times 1.0 + 125.7$

$= 425.7$ kN.

Design shaft resistance $= R_d = R_k / \gamma_s$

$= 47.1 \times L_R / 1.3.$

Require $R_d \geq F_d$.

Hence $47.1 \times L_R / 1.3 \geq 425.7$ kN.

So $L_R \geq 11.75$ m.

Design force for concrete shaft $= F_d = 425.7$ kN.

E10.6 Case B1 – downdrag force taken as action

Partial factors for actions:

Vertical load, V: $\gamma_G = 1.35$ (Table 2.1)

Downdrag, D: $\gamma_G = 1.35$ (Table 2.1).

Partial factors for resistances:

Shaft resistance, R: $\gamma_s = 1.0$ (Table 7.2).

Total design vertical load $= F_d = V_d + D_d$

$= V_k \times \gamma_G + D_k \times \gamma_G$

$= 300 \times 1.35 + 94.2 \times 1.35$

$= 532.2$ kN.

Design shaft resistance $= R_d = R_k / \gamma_s$

$= 47.1 \times L_R / 1.0.$

Require $R_d \geq F_d$.

Hence $47.1 \times L_R / 1.0 \geq 532.2$ kN.

So $L_R \geq 11.29$ m.

Design force for concrete shaft $= F_d = 532.2$ kN.

E10.7 Case B2 – settlement taken as action

Partial factors for actions:

Vertical load, V: $\gamma_G = 1.35$ (Table 2.1).

Any partial factor applied to settlement would have no effect in this case.

Partial factors for resistances:

Shaft resistance, R: $\gamma_s = 1.0$ (Table 7.2).

Partial factor for unfavourable soil strength transmitting effect of settlement to pile = 1.0.

Hence design downdrag force = 94.2 kN.

Total design vertical load $= F_d = V_d + D_d$

$= V_k \times \gamma_G + 94.2$

$= 300 \times 1.35 + 94.2$

$= 499.2$ kN.

Design shaft resistance $= R_d = R_k / \gamma_s$

$= 47.1 \times L_R / 1.0.$

Require $R_d \geq F_d$.

Hence $47.1 \times L_R / 1.0 \geq 499.2$ kN.

So $L_R \geq 10.6$ m.

Design force for concrete shaft $= F_d = 499.2$ kN.

Table E10.1 Summary of results

Calculation	L_R (m)	F_d (kN)
C1 – downdrag force taken as action	10.88	394.2
C2 – settlement taken as action	11.75	425.7
B1 – downdrag force taken as action	11.29	532.2
B2 – settlement taken as action	10.60	499.2

E10.8 Conclusion and discussion

The results of the calculations are summarised in Table E10.1.

It is necessary to satisfy both cases B and C, but the choice of force or displacement as the action is open to the designer. A pile with a penetration into the bearing stratum of $L_R = 10.88$ m and structural capacity of $F_d = 499.2$ kN would therefore comply with the code.

This combination of L_R and F_d is derived from calculations C1 and B2, which are not consistent in their use of downdrag force or settlement as the action. However, both calculations C1 and C2, for example, are sufficient but not necessary, and similarly B1 and B2. Hence a design which conforms with C1 and B2 is acceptable.

The Case B2 calculation has a factor of 1.0, in effect, on the negative skin friction effect. The code may be open to criticism at this point.

The results for Cases B1 and C1 illustrate a situation which will frequently arise for shaft controlled piles subject to permanent loads, for the present boxed values in Table 7.2. Because the factor on shaft resistance [1.3] is less than the load factor for permanent loads [1.35], Case B requires a longer pile than Case C. The problem does not occur when variable loads dominate because of the additional factor of 1.3 in Table 2.1 for variable loads in Case C, compared with 1.5 in Case B ($\gamma_q \times \gamma_s = 1.3 \times 1.3$ for Case C $> 1.5 \times 1.0$ for Case B). This inconvenience could be removed from EC7 by small adjustments to the partial factors and ξ values.

E13 DESIGN OF A CONCRETE STEM WALL

E13.1 Data

Figure E13.1 shows a concrete stem wall supporting sloping ground. The characteristic values of the soil parameters are shown in the figure, together with some fixed dimensions. The design is to find the required length of the heel of the wall, and hence the footing width, B, together with the bending moments and shear forces in the wall for structural design. Both ultimate and serviceability limit states must be considered.

E13.2 Ultimate limit states

For ultimate limit state design, the steps of the calculation are shown in Table E13.1. Both Cases B and C must be considered. Initially, it is assumed that active earth pressures may be used for the design, and the implications of this are reviewed later.

Table E13.1 Calculations for concrete stem wall

		Case C1	Case C2	Case B1	Case B2
Characteristic ϕ' (°)		32.5	32.5	32.5	32.5
Factor on $\tan\phi'$		1.25	1.25	1.0	1.0
Design ϕ' (°)		27.0	27.0	32.5	32.5
γ_{soil} (kN/m^3)		19	19	19	19
γ_{cct} (kN/m^3)		25	25	25	25
Characteristic surcharge p (kPa)		5	5	5	5
Factor on surcharge		1.3	1.3	1.1[b]	1.1[b]
Design surcharge p (kPa)		6.5[a]	6.5[a]	5.5[a]	5.5[a]
δ/ϕ'	active		$^2/_3$		$^2/_3$
	passive	$^2/_3$	$^2/_3$	$^2/_3$	$^2/_3$
	base	1	1	1	1
Design δ°	active	20 (= β)[c]	18	20 (= β)[c]	21.3
	passive	18	18	21.3	21.3
	base	27	27	32.5	32.5
Characteristic K_a				0.35	0.35
Factor on K_a				1.35	1.35
Design K_a[d]		0.48	0.49	0.47	0.47
Design K_p[d]		4.1	4.1	6.0	6.0
Design N_q[e]		13.2		24.6	
Design N_c[e]		24.0		37.0	
Design N_γ[e]		12.4		30.1	
B (m)		5.3		~ 4.3	
$\tan^{-1}$ (H / V) for base (°)[f]		22.7		22.2	
		< 27 √		< 32.5 √	
Max bending moment in wall (kN m/m)			396		372
Max bending moment in toe (kN m/m)			79		78
Max bending moment in heel (kN m/m)			365		336

[a] The partial factor on surcharge, taken to be a variable load, is zero when it is beneficial (ie beneficial surcharge is ignored). Cases C1 and B1 consider the overall stability of the wall, so no surcharge is assumed between the wall and the virtual back. Cases C2 and B2 consider the bending moments in the wall, for which surcharge between the wall and virtual back is adverse and so is included

[b] Because the effect of the surcharge will later be factored by 1.35, the input value of this variable action is here factored by 1.5 / 1.35 = 1.1

[c] On the virtual back, δ is set equal to the slope angle β; this is consistent with 8.5.2(2), though this paragraph refers to at rest states. Elsewhere, δ is limited to $^2/_3\,\phi'_d$, in accordance with 8.5.1(4) for 'precast' concrete (which really means concrete not cast directly against the soil). The use of $\delta = \beta$ is noted by Clayton et al (1993, p163)

[d] Values taken from Figures G2 and G3 in Annex G

[e] Values calculated using Annex B

[f] H and V are the horizontal and vertical components of the force between the base and the ground

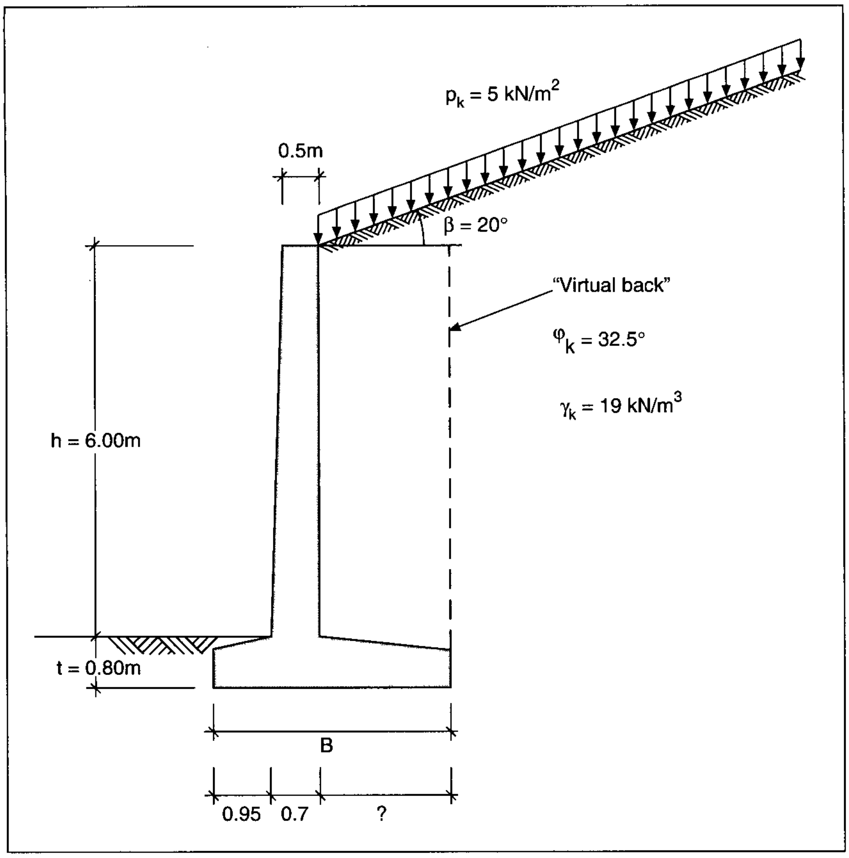


Figure E13.1 Concrete stem wall supporting sloping ground

The surcharge is a variable action, which may occur, or be absent, randomly at any point. It is therefore necessary to consider two load cases, with and without a surcharge in the area between the wall and the 'virtual back'. In checking the overall equilibrium of the system, the surcharge in this area would be favourable (Cases C1 and B1), and so has an applied partial factor of 0.0, ie it is omitted. However, in checking the strength of the wall, the surcharge is unfavourable, and so is included (Cases C2 and B2). Surcharge beyond the virtual back is always unfavourable and so is always included.

It is reasonable to expect that Case C will govern the dimensioning of the heel so this case is checked first, giving a base width of 5.3 m. During the calculation for Case B, a rough calculation is performed to check that the dimension derived for Case C is adequate; a base width of about 4.3 m would be sufficient for Case B.

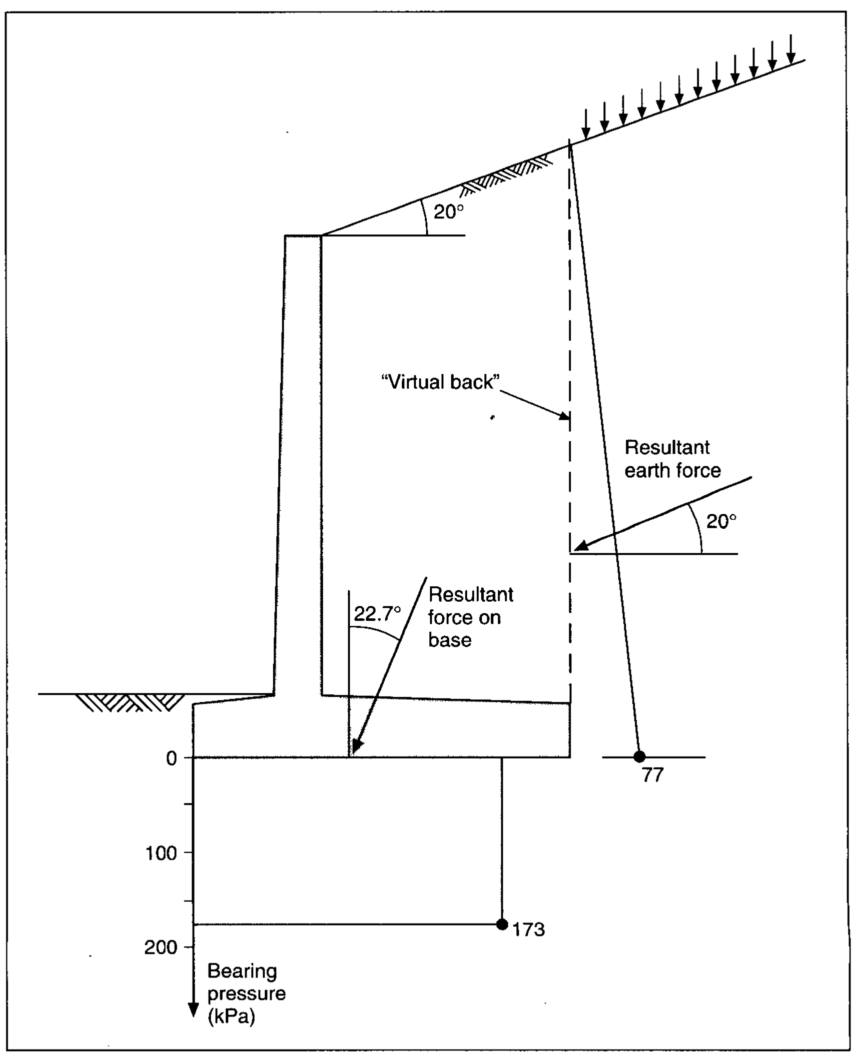


Figure 13.2 Results for Case C 1

Each column in the table represents a continuous calculation. Partial factors are applied to the soil strength (tanϕ' in this case) and to the surcharge load p. Values of the coefficients of earth pressure, K, and of the bearing capacity factors, N, are then derived directly from EC7 Annexes G and B, using the design values of ϕ'.

The base width B is required to provide equilibrium between the lateral earth pressures and available bearing resistance. This may be achieved by hand calculation or using suitable software; iteration may be required to find the minimum base width which provides equilibrium. In this case, the *Oasys* program GRETA was used for the wall equilibrium calculations. The shear on the base, represented by the ratio of forces H / V must be checked to be less than the available base friction, δ.

The earth pressures for the critical Case C1 are shown in Figure E13.2. On the passive side of the wall, the ground level shown in Figure E13.1 should be the worst that is foreseeable, allowing for excavation of service trenches, etc. Even then, the upper 0.5 m of passive soil is neglected (EC7, 8.3.2.1(2)); although the remaining passive resistance is included, it plays a minor role in the calculation.

The calculations for Case B are applied to the same wall geometry as Case C, ie that which will finally be built. The method of applying partial factors in Case B to designs of this type is slightly uncertain. The approach adopted here is to increase the coefficient of active earth pressure by a factor of 1.35. This is considered to be consistent with 2.4.2(17), at least in spirit. Because a larger factor of 1.5 should be applied to the surcharge (a variable action), the surcharge is multiplied at source by 1.1 (= 1.5 / 1.35), in the knowledge that the earth pressures it causes will be factored by 1.35. The vertical earth pressures and passive earth pressure have not been increased by 1.35. Such a factor seems unreasonable; although it would affect parts of the Case B calculation, it would have very little effect on the final design.

Bending moments from Case C2 are shown in Figure E13.3. Following 6.8(2), the bearing pressure is assumed to be distributed linearly beneath the base for calculation of bending moments in the base. For this problem, there is little difference between the results of the two cases, Case C proving to be critical. These bending moments may be taken directly into Eurocode 2 as ultimate limit state design values; no further factors are applied to them. More typically, for level ground supported behind the wall, Case B normally gives the more severe structural action effects, especially if water pressure is present.

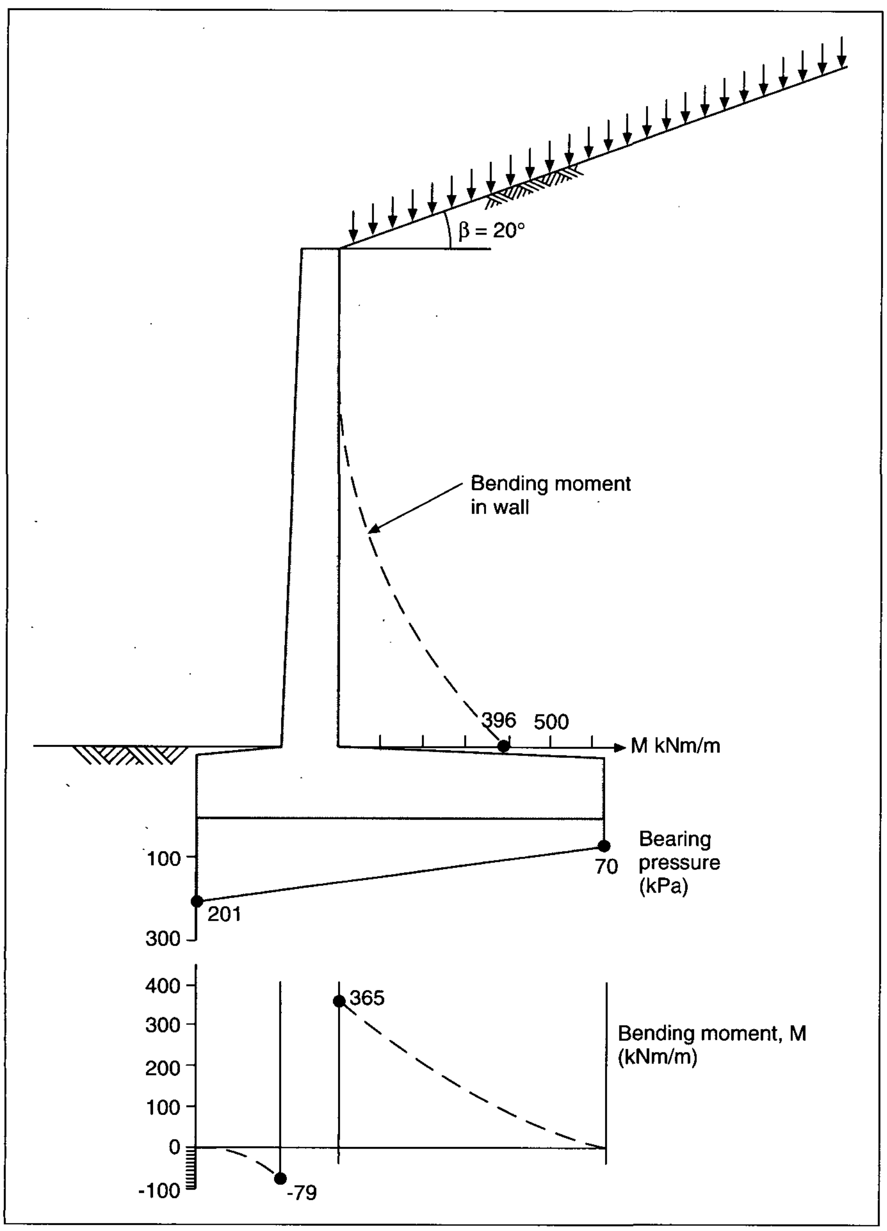


Figure E13.3 Results for Case C2

E13.3 Serviceability limit states

Serviceability considerations apply to the displacement of the structure and surrounding ground, and to the performance of the concrete, especially with regard to cracking.

EC7, 8.7.2 notes that it is often possible to avoid detailed analysis of displacement by noting comparable examples. It would not normally be necessary to calculate the displacement of a wall of this type on a sand foundation. If it were, the methods of calculating settlements of footings, noted in E5, could be adopted. For economy of effort, the loads on the footing could be taken from the ULS Case B calculation, but if these lead to a marginal situation a more accurate calculation of service loads should be undertaken.

EC7, 8.5.1(6) notes that the earth pressures for ultimate and serviceability limit states are, in principle, derived from different calculations. EC7, 8.5.4 considers the relationship of earth pressure to movement, and 8.5.5 considers compaction effects. In considering 'Structural serviceability limit states', subclause 8.7.4 notes that all these factors are relevant and that design earth pressures for serviceability limit states will not necessarily be limiting values. EC7 is unable to be more specific because earth pressures existing in the service state are very dependent on individual circumstances.

A minimum earth pressure sometimes used for the serviceability check of this type of wall may be obtained using a coefficient of earth pressure, K_{SLS}, given by:

$K_{SLS} = {}^1\!/_2(K_a + K_{nc})$

where K_a is the coefficient of active pressure and $K_{nc} = (1 - \sin\phi')$. For sloping ground, K_{nc} would logically be replaced by $K_{nc}\,(1 + \sin\beta)$, as in 8.5.2(2). Where it is known that heavy compaction will be used, a larger value should be adopted, however. (Currently, for design of backfilled retaining walls and bridge abutments on the trunk road network in the United Kingdom, BD30/87 has much more severe requirements – see Carder (1998).)

For this calculation, characteristic values of ϕ' should be used, so:

$K_{SLS} = {}^1\!/_2(K_a + K_{nc,k})$.

Typically, if $\delta = {}^2\!/_3\,\phi'$ is used in deriving K_a, this gives a value for K_{SLS} roughly equal to the value used for ULS design, which cannot be less than 1.35 K_{ak}, used in Case B. So it is likely that serviceability will dominate the structural design. This situation may be compared with that of BS 8002, which proposes that both SLS and ULS design of the structural sections should be based on the same earth pressures. In this problem, they would be equivalent to about 1.25 K_{ak}, slightly smaller than the value of K_{SLS} suggested here. Using either code, structural designers may prefer to carry out the ULS design for a larger bending moment in order to give simple SLS calculations, as discussed in C8.6.6.

E14 DESIGN OF A CANTILEVER SHEET PILE WALL

E14.1 Data and method

Figure E14.1 shows Example B2 taken from CIRIA Report 104. It requires the design of a cantilever sheet pile wall in a single soil, with differential water pressure across the wall and no surcharge behind the wall. It is considered that the 'moderately conservative' values of CIRIA 104 are equivalent to characteristic values for the soil parameters. Only the permanent works design will be considered here. EC7 does not provide specific requirements for temporary works; this point is considered in B6 and C2.4.2(14)P.

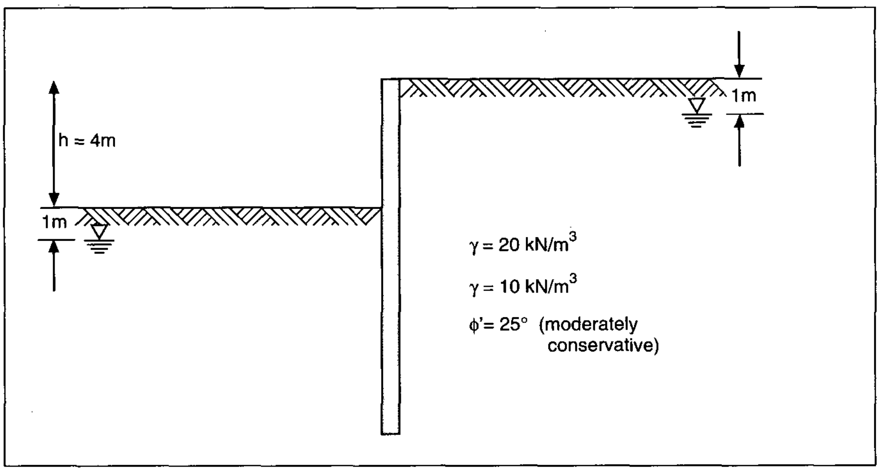


Figure E14.1 Cantilever wall example (CIRIA Report 104, Example B2)

EC7, 8.3.2.1 requires that an 'overdig' allowance shall be made for walls which rely on passive resistance, without distinction between temporary and permanent situations. This is discussed in C8.3.2.1. The overdig allowance will be applied in this example.

Because the wall is unpropped, the earth pressure distribution relevant to ultimate limit states in both the ground and the structure is of the simple form shown in Figure E14.2, which shows the design conditions and earth pressures calculated for EC7 Case C. Hand calculations or a computer program may be used to find the wall length which gives equilibrium.

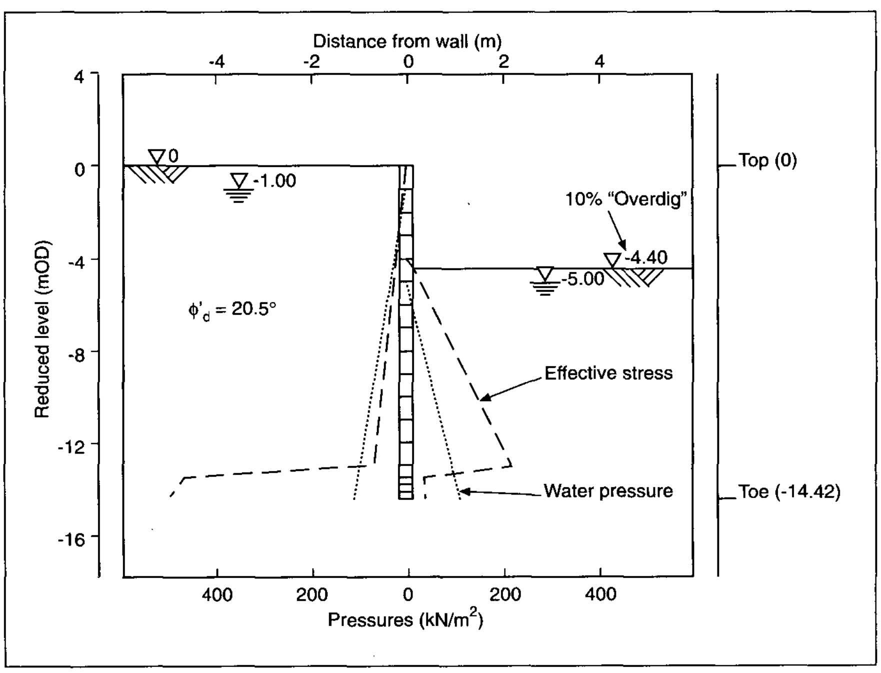


Figure E14.2 Cantilever sheet pile wall: Case C conditions and earth pressures

E14.2 Comparative calculations

In CIRIA Report 104, the calculated wall length for simplified hand calculations was 14.6 m, and this was increased to 16.6 m to allow for the simplifications made. Table E14.1, Column 1 shows the results of a calculation based on the same principles, but carried out using the *Oasys* program STAWAL, so avoiding the need for simplifications in the equilibrium assumptions. The resulting wall length is 15.2 m, lying between the two values of CIRIA 104. The bending moment derived from this calculation, 822 kNm/m, is disregarded in the CIRIA method. Column 2 shows the results of CIRIA's bending moments calculation, in which unit factors are applied to soil strength, but the resulting bending moment is increased by a factor of 1.5. The bending moment derived for ULS design of the sheet pile is 455 kNm/m.

Table E14.1 Results for a cantilever sheet pile wall

Case	CIRIA $F_s = 1.5$ Col 1	CIRIA F = 1 Col 2	CIRIA F = 1 overdig Col 3	EC7 Case C overdig Col 4	EC7 Case C no overdig Col 5	BS 8002 Col 6
$\gamma_\phi = F_s$	1.5	1.0	1.0	1.25	1.25	1.2
ϕ'_d	17.3°	25°	25°	20.5°	20.5°	21.2°
δ/ϕ active	$^2/_3$	$^2/_3$	$^2/_3$	$^2/_3$	$^2/_3$	$^3/_4$
δ/ϕ passive	$^1/_2$	$^1/_2$	$^1/_2$	$^2/_3$	$^2/_3$	$^3/_4$
K_a	0.49	0.36	0.36	0.43	0.43	0.41
K_p	2.28	3.47	3.47	2.8	2.8	3.1
Overdig (m)	0	0	0.4	0.4	0	0.5
Surcharge (kPa)	0	0	0	0	0	10
Data		CANTB6A	CANTB5	CANTB3	CANTB4	CANTB8
Length (m)	15.2	(10.0)	11.95	14.42	12.28	14.60
BM (kNm/m)	(822)	303	511 (= 303 × 1.7)	808	507	959
BM factor		1.5	1.5 ??	1.0	1.0	1.0
ULS BM (kNm/m)	(822)	455	767 ??	808	507	959
Factor α (Z_{el}/Z_{pl})		1		0.8	0.8	??
ULS BM for Z_{el}	(822)	455		646	406	??

Notes: This example is based on CIRIA 104 example B2, page 107
Figures shown in brackets are not used in the design methods
All calculations by *Oasys* STAWAL

Column 4 shows the results of a Case C calculation for EC7. The wall is slightly shorter than derived from CIRIA 104, but the bending moment is very much larger at 808 kNm/m. Comparison of columns 4 and 5 shows that much of the reason for this larger bending moment is EC7's requirement that the surface of the passive soil be considered to be 0.4 m (10% of the supported height) lower than is intended. Without this allowance, Column 5 shows that the bending moment would be 507 kNm/m, 11% bigger than for CIRIA 104, but the overdig causes a further 59% increase in bending moment. It is alarming to realise that a small overdig (0.4 m) places such a large demand on the structural system! This is why a specific allowance is made for this effect in EC7 and in BS 8002.

For comparison, a calculation to BS 8002 is shown in Column 6. This requires a slightly larger allowance for overdig and also imposes a minimum surcharge of 10 kPa. It therefore gives an even higher bending moment, despite the fact that its factor on soil strength (1.2) is less than that of EC7 (1.25), and δ/ϕ is taken as $^3/_4$, compared with $^2/_3$ for steel sheet piling in EC7 (8.5.1(4)).

E14.3 Required bending resistance

Design methods cannot be compared, however, simply on the basis of the bending moments derived. The design of a sheet pile wall is not complete until the steel section has been chosen, and for this purpose, it is necessary to consider the structural codes with which EC7, CIRIA 104, etc are to be used.

EC7 is to be used as part of the Eurocode system with Eurocode 3 (ENV 1993). EC3 Part 5 considers the design of steel sheet piles and is based on concepts of plastic design. For robust sections, which will deform in a ductile manner, EC3-5 allows the use of plastic moments of resistance. Values for these are still somewhat uncertain (early 1998), but they probably exceed the elastic moments of resistance used by previous codes by about 20%, typically. If this proves to be the case, it will roughly halve the difference between the structural demands of EC7 and CIRIA 104. In fact, if overdig were disregarded, EC7 would require a less strong sheet pile than does CIRIA 104.

E14.4 Discussion

The requirement that overdig be specifically allowed for was discussed in C8.3.2.1. This example shows that this requirement, which is common to EC7 and BS 8002, has an important effect and should only be discounted in exceptional circumstances.

It is recognised, however, that many designs to CIRIA Report 104 have performed satisfactorily, despite no allowance for overdig. The analysis above suggests that this might be expected. CIRIA 104 designs generally give longer walls than required by the newer codes, but with lower design bending moments. However, the use of elastic moments of resistance for the section design to CIRIA 104 probably ensures that the wall has sufficient moment capacity. The Eurocode system will allow plastic moments of resistance. Overall, the Eurocodes probably give a better understanding of the compatibility of length and strength in the wall design.

It is noted that, in practice, the choice of steel section is often governed by drivability, which may require a stronger section than that needed for bending moments.

E15 DESIGN OF A PROPPED EMBEDDED WALL

E15.1 Description of problem

A propped, embedded wall is to be designed for the situation shown in Figure E15.1. In this example, a sheet pile wall will be designed, with the aim of minimising the steel section. Deflection will be considered, but it will not govern the design. Water pressure is equalised, approximately, at the bottom of the wall, with linear distributions on either side of the wall.

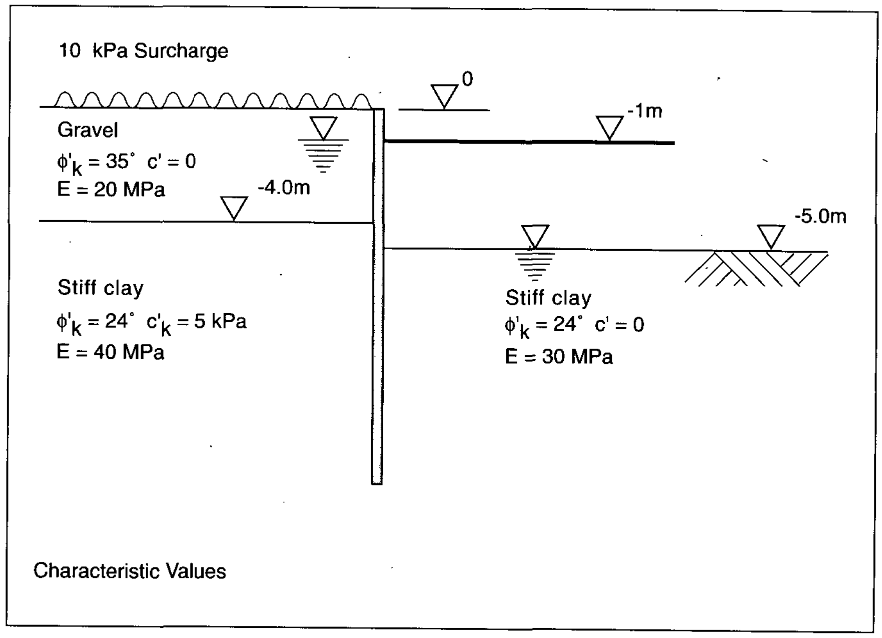


Figure E15.1 Propped embedded wall: characteristic conditions

A brief account of a suitable set of calculations for this design will first be presented. Following this, a more detailed description of comparative calculations carried out for a slight variant on this problem will be given.

Attention has been concentrated on Frodingham (Z) sections in order to avoid the long-standing dispute about shear transfer in the clutches of Larssen (U) sections. This is noted in EC3-5, but is at present left to national decision, to be stated in the NAD of EC3.

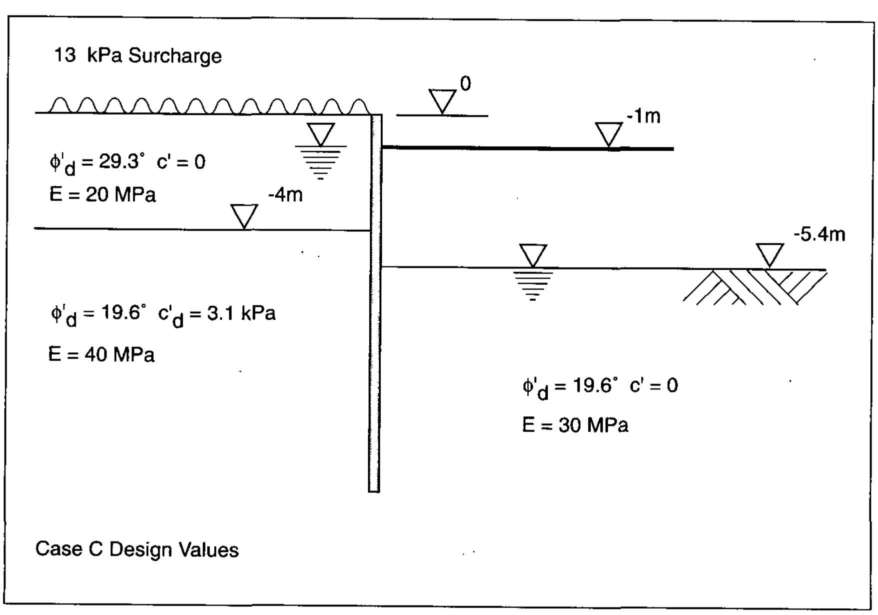


Figure E15.2 Propped embedded wall: Case C design conditions

E15.2 Basic set of calculations

E15.2.1 Parameters and factors

For ULS design, the partial factors given in EC7, Table 2.1 will be used. If the sheet pile wall had only a temporary purpose, and if the consequences of failure would be less than for a typical design of permanent works, it is arguable that smaller partial factors could be used in accordance with 2.4.2(14)P (see C2.4.2 (14)P). CIRIA Report 104, Table 5, could be used to form a view on the proportion by which factors may be reduced for the temporary case.

Figure E15.1 shows the characteristic values of the ground parameters and surcharge. The design values for ULS Case C are shown in Figure E15.2. The depth of excavation is increased by 10%, as required by 8.3.2.1(2), and a partial factor of [1.3] is applied to the surcharge (a variable action).

For ULS Case B, the ground parameter values are unchanged from those in Figure E15.1, but the depth of excavation is increased as in Figure E15.2. The factor on permanent actions, [1.35], is applied as a model factor, as suggested by 2.4.2(17), since it would be unreasonable to apply it directly to water pressures. The effect of the surcharge (a variable action) is to be increased by a factor of [1.5]; this is achieved by multiplying it by 1.1 (≃ 1.5 / 1.35) at source, in the knowledge that its effects on bending moments will be increased by a further factor of 1.35.

E15.2.2 Quick, conservative ULS design

The calculations required for a quick, conservative approach are summarised in Table E15.1. Both Case B and Case C calculations are undertaken, using earth pressure diagrams of the type shown in Figure E15.3. In this case, Case C is found to govern in all respects: length, bending moment, prop force and shear force. For prop and shear force, the difference between the two cases is small, and in some designs Case B is found to govern. Selection of the sheet

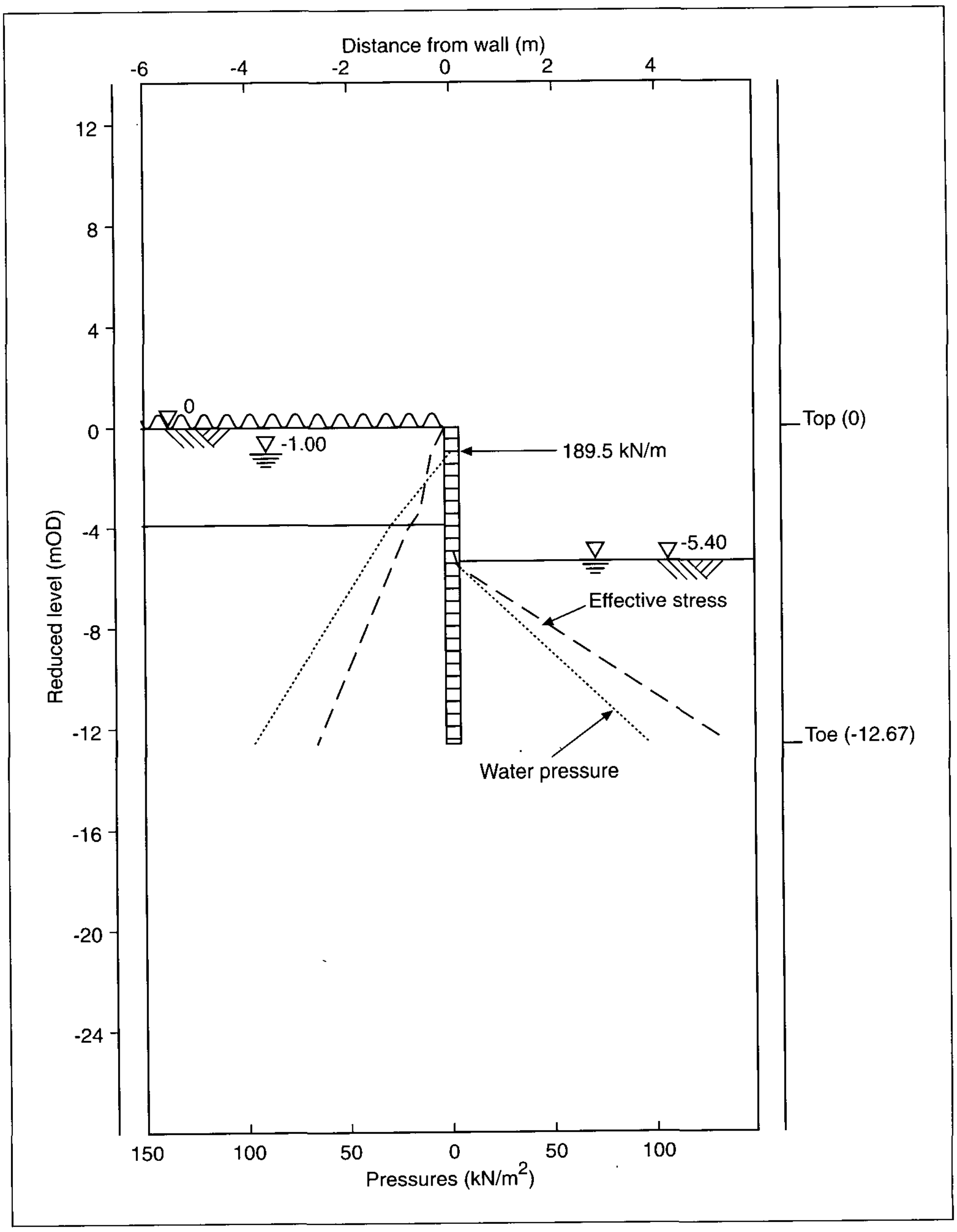


Figure E15.3 Simple earth pressure distribution

Table E15.1 Quick, conservative calculations for a propped embedded wall

	Case C Column 1	Case B Column 2
Dig level (m) – Stage 2	5.4	5.4
Sand and gravel		
Characteristic ϕ' (°)	35	35
γ_ϕ	1.25	1.0
Design ϕ' (°)	29.3	35
δ/ϕ' active	$^2/_3$	$^2/_3$
passive	0	0
K_a	0.29	0.23
K_p	2.9	
Clay		
Characteristic ϕ' (°)	24	24
γ_ϕ	1.25	1.0
Design ϕ' (°)	19.6	24
Characteristic c' (active only – kPa)	5	5
γ_c	1.6	1
Design c' (active – kPa)	3.1	5
δ/ϕ' active	$^2/_3$	$^2/_3$
passive	$^2/_3$	$^2/_3$
K_a	0.43	0.36
K_p	2.6	3.6
a'/c'	$^1/_2$	$^1/_2$
K_{ac}	1.61	1.47
K_{pc}	3.95	
γ_Q	1.3	1.1
Computer program	STAWAL	STAWAL
Data file	JB3C1B	JB3B1B
Wall length (m)	12.7	10.3
Bending moments (kNm/m)		
Maximum calculated	531	312
ULS factor γ_F	1	1.35
ULS design	531	421
SLS design		
Prop force (kN/m)		
Maximum calculated	190	129
ULS factor γ_F	1	1.35
ULS design	190	174
Max shear force (kN/m)		
Maximum calculated	183	124
ULS factor γ_F	1	1.35
ULS design	183	167

Table E15.2 Basic design of a propped embedded wall

	Case C Column 1	Case C Column 2	Case B Column 3	SLS Column 4
Dig level (m) – Stage 2	5.4	5.4	5.4	5.0
Sand and gravel				
Characteristic ϕ' (°)	35	35	35	35
γ_ϕ	1.25	1.25	1.0	1
Design ϕ' (°)	29.3	29.3	35	35
d/ϕ' active	–	–	–	–
passive	0	0	0	0
K_a	0.29	0.29	0.23	0.23
K_p	2.9	1.0	3.6	1.0
Clay				
Characteristic ϕ' (°)	24	24	24	24
γ_ϕ	1.25	1.25	1.0	1
Design ϕ' (°)	19.6	19.6	24	24
Characteristic c' (active only - kPa)	5	5	5	5
γ_c	1.6	1.6	1	1
Design c' (active – kPa)	3.1	3.1	5	5
δ/ϕ' active	$^2/_3$	$^2/_3$	$^2/_3$	$^2/_3$
passive	$^2/_3$	$^2/_3$	$^2/_3$	$^2/_3$
K_a	0.43	0.43	0.36	0.36
K_p	2.6	2.6	3.6	3.6
a'/c'	$^1/_2$	$^1/_2$	$^1/_2$	$^1/_2$
K_{ac}	1.61	1.61	1.47	1.47
K_{pc}	3.95	3.953		
γ_Q	1.3	1.3	1.1	1.0
Computer program	STAWAL	FREW	FREW	FREW
Data file	JB3C1B	JB3C9B	JB3B9B	JB3S5
Wall length (m)	12.67	11.3[a]	11.3[a]	11.3[a]
Bending moments (kNm/m)				
Maximum calculated	531	424	227	119
ULS factor γ_F		1	1.35	
ULS design		424	306	
SLS design				119
Prop force (kN/m)				
Maximum calculated	190	238	165	109
ULS factor γ_F		1	1.35	
ULS design		238	223	
Max shear force (kN/m)				
Maximum calculated	183	213	140	109
ULS factor γ_F		1	1.35	
ULS design		213	189	

[a] Given as data – not calculated by the program

pile section and design of the prop could now proceed using the worst of the values in Table E15.1 as ULS design values for EC3. If needed, a separate analysis for SLS could be carried out as discussed below or by other means.

This approach is likely to lead to larger bending moments than other conventional approaches. However, in many situations the section selected for sheet pile walls has plenty in reserve for bending, often because it is chosen for drivability, so there may be no penalty incurred by larger calculated bending moments.

E15.3 More refined calculation

EC7 requires that the design must demonstrate that equilibrium can be achieved using the design actions and the design strengths, with compatibility of deformations (8.6.1(4)P and 2.1(9)). It does not specify, however, a particular distribution of earth pressures to be used for embedded walls. For the ULS calculations, there is no limit to the magnitude of displacements allowed, unless they would cause a ULS in an adjacent structure. For propped walls, the computed bending moments and prop forces are very dependent on the assumed distribution of earth pressures. The approach adopted here is therefore to make a rough assessment of the required length of the wall, then to use a more refined analysis to verify this and to find the bending moments, which determine the steel section, and the prop forces.

An estimate of the wall length is first determined using the Case C parameters with simple equilibrium calculations, adopting the earth pressure distribution shown in Figure E15.3. This calculation shows that a wall length of 12.7 m will be adequate, though more refined calculation might justify a shorter length. On the basis of judgement, a length of 11.3 m is then used in checks for both Cases B and C, using the *Oasys* program FREW; this allows for redistribution of earth pressure on a basis compatible with deformations. In order to do this calculation, it is necessary to specify stiffness parameters for both the ground and the wall. The ground stiffness has been characterised by Young's moduli, E, as shown in Figure E15.2 and the wall by a bending stiffness EI = 50158 kNm^2/m, corresponding to a Frodingham 3N section. The final results, for ULS design, are not very sensitive to these stiffnesses.

Figures E15.4a and b show the computed earth pressure distributions and bending moments for Case C, at two stages of the excavation. (Note that the scales for bending moment are different in these two plots.) The redistribution of earth pressure, with pressure attracted towards the prop in the second diagram, is clearly evident. It should be remembered that these diagrams are for ultimate limit states in which deformations are large. The maximum computed deflection of the wall in Case C is 137 mm; there is no limit on this figure for the ULS calculation.

The FREW calculations in Table E15.2 show that equilibrium can be achieved for both cases B and C with a maximum ULS design bending moment of 424 kNm/m. As is normal for this type of wall, Case C gives the larger bending moment. The ULS design prop force is 238 kN/m; this is also determined by Case C in this case, but in others Case B sometimes leads to the larger prop force.

The ULS design bending moment may be resisted using a Frodingham 3N sheet pile, which has a ULS capacity of about 590 kNm/m. This section therefore satisfies the design requirements (590 > 424) It may be that design to EC3-5, using the plastic capacity of the sheet piles, might make it possible to adopt a lighter section. The props should be designed to Eurocode 3, using the design ULS force of 238 kN/m.

E15.4 SLS check

Table E15.2 also shows the results of a serviceability computation by FREW. For this, all partial factors are unity and there is no allowance for 'overdig'. The length of wall gives it fixity at the base, as can be seen in Figure E15.5, and the SLS bending moment is 119 kNm/m, only 28% of the design ULS value. Calculations of this type, if done at all, would normally only be used to assess the maximum SLS deflection, which was calculated as 29 mm. This value should be used in assessing effects on the serviceability of adjacent structures or services.

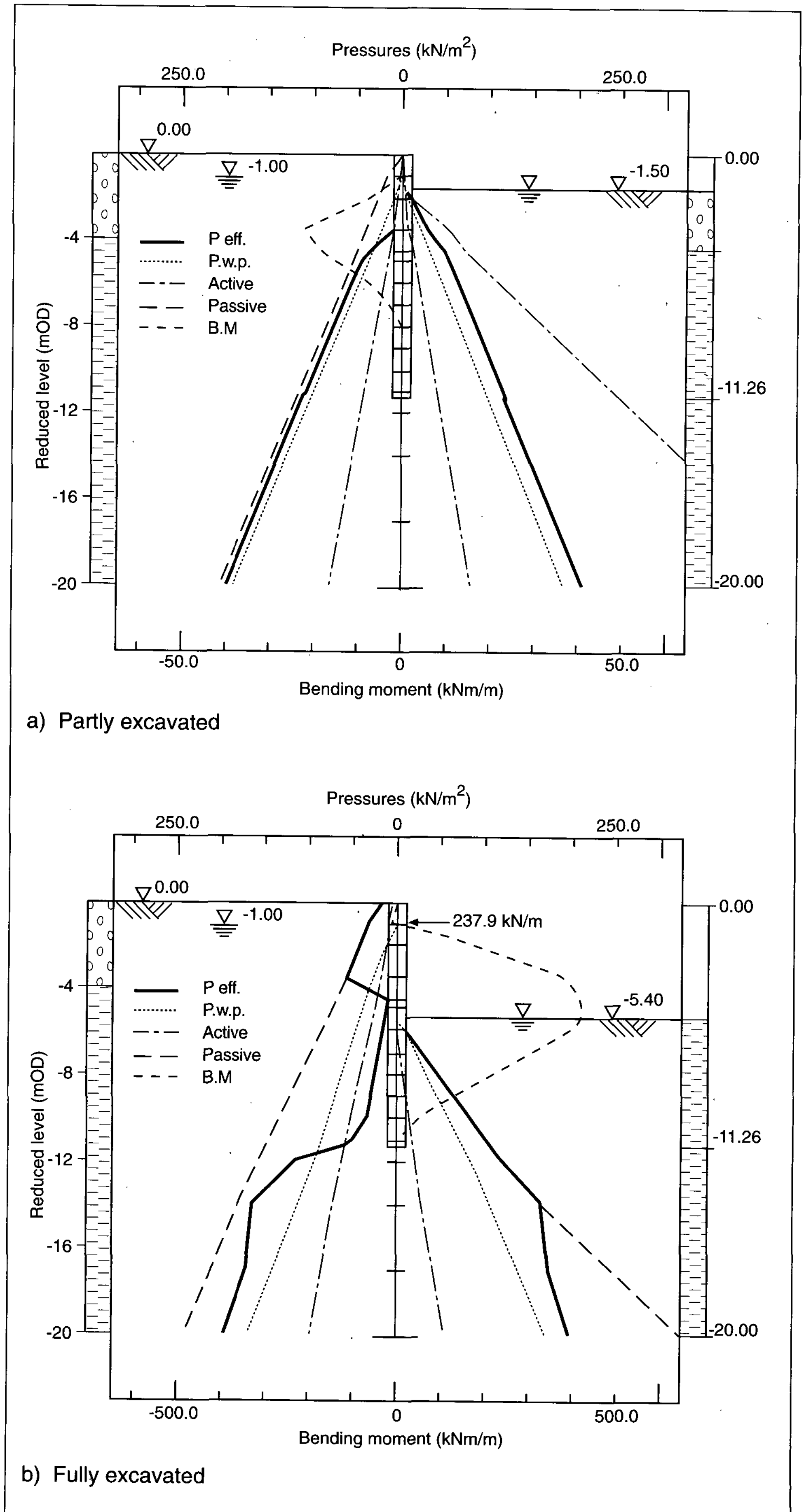


Figure E15.4 Earth pressure distributions from FREW Case C analysis

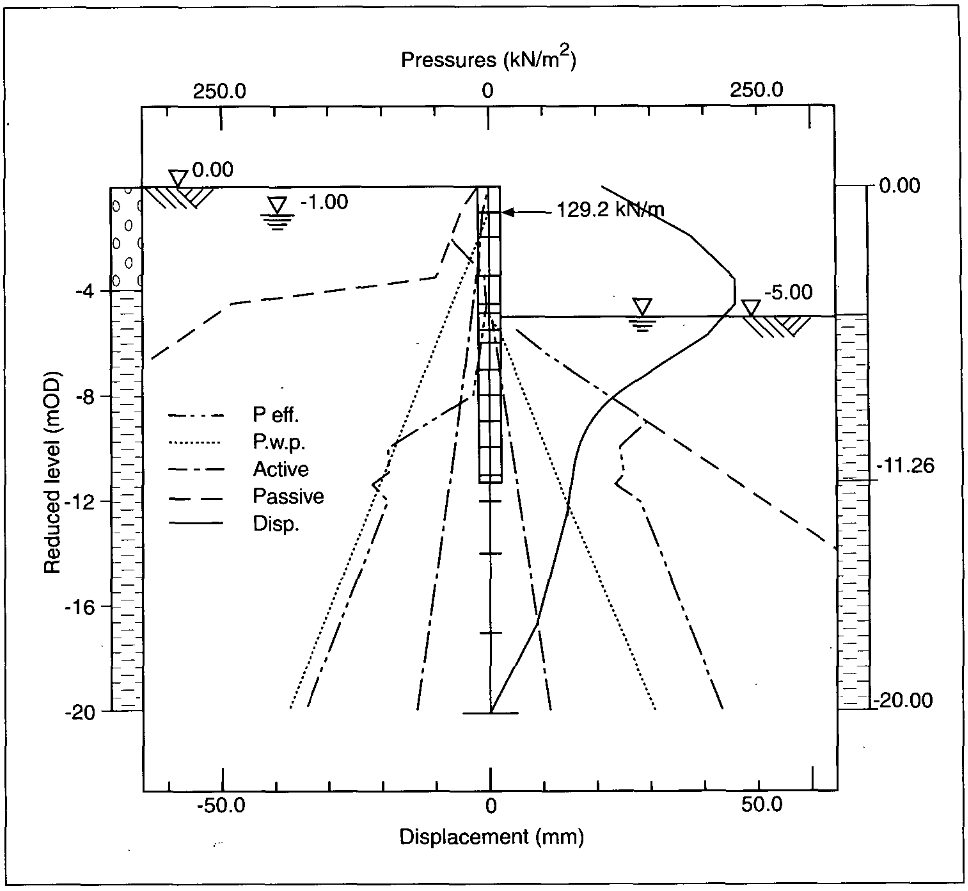


Figure E15.5 Results from FREW SLS analysis

E15.5 Comments

It is of interest to compare the results of the FREW analyses with the simpler calculations represented by Column 1 of Table E15.1, and represented by Figure E15.3. These give a maximum bending moment which is 26% higher than adopted in the design, with a prop force 20% lower. In general, it would be safe to design the sheet pile section using these simpler calculations, but the prop force would be underestimated. Experience may show that these simpler Case C calculations could be used, provided an additional model factor is applied to the prop force, in accordance with the approach of 2.4.1(5)P. This example suggests that the factor might be about 1.25 (=238 / 190).

In German practice for design of sheet pile walls, a set of conventional rules has been developed for redistribution of earth pressures. Reference may be made to Grundbau-Taschenbuch (1992) and EAB (1994).

E15.6 Other checks required by EC7

E15.6.1 Overall equilibrium

Overall equilibrium should be checked in accordance with 8.2(1)P. This depends on the form of the prop or anchor, and could involve slip surfaces passing behind anchorages, for example, or cutting through the fixed anchor length.

E15.6.2 Vertical equilibrium

Vertical equilibrium of the sheet pile should be checked in the ultimate limit state (EC7, 8.6.5). Small inequalities between the downward force from the retained soil and the upward force from the passive soil can be taken out at the base of the wall.

If inclined anchors were used, or if the wall carried a vertical load, this check would be more critical and could lead to revision of the values of δ / ϕ'. If the wall was a concrete diaphragm wall, and $\delta / \phi' = 1$ was used, as allowed by the code, vertical equilibrium might again be critical.

In many cases, the risk of a further iteration of calculation could be avoided by using $\delta / \phi' = 1/2$ in the retained soil but $\delta / \phi' = 2/3$ in the passive soil. This was recommended by Terzaghi (1954), though British practice has tended to interchange these values.

E15.6.3 Drivability

EC7, 8.4(4) notes that sheet piles must have a sufficiently stiff section to be drivable. Table 1.6 of the 'British Steel Piling Handbook' (1997) recommends that 3N sections should not be driven more than 18 m, so this is acceptable here.

The 3N section has an elastic section modulus of 1688 cm^3/m. Table 1 of 'Sheet piling in permanent land based structures' (British Steel General Steels, 1989) suggests a minimum modulus of 2300 cm^3/m to penetrate soil with SPT > 40 for grade 43 steel, and SPT > 45 for grade 50 steel. Hence a bigger section may be required if the upper granular layer is very dense.

In practice, the choice of steel section is often governed by drivability, in which case concerns about calculation of bending moment become academic.

E15.7 Detailed comparative study

E15.7.1 Design problem

In this section, an extensive series of comparative calculations is presented. These would not be needed in a design process, but are included here to facilitate comparisons between results from EC7 and those from other documents and methods. The problem addressed is as shown in Figures E15.1, E15.2 and E15.3, with two aspects differing from the calculations in the previous section:

a water pressures are not equalised at the bottom of the wall, but are assumed hydrostatic on both sides, and

b the bending stiffness used in the FREW analyses is 30 000 kNm^2/m, roughly equivalent to a Frodingham 2N section.

A complete set of results is shown in Table E15.3. The calculations will be referred to by column numbers in this table.

E15.7.2 EC7 calculations

Columns 1 and 2 represent simple calculations similar to those in E15.2 above, with earth pressure diagrams of the type shown in Figure E15.1. Using these calculations alone, the required wall length would be 11.8 m and the ULS design bending moment 465 kNm/m. Column 3 uses a similar simple calculation for SLS, should an SLS bending moment be required. (If a much stiffer wall were to be used, such as a concrete diaphragm wall, this SLS computation would be questionable because it assumes that the retained soil will have reached active pressures in the serviceability state.) Column 4 is based on a FREW calculation for the wall length obtained in Column 1, and shows that the working bending moment for a very flexible wall (EI = 30 000 kNm^2/m) is likely to be about 30% of the ULS design moment in Column 1.

Columns 10, 11 and 12 show three alternative computations for Case C, using the computer programs FREW, SAFE and SPOOKS. All three of these programs compute the redistribution of stresses in the retained material, but in quite different ways: FREW is a pseudo-finite element approach; SAFE is a standard finite element analysis using elastic-Mohr-Coulomb material; SPOOKS uses the plasticity approach to sheet pile design proposed by Brinch Hansen (1953) and Mortensen (1983). These solutions give wall lengths of 11.3 to 11.5 m and bending moments in the range 318 to 376 kNm/m.

Table E15.3 Comparative results for a propped embedded wall

	Case C	Case B	SLS STAWAL	SLS FREW	F_p=2	F_s=1.2, 1.5	SLS STAWAL	Case C Rowe	Case C	Case C	Case C SAFE	Case C SPOOKS	Case C
	Col 1	Col 2	Col 3	Col 4	Col 5	Col 6	Col 7	Col 8	Col 9	Col 10	Col 11	Col 12	Col 13
Dig level (m) – Stage 2	5.4	5.4	5.0	5.0	5.0	5.0	5.0	5.4	5.0	5.4	5.4	5.4	5.0
Sand and gravel													
Characteristic ϕ' (°)	35	35	35	35	35	35	35	35	35	35	35	35	35
γ_ϕ	1.25	1	1	1	1	1.2	1	1.25	1.25	1.25	1.25	1.25	1.25
Design ϕ' (°)	29.3	35	35	35	35	30.3	35	29.3	29.3	29.3	29.3	29.3	29.3
δ/ϕ' active	2/3	2/3	2/3	2/3	2/3	2/3	2/3	2/3	2/3	2/3	2/3 max	2/3	2/3
passive	0	0	0	0	0	0	0	0	0	0	2/3 max	0	0
K_a	.29	0.23	0.23	0.23	0.23	0.28	0.23	0.29	0.29	0.29		0.29	0.29
K_p	2.9	3.6	3.6	1.0	1.8	3.0	3.6	2.9	2.9	1.0		2.9	1.0
Clay													
Characteristic ϕ' (°)	24	24	24	24	24	24	24	24	24	24	24	24	24
γ_ϕ	1.25	1	1	1	1	1.5	1	1.25	1.25	1.25	1.25	1.25	1.25
Design ϕ' (°)	19.6	24	24	24	24	16.5	24	19.6	19.6	19.6	19.6	19.6	19.6
Characteristic c' (active only – kPa)	5	5	5	5	5	5	5	5	5	5	5	5	5
γ_c	1.6	1	1	1	1	1	1	1.6	1.6	1.6	1.6	1.6	1.6
Design c' (active – kPa)	3.1	5	5	5	5	3.33	5	3.1	3.1	3.1	3.1	3.1	3.1
δ/ϕ' active	2/3	2/3	2/3	2/3	2/3	2/3	2/3	2/3	2/3	2/3	2/3 max	2/3	2/3
passive	2/3	2/3	2/3	2/3	1/2	1/2	1/2	2/3	2/3	2/3	2/3 max	2/3	2/3
K_a	0.43	0.36	0.36	0.36	0.36	0.50	0.36	0.43	0.43	0.43		0.43	0.43
K_p	2.6	3.6	3.6	3.6	2.0	2.2	3.4	2.6	2.6	2.6		2.6	2.6
a'/c'	1/2	1/2	1/2	1/2	1/2	1/2	1/2	1/2	1/2	1/2	1/2	1/2	1/2
K_{ac}	1.61	1.47	1.47	1.47	1.47	1.73	1.47	1.61	1.61	1.61		1.61	1.61
K_{pc}	3.95	4.60	4.60	4.6	2.6	3.6	4.5	3.95	3.95	3.95		3.95	3.95
γ_0	1.3	1.1	1.0	1.0	1.0	1.0	1.0	1.3	1.3	1.3	1.3	1.3	1.3
Computer program	STAWAL	STAWAL	STAWAL	FREW	STAWAL	STAWAL	STAWAL	Rowe	STAWAL	FREW	SAFE	SPOOKS	FREW
Data file	JB3C1A	JB3B1	JB3S3	JB3S5	JB3FPC JB3S4	JB3FSB JB3S4	JB3S4	STAWAL JB3C1A	JB3C7A	JB3C2B	JB3C9		JB3C6B
Wall length (m)	11.8	9.6	(8.9)	11.4[a]	12.9	12.5	(9.1)	11.8	10.9	11.3[a]	11.3[a]	11.5	11.3[a]
Bending moments (kNm/m)													
Maximum calculated	465	275	215	135	(436)	(482)	224	465	366	318	350	376	273
ULS factor γ_F	1	1.35					1.5		1	1	1	1	1
ULS design	465	371			336	336	336	349/314	366	318	350	376	273
SLS design			215	135	224	224	224						
Prop force (kN/m)													
Maximum calculated	175	120	103	129	(158)	(176)	105		152	194	236	191	179
ULS factor γ_F	1	1.35					2		1	1	1	1	1
ULS design	175	162			210	210	210		152	194	236	191	179
Max shear force (kN/m)													
Maximum calculated	169	115	98	107	(153)	(170)	101		145	169	180		154
ULS factor γ_F	1	1.35					1.5		1	1	1		1
ULS design	169	155		152		152	152		145	169	180		154

[a] Given as data – not calculated by the program

E15.7.3 CIRIA 104 calculations

Column 5 shows a traditional calculation, noted by CIRIA Report 104, in which a factor of safety of 2 on passive effective stress used. For this, a longer length is required, 12.9 m, and the calculated bending moment is 436 kNm/m. However, this bending moment would traditionally have been disregarded.

Column 6 shows the calculation required by CIRIA Report 104's 'strength factor method', with factors of 1.2 on $\tan\phi'$ in the sand and gravel, and 1.5 in the clay. The calculated length is 12.5 m and the bending moment 482 kNm/m. CIRIA 104 recommends that this bending moment be disregarded and the value calculated in Column 7 substituted. Here, soil parameters are unfactored, but a factor of 1.5 is applied to bending moment and 2.0 to prop force. These factored results could be used with the wall lengths obtained in Columns 5 or 6. The Column 7 calculation is very similar to Column 3, except that CIRIA 104 limits δ / ϕ' in the passive soil to $^1/_2$, whereas EC7 allows $^2/_3$. The resulting ULS design bending moment from this CIRIA 104 approach is 336 kNm/m.

E15.7.4 Rowe's method

Rowe (1952, 1955) recognised that the type of calculation in Column 5 gave large bending moments, and so derived 'working' values by applying factors to allow for flexibility to the moment from $F_p = 2$ calculations. The method is very rational, but ingenuity is required to apply it within a limit state framework, especially to a problem involving clays.

Rowe's factor r_t allows for the beneficial effect of higher pressures above the prop and correspondingly lower active pressures below. It seems reasonable to apply this to the calculated Case C bending moment. Its value in this case is about 0.75, reducing the bending moment to $431 \times 0.75 =$ 323 kNm/m.

Rowe's factor r_d allows for the beneficial effect, in the working state, of the length of wall greater than required by the working soil strengths. Its value here is about 0.6 and it could be used to derive a working bending moment of $323 \times 0.6 = 194$ kNm/m. Following CIRIA Report 104, this could be multiplied by 1.5 to derive an alternative ULS design bending moment of $194 \times 1.5 = 290$ kNm/m.

E15.7.5 Significance of 'overdig'

Columns 9 and 13 are used to show how big is the effect of the 'overdig' allowance of 0.4m. Comparing Columns 1 and 10, the effect is 29% and 16%, respectively, depending on the type of calculation used.

E15.7.6 Discussion

For this situation, traditional calculations lead to a wall length of about 12.7 m (Columns 5 and 6) with a design ULS bending moment of about 336 kNm/m (Columns 7 and 8). EC7 design allows a shorter wall, 11.8 m long or, with more refined calculation, down to 11.3 m or less. In its simplest form, the EC7 calculation is likely to lead to higher bending moments (Column 1), with the consequent need for a stronger wall section in some cases. However, within the design requirements of EC7 the bending moments can be reduced to be similar to traditional values (Columns 8, 10, 11 and 12). The selection of steel section will be dependent on EC3-5; although this allows the plastic moment of resistance to be used for more robust steel sections, but the effect may be fairly small for the relatively light sections used here. The section selected for an EC7 design following Columns 8, 10, 11 or 12 may therefore be similar to that of a CIRIA 104 design.

The prop forces computed for the EC7 design (Columns 10 to 12) are similar to that required by CIRIA 104, including its requirement of a factor of safety of 2 on the initially calculated value (Column 7). The simpler calculation to EC7 in Column 1 gives a lower ULS design prop force; whilst this could be safe, given the higher requirement for bending strength of the wall, there may be danger in lowering the design force in props, which may create instability by buckling.

The conventional methods, reported by CIRIA Report 104, have been criticised for giving walls with incompatible length and strength. Their bending capacity is not sufficient to use their full length if it should prove to be needed. This results from the use of two independent calculations for length and strength. The EC7 design requires that, however the calculations are performed in detail, the resulting wall must be both long enough and strong enough to survive in the applied ULS conditions. The net result will be walls which are a little shorter, but probably of similar strength to those of conventional British practice.

It may be that, with further trials and experience, it will be possible to show that simpler calculations, such as those of CIRIA Report 104, always lead to designs which satisfy the requirements of EC7, used with EC3-5.

In the simpler approach presented in E15.2, the design used the equivalent of Column 1, only as an initial indication of length, with Column 10 to validate a shorter length and find the design bending moment.

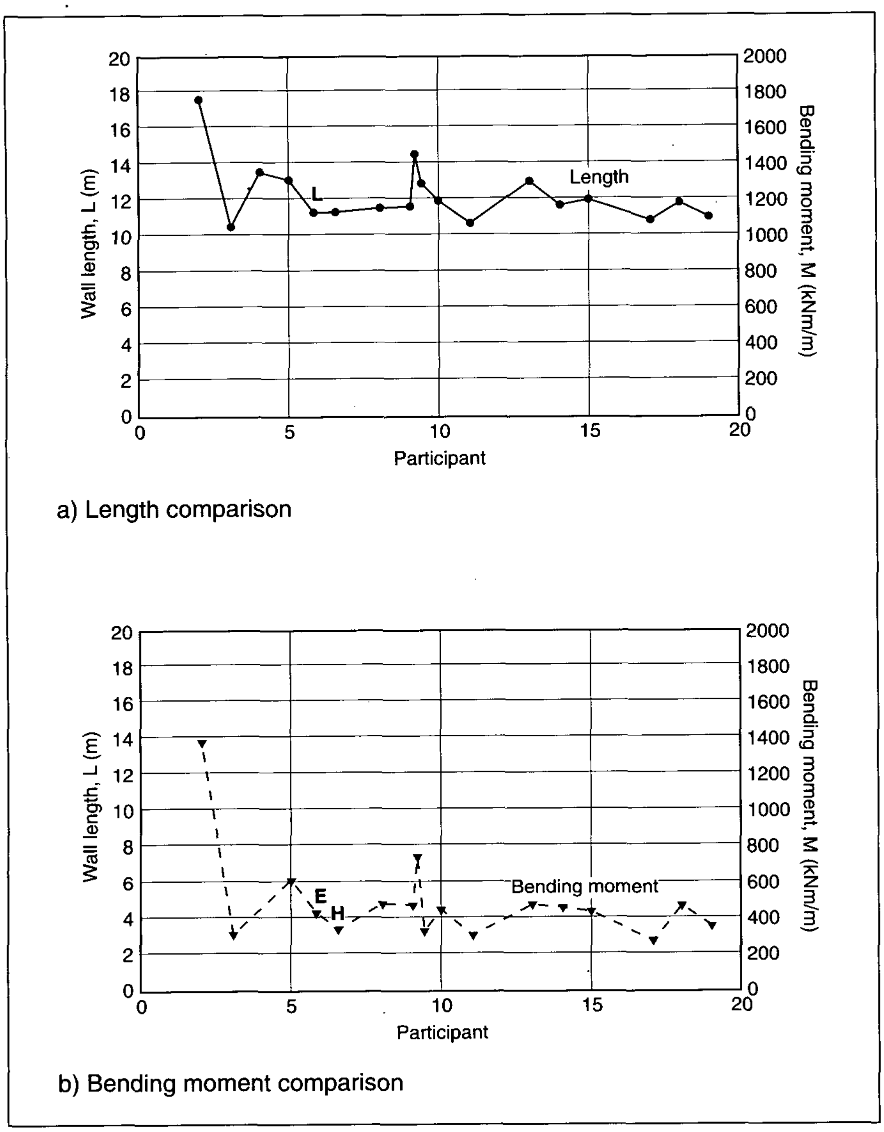


Figure E15.6 Propped wall: international comparison

E15.7.7 International comparison

Figure E15.6 shows results of calculations submitted to the Eurocode committee CEN/TC250/SC7 for this design by 19 engineers representing most of the European countries. Most of the derived lengths and ULS design bending moments lie in a small range, and it is likely that the few outliers represent clear mistakes or misunderstandings. The length obtained in Column 10 of Table E15.3 is marked 'L' and can be seen to be typical of the designs proposed. The bending moment from Column 10 is marked 'H', noting that this was for hydrostatic water pressures either side of the sheet pile. The value from Table E15.2, Column 2, for which water pressures were equalised at the bottom of the sheet pile, is marked 'E'. Both points E and H are within the range obtained by other European engineers; their assumptions about pore pressure distribution are not known.

E16 DESIGN OF A GROUND ANCHOR

E16.1 Description of problem

In E15 it was assumed that the sheet pile wall was supported by a prop. Here, it will be assumed that ground anchors are to be provided, which will give the same horizontal force as that obtained in Table E15.2. The critical prop force for ULS design was 238 kN/m from Case C.

It will be assumed that the anchors are to be inclined at 30° to the horizontal and are to be permanent. At least three anchors will be subject to assessment tests.

The design approach provided below is considered to be consistent with ENV 1997-1:1994. This is not fully consistent with ENV 1537, and further harmonisation is expected.

E16.2 Method

In C8.8.2, it was recommended that the force taken from the ULS wall calculation should be treated as the required ULS design anchorage resistance. The required **ULS design** anchorage resistance is therefore $R_{ad} = 238 \sec 30°$, ie 275 kN/m inclined at 30°. Reference should be made to Table C8.2 and to EC7, 8.8.4 and 8.8.5.

From 8.8.5(6)P, the **characteristic** resistance, R_{ak}, is given by

$R_{ak} = R_{ad} \times \gamma_m$

$= 275 \times [1.5]$ for a permanent anchorage

$= 413$ kN/m.

EC7 only allows anchors to be designed by testing. Methods of sizing anchors which will be subject to test are not given in EC7, but can be found in ENV1537. EC7, 8.8.5(4)P shows how the required test result is obtained from the characteristic resistance:

$R_{am} \geq R_{ak} \times \xi$.

If there will be at least three tests, the values of ξ are [1.3] on the mean test result and [1.1] on the lowest value. Hence the mean test result must be not less than 413 × [1.3] = 537 kN/m, and the minimum not less than 413 × [1.1] = 454 kN/m.

In relation to the design ULS required resistance, this will give an average overall factor of safety not less than 537 / 275 = 1.95. The design ULS required resistance was derived from calculations which already incorporated partial factors of safety on the soil materials.

E16.3 Testing

Suppose that four tests are carried out, giving results 780 kN, 840 kN, 730 kN and 890 kN. Then the minimum is 730 kN and the mean 810 kN. The minimum would allow a spacing of 730 / 454 = 1.61 m, and the mean would allow 810 / 537 = 1.51 m. A spacing of 1.50 m might sensibly be selected.

Hence the ULS design resistance per anchor becomes 275 × 1.5 = 413 kN, and the characteristic resistance 620 kN per anchor.

E16.4 Use in the construction

Table C8.2 suggests that, for use in the construction, these anchors would typically be preloaded to 1.5 times the design resistance, ie 1.5 × 413 = 620 kN per anchor, which happens to equal their characteristic resistance. They might be locked off at 1.1 times the design resistance, ie 454 kN per anchor.

E16.5 Other design checks

Overall equilibrium must be checked in accordance with EC7, 8.6.2, and vertical equilibrium of the wall, with the anchor load, in accordance with EC7, 8.6.5. The length of the anchor must be sufficient to comply with the assumptions made in the design of the retaining wall. In E15, for example, no attention was given to the anchor, so it is necessary that the fixed anchor length is sufficiently remote to have no influence on the earth pressures on the wall. EC7, 8.8.2(7) requires a minimum free anchor length of 5 m.

The strength of steel strand and of the steel-grout interface must be checked in accordance with ENV 1537.

EC7, 8.8.3 provides durability requirements.

E16.6 Comment

The basic formulation of Eurocode calculations requires that design actions and action effects are matched by design resistances. In this case, the anchor load taken from the wall calculations already incorporates factors of safety (for either Case B or Case C), so that it could be used without further load factors for design of steel or concrete struts. It is logical that this should be taken as the design resistance of the anchors, but the application of a further factor of 1.5 to convert the required design resistance to a required characteristic resistance may be rather conservative. The use of γ_m closer to unity for anchor design might be considered when used in conjunction with ULS design forces from wall calculations.

Table E16.1 shows the 'overall factor of safety' at various stages in the calculation, expressed in relation to the SLS (ie characteristic) or ULS anchor load.

Table E16.1 Overall factors of safety implied by anchor design

Calculation stage	Force kN/m	'FOS' cf SLS	'FOS' cf ULS
SLS anchor load from wall calculations	126[a]	1.00	
ULS design anchor load from wall calculations = design ULS anchor resistance	275[a]	2.18	1.00
Characteristic anchor resistance	275 × 1.5 = 413	3.04	1.50
Required minimum test result	413 × 1.1 = 454	3.60	1.65
Required mean test result	413 × 1.3 = 537	4.26	1.95
Typical preload for working anchors	275 × 1.5 = 413	3.04	1.50
Typical lock-off load for working anchors	275 × 1.1 = 303	2.40	1.10

[a] Values from Table E15.2 have been multiplied by sec30°.

E17 DESIGN OF SLOPE IN DRAINED GROUND

E17.1 Introduction

EC7, 9.5 considers the design of slopes and embankments by adopting partial factors for permanent and variable actions and specific ground properties. It states that these factors shall generally be used for verification of ultimate limit states of conventional types of structures and foundations in persistent and transient situations.

In C9 it is noted that when considering the Ultimate Limit State for a slope it is only necessary to consider Case C in Table 2.1. It is noted in Paragraph 2.4.2(15) that Case A is only applicable to buoyancy problems and that where there is no strength of structural materials involved Case B is irrelevant.

E17.2 Data

The slope in question is a cutting in a stiff overconsolidated clay, as shown in Figure E17.1. The characteristic soil strength properties are:

$\phi'_k = 24°$

$c'_k = 10$ kPa.

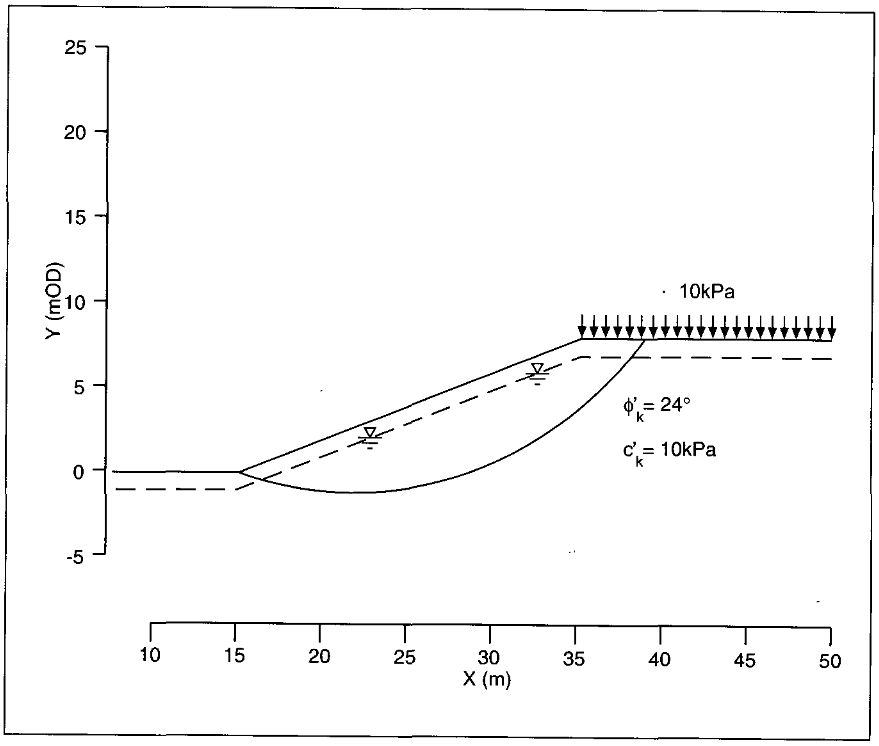


Figure E17.1 Slip circle in drained ground

The bulk unit weight of the clay is 20kN/m^3. The slope is 8 m high and has an inclination of 22°. Drainage provisions at the site provide a groundwater level which is hydrostatic from 1 m below the finished ground level. There is a permanent UDL of 10 kPa at the top of the slope.

Three slope stability problems have been analysed using the *Oasys* program SLOPE. The three analyses are:

Calculation 1 Demonstration that the slope as designed with the design parameters satisfies the requirements of EC7;

Calculation 2 Calculation of the global Factor of Safety for the slope in Calculation 1 assuming that the effective cohesion (c′) is equal to zero;

Calculation 3 Calculation of allowable slope using EC7 factors when the effective cohesion is equal to zero.

All analyses use a solution based on Bishop (1955) with variably inclined interslice forces.

The data and results of the analyses are shown in Table E17.1.

The critical slip circle for Calculation 1 is shown in Figure E17.1, and is typical of the circles for the three calculations.

Table E17.1 Summary of calculations

Input conditions	Calculation 1	Calculation 2	Calculation 3
UDL_k	10 kPa	10 kPa	10 kPa
γ_{UDL}	1.3	1.0	1.3
UDL_d	13 kPa	10 kPa	13 kPa
ϕ'_k	24°	24°	24°
$\gamma_{\tan\phi'}$	1.25	1.0	1.25
ϕ'_d	19.6°	24.0°	19.6°
χ'_k	10 kN/m^2	0	0
$\gamma_{c'}$	1.6	–	–
c'_d	6.25	–	–
Results:			
angle of slope	22°	22°	13.9°
additional factor of safety, global	1.01	0.84	1.01

E17.3 Discussion

Provided there is sufficient confidence in the characteristic values of the parameters ('cautious values' – see B4 and EC7, 2.4.3), only Calculation 1 is needed to satisfy EC7. It demonstrates that the slope is stable when the partial material factors are applied to the soil strength.

The results of Calculations 1 and 2 show that the effective cohesion, c′, plays a very significant role in the stability of the slope. In Calculation 1 the slope is seen to be stable with a global factor of safety in excess of 1.26 (ie 1.25 [γ_ϕ] × 1.014 [additional FOS]), satisfying stability requirements. However, in Calculation 2, when the effective cohesion is removed, the slope is seen to have a global factor of safety of 0.84, which is clearly unacceptable. This leads to the observation that designers must be very confident in the parameters that they use in slope stability analyses. This is particularly the case where c′ is ascribed to the ground at shallow levels where it plays a dominant role in securing the stability of a slope. If it is decided that the effective cohesion cannot be relied upon, then the slope presented in Calculation 3 will be the limiting design slope. This has an inclination of only 13.9° or approximately 1 in 4.

E18 ULS CHECK ON A SIMPLE, POTENTIALLY BUOYANT STRUCTURE

In this example, the values of partial factors are taken directly from EC7 Table 2.1. However, it is argued in B5.7 that the value assigned to γ_G when it reduces beneficial permanent loads is uncomfortably small.

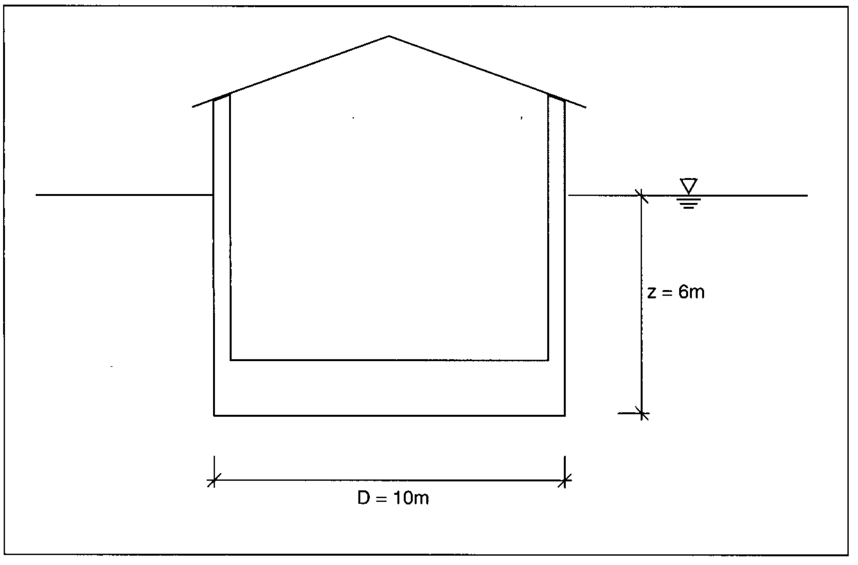


Figure E18.1 Simple, potentially buoyant structure

The partially buried structure shown in Figure E18.1 is circular, 10 m in diameter (D, externally) and extends z = 6 m below ground level. The worst credible groundwater pressures are represented by a water table at the ground surface with hydrostatic pressures beneath ($\gamma_w = 10$ kN/m^3). The characteristic minimum weight of the structure is $W_k = 2400$ kN, and the characteristic shear strength s_k between soil and structure may be taken to be 15 kPa in the long term (based on tanϕ', which, in this case, is found to give a lower resistance than use of undrained strength).

Therefore uplift due to design water pressure
$= \gamma_w.z.\pi D^2 / 4$
$= 4710$ kN.

Characteristic side shear resisting uplift
$= \pi Dz \times s_k$
$= 2827$ kN.

For Case A, EC7 Table 2.1 requires a partial factor of 1.1 to be applied reducing the side shear (1.2 if it were based on c_u) and 0.95 reducing the available weight of the structure.

So Case A requires:

$W_k \times 0.95 + (\pi Dz \times s_k) / 1.1 \quad \geq \gamma_w.z.\pi D^2 / 4$
ie $2400 \times 0.95 + 2827 / 1.1 \quad \geq 4710$
ie $2280 + 2570 = 4850 \quad \geq 4710$ – which is OK.

By inspection, Case B is less critical than Case A (but see note below).

For Case C, Table 2.1 requires a partial factor of 1.25 to be applied reducing the side shear (1.4 if it were based on c_u) with 1.0 on the available weight of the structure.

So Case C requires:

$W_k \times 1.0 + (\pi Dz \times s_k) / 1.25 \quad \geq \gamma_w.z.\pi D^2 / 4$
ie $2400 \times 1.0 + 2827 / 1.25 \quad \geq 4710$
ie $2400 + 2262 = 4662 \quad < 4710$ – which marginally fails at Case C.

The structure must comply with all three cases, so the Case C calculation shows that its weight must be increased.

Note: The implication of 2.4.2(17) is that EC7 does not require that a factor of 1.35 be applied to the water pressure if this would be physically unreasonable. However, in this case, the factor 1.35 should be applied to calculated bending moments in the base slab. For other examples of this, see C2.4.2(17); on water pressures, see C8.3.2.2.

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