A Designers’ Simple Guide
to BS EN 1997
On 5th May 2006 the responsibilities of the Office of the Deputy Prime Minister (ODPM)
transferred to the Department for Communities and Local Government

Department for Communities and Local Government
Eland House
Bressenden Place
London
SW1E 5DU
Telephone: 020 7944 4400
Website: www.communities.gov.uk

A Designers’ Simple Guide to BS EN 1997 December 2005
It should be noted that this guidance has been based on the published
Eurocode, BS EN 1997-1:2004, together with the draft of its National Annex
that was available at the time of writing

© Crown Copyright, 2006

Copyright in the typographical arrangement rests with the Crown.

This publication, excluding logos, may be reproduced free of charge in any format or medium
for research, private study or for internal circulation within an organisation. This is subject to
it being reproduced accurately and not used in a misleading context. The material must be
acknowledged as Crown copyright and the title of the publication specified.

Any other use of the contents of this publication would require a copyright licence. Please apply
for a Click-Use Licence for core material at www.opsi.gov.uk/click-use/system/online/pLogin.asp,
or by writing to the Office of Public Sector Information, Information Policy Team, St Clements
House, 2-16 Colegate, Norwich, NR3 1BQ. Fax: 01603 725000 or email:
HMSOlicensing@cabinet-office.x.gsi.gov.uk

If you require this publication in an alternative format please email
alternativeformats@communities.gsi.gov.uk

Department for Communities and Local Government
PO Box 236
Wetherby
West Yorkshire
LS23 7NB
Tel: 08701 226 236
Fax: 08701 226 237
Textphone: 08701 207 405
Email: communities@twoten.com
or online via the Department for Communities and Local Government website:
www.communities.gov.uk

January 2007

Product Code: 06 BD 04021 (d)
CONTENTS

CHAPTER 1 A DESIGNERS’ SIMPLE GUIDE TO BS EN 1997
1.1 Introduction to the new EU geotechnical Codes and Standards 7
1.2 About this Guide 9
1.3 Some basic points about BS EN 1997 11

CHAPTER 2 A BRIEF OVERVIEW OF BS EN 1997-1 - GEOTECHNICAL DESIGN, GENERAL RULES. 14
2.1 Introduction 14
2.2 The design philosophy of BS EN 1997-1 17
  2.2.1 Limit states 17
  2.2.2 Design Requirements 18
  2.2.3 Design Situations 22
  2.3. Geotechnical design methods 22
    2.3.1 Ultimate Limit State Design by Calculation 22
    2.3.2 Calculations for STR and GEO Ultimate Limit State design 25
    2.3.3 Performing Serviceability Limit State design checks 26
    2.3.4 Design by Prescriptive Measures 27
    2.3.5 Design using load tests and tests on experimental models 27
    2.3.6 Design using the Observational Method 27
   2.4 The Geotechnical Design Report 28

CHAPTER 3 OBTAINING GEOTECHNICAL DESIGN INFORMATION 30
3.1 Introduction 30
3.2 Using prEN 1997-2 to obtain ground parameter values from tests 30
  3.2.1. Introduction 30
  3.2.2 Geotechnical Investigations 31
  3.2.3. Soil and rock sampling and groundwater measurements 33
  3.2.4. Field tests in soil and rock 34
  3.2.5. Laboratory tests in soil and rock samples 34
  3.2.6. The Ground Investigation Report 35
3.3 Characteristic values of geotechnical parameters 35
  3.3.1. Characteristic values depend on failure mode 39
  3.3.2. Other attempts to express uncertainty in ground parameter values 39
  3.3.3. Significance of statistical methods 40
  3.3.4. Characteristic values of stiffness and weight density 40
CHAPTER 4 DESIGN CALCULATIONS FOR FOUNDATIONS AND RETAINING STRUCTURES 42

4.1 Introduction 42

4.2 Using Design Approach 1 in GEO and STR ULS calculations 42
  4.2.1 Combination 1 44
  4.2.2 Combination 2 44

4.3 Spread foundations 45
  4.3.1 Overall Stability 45
  4.3.2 Design of the foundation 45

Example 4.1. 48
Example 4.2 57

4.4 Piles 62
  4.4.1 General 62
  4.4.2 Calculating ultimate compressive resistance using ground parameters from tests 64
  4.4.3 Vertical displacements of pile foundations (serviceability of supported structure) 68
  4.4.4 The structural design of piles 69
  4.4.5 Aspects of the construction of piles 69

4.5 Anchorages 69

4.6 Retaining structures 69
  4.6.1 Introduction 69
  4.6.2 Limit States 69
  4.6.3 Actions, geometrical data and wall friction 70
  4.6.4 Design and construction considerations 72

Example 4.3 76
Example 4.4 82
Example 4.5 86
Example 4.6 94
Example 4.7 102
### CHAPTER 5 OTHER GEOTECHNICAL DESIGN AND CONSTRUCTION MATTERS

#### 5.1 General

#### 5.2 Overall stability

- **5.2.1 Limit States**
- **5.2.2 Actions and design situations**
- **5.2.3 Design and construction considerations**
- **5.2.4 Ultimate limit state design**

#### 5.3 Design of embankments

- **5.3.1 Limit States**
- **5.3.2 Actions and Design Situations**
- **5.3.3 Design and Construction Considerations**
- **5.3.4 Ultimate Limit State Design**
- **5.3.5 Serviceability Limit State Design**
- **5.3.6 Supervision and Monitoring**

#### 5.4 Supervision of construction, monitoring & maintenance

- **5.4.1 Introduction**
- **5.4.2 Supervision**
- **5.4.3 Checking ground conditions**
- **5.4.4 Checking construction**
- **5.4.5 Monitoring**

#### 5.5 Fill, Dewatering, Ground Improvement and Reinforcement

- **5.5.1 Introduction**
- **5.5.2 Fundamental requirements**
- **5.5.3 Constructing with Fill**
- **5.5.4 Dewatering**
- **5.5.5 Ground improvement and reinforcement**

### CHAPTER 6 EUROPEAN GEOTECHNICAL CONSTRUCTION STANDARDS

#### 6.1 Introduction

#### 6.2 Compatibility between BS EN 1997-1 and Execution Standards
CHAPTER 7 HOW THE NEW GEOTECHNICAL CODES AND STANDARDS WILL BE APPLIED AND MAY IMPACT ON UK PRACTICE

7.1 Introduction

7.2 How the BS ENs will be implemented
   7.2.1 The UK National Annexes
   7.2.2 Withdrawal of BS codes and Standards.

7.3 A timetable for change

7.4 How the BS ENs may be applied

CHAPTER 8 REFERENCES AND BIBLIOGRAPHY

CHAPTER 9 APPENDICES

9.1 Contrasting design philosophies

9.2 The three alternative ‘Design Approaches’

9.3 The other Ultimate Limit States

9.4 The correspondence for ground investigation and testing between BS documents and BS EN documents.

9.5 Significance of statistical methods

9.6 Calculating pile ultimate compressive resistance using static load tests

9.7 Calculating pile ultimate compressive resistance using the results of dynamic testing

9.8 Tensile resistance of piles
1. ‘A DESIGNERS’ SIMPLE GUIDE TO BS EN 1997’

1.1. Introduction to the new EU geotechnical Codes and Standards

The current suite of BS Codes and Standards will, in due course, be almost entirely replaced by a system of Eurocodes and Standards (ENs) published by BSI as BS ENs; it is expected that the replacement will be complete by about 2010.

The Eurcodes adopt, for all civil and building engineering materials and structures, a common design philosophy based on the use of separate limit states and partial factors, rather than ‘global’ factors (of safety); this is a substantial departure from much traditional geotechnical design practice as embodied in BS Codes such as BS 8004. Furthermore, the geotechnical design Eurocode (BS EN 1997-1) provides one, unified methodology for all geotechnical design problems; an advantage of BS EN 1997-1 is that its design methodology is largely identical with that for all of the structural Eurocodes, making the integration of geotechnical design with structural design more rational.

Reaching agreement between the EU Member States on this unified geotechnical methodology resulted in what might initially appear to be a rather complicated system of equations for use in design calculations; in addition, it led to the introduction of concepts and terminology that may not be familiar to many designers of foundations and other geotechnical structures. For these reasons, it was felt appropriate to provide this Guide to help people to interpret the new geotechnical Eurocode system of Codes and Standards and to understand how this system will eventually replace the current BS Codes and Standards.

---

1 The following abbreviations are used in this Guide:
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)
BS EN British version of Euronorm
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
2 The process of implementation of the BS ENs and an approximate time-table are discussed in Section 7.3.
3 BS EN 1990 defines limit states as “states beyond which the structure no longer fulfils the relevant design criteria”.

A set of European codes of practice for building structures, the purpose of them being to encourage free trade between Member States of the EU. Over the following years, various groups of specialists have worked to produce a commonly-accepted set of design principles and rules for most aspects of building and construction technologies, leading to a suite of structural ‘Eurocodes’ that is being presented by CEN to National Standards Bodies (BSI, in our case) for implementing. The complete set of CEN Structural Eurocodes is shown in Table 1.1, each generally having a number of Parts presently at various stages of development. The links between the British versions of the Eurocodes are depicted in Fig. 1.1.

The Eurocodes are intended to ensure safe structures, so they will be used by both designers and the checkers of designs.

While the adoption of the Eurocodes will become obligatory in all Member States, the level of safety applicable in a state remains its responsibility. Therefore, such items as the value of partial (safety) factors are left to individual states to determine. Each Eurocode, in Notes, indicates where and how ‘national determination’ may be exercised. This national determination is expressed in a National Annex. Thus the geotechnical Eurocode, EN 1997-1, becomes ‘nationalised’ as BS EN 1997-1 by the inclusion in it of a National Foreword and a National Annex. The National Annex is discussed in Section 7 of this Guide.

<table>
<thead>
<tr>
<th>Number⁷</th>
<th>Name</th>
<th>Subject</th>
</tr>
</thead>
<tbody>
<tr>
<td>EN 1990</td>
<td>Basis</td>
<td>of structural design</td>
</tr>
<tr>
<td>EN 1991</td>
<td>Eurocode 1</td>
<td>Actions on structures</td>
</tr>
<tr>
<td>EN 1992</td>
<td>Eurocode 2</td>
<td>Design of concrete structures</td>
</tr>
<tr>
<td>EN 1993</td>
<td>Eurocode 3</td>
<td>Design of steel structures</td>
</tr>
<tr>
<td>EN 1994</td>
<td>Eurocode 4</td>
<td>Design of composite steel and concrete structures</td>
</tr>
<tr>
<td>EN 1995</td>
<td>Eurocode 5</td>
<td>Design of timber structures</td>
</tr>
<tr>
<td>EN 1996</td>
<td>Eurocode 6</td>
<td>Design of masonry structures</td>
</tr>
<tr>
<td>EN 1997</td>
<td>Eurocode 7</td>
<td>Geotechnical design</td>
</tr>
<tr>
<td>EN 1998</td>
<td>Eurocode 8</td>
<td>Design of structures for earthquake resistance</td>
</tr>
<tr>
<td>EN 1999</td>
<td>Eurocode 9</td>
<td>Design of aluminium structures</td>
</tr>
</tbody>
</table>

Table 1.1 — The suite of primary structural Eurocodes

---

⁴ The National Standards Body must publish the Eurocode without any change to the text. It may also incorporate a National Foreword and a National Annex that implement additional requirements that are generally prescribed by the European Commission (see Section 7).

⁵ Depending on the nature of the project, Eurocode 7 will, of course, be used with one or more of the other structural material Eurocodes for the complete project design.

⁶ Note that while BS EN 1997-1 was published by BSI in December 2004, its National Annex is not yet available.

⁷ The CEN number EN 199x becomes BS EN 199x after publication by BSI.

⁸ Eurocode 7 consists of two Parts: Part 1 (BS EN 1997-1) - Geotechnical design – General rules and Part 2 (prEN 1997-2) - Ground investigation and testing, not yet published.
1.2. About this Guide

Aims of the Guide
The Guide seeks to:

a) give readers a simple explanation of the new suite of complementary geotechnical design, construction, investigation and testing documents that will largely replace the suite of national BSI codes and standards over the next few years;

b) clarify the meanings of new terminology, to explain design calculations and methods that are not covered explicitly in current Codes, to present easy-to-understand explanations of how the new methods work and to do this using simple design examples.

c) explain how to implement the new suite of documents in order to comply with the requirements of the Building Regulations.

What the Guide contains.
This Designers’ Guide contains the following additional Sections:

2. A brief overview of BS EN 1997-1 - Geotechnical design, general rules.
3. Obtaining geotechnical design parameters.
4. Design calculations for foundations and retaining structures.
5. Other geotechnical design and construction matters.
6. European geotechnical construction standards
7. How the new geotechnical Codes and Standards will be applied and may impact on UK practices.
8. References and Bibliography.
9. Appendices.

The principal purpose of the Guide is to provide simple guidance on the interpretation and use of BS EN 1997-1 for the more common design problems encountered in geotechnical practice. The Guide does not offer a detailed, clause-by-clause, commentary on the Code but provides appropriate references to Code clause numbers.

In aiming to keep the Guide as succinct and generally useful as possible, some of the material that deals with less common geotechnical design problems has been moved from the main body of text into Appendices;

9 See Section 1.3 and Fig. 1.2
10 See Section 7.
11 A comprehensive Designers’ Guide is available (Frank et al, 2004). For detailed comments on the material common to all the Eurocodes, the reader may refer to the Introduction in the Designers’ Guide to EN 1990 Eurocode: Basis of structural design (Gulvanessian et al, 2002); this also provides information on the implementation and use of the structural Eurocodes in the EU Member States. An essential background paper on implementation is Guidance Paper L (concerning the Construction Products Directive – 89/106/EEC). Application and Use of Eurocodes by the European Commission (2001). In addition, Appendix A of the Designers’ Guide to EN 1990 gives detailed information on the Construction Products Directive, the EU directive with which all the national building regulations in the EU Member States must comply.
additional, supplementary information has been placed in ‘Footnotes’, while further information is highlighted in Boxes.

Throughout the Guide emphasis is placed on everyday practice, avoiding complicated geotechnical design cases, in order to ease the understanding of the new concepts and rules for geotechnical design that appear in BS EN 1997-1.

**How to use the Guide.**

This Guide must be read in conjunction with the Code itself. The Guide has been structured somewhat differently from the Code, which does not always follow a logical sequence of design decisions, and so paragraphs in the Guide do not necessarily correspond to clauses in the Code; to aid cross-referencing, important Code clause numbers are given in the right-hand margin of corresponding guidance material. Further, any text reproduced from the Code is shown in *italics*. The symbols adopted in the Guide are those used and defined in BS EN 1997-1.

Because this Guide has deliberately been kept as succinct and simple as possible, it may be rather less detailed than some readers might like. More comprehensive explanatory material and background information is available (see Section 8).

**Who it's for.**

It is assumed that BS EN 1997 will be used by engineers for general building and civil engineering design. The Eurocode has been written for use by *suitably qualified personnel with relevant experience*. The precise meanings of these terms are not stated, except that qualifications and experience must be appropriate and adequate for the project in hand.

The main purpose of BS EN 1997-1 is to provide a framework for design ‘Principles’ that must be met and it does not give details of how to perform design. Hence inexperienced designers will not be able to proceed adequately using the Eurocode alone. BS EN 1997-1, and therefore this Guide, assumes that the reader has a level of knowledge and experience of soil mechanics and geotechnical engineering appropriate for the project.

The Guide has been written for three different types of readership:

i) The general geotechnical engineer who may often not have routine recourse to Codes but who will, nevertheless, need to be assured that his design complies with the Code requirements;

ii) The more specialised, geotechnical, designer whose employer will be tendering for publicly-procured projects for which designs using the Eurocodes may be obligatory; a good understanding of the details of the Eurocodes will be a necessity.

iii) The non-geotechnically qualified engineer who may carry out a simple design for small projects for which the ground conditions are not regarded as ‘problematic’, so that a geotechnical

---

12 Note that references to *clauses* in BS EN 1997-1 and prEN 1997-2 (in Section 3) are shown in *italics* while those to paragraphs in this Guide are shown in normal type face.

13 In most cases, therefore, this requires that the performing and checking of designs using BS EN 1997-1 are at least supervised by a qualified engineer with relevant geotechnical training and experience.
specialist is not felt to be required. Such projects often include small housing developments where the foundations may be ‘prescribed’\textsuperscript{14} and where other geotechnical structures require recourse to relatively straight-forward design (such as small retaining walls currently designed using BS 8002:1999).

1.3. Some basic points about BS EN 1997

Eurocode 7 consists of two Parts: Part 1 (BS EN 1997-1) - \textit{Geotechnical design – General rules} and Part 2 (prEN 1997-2) - \textit{Ground investigation and testing}. Both are discussed in this Guide, though Part 1 is of most concern since it lays down the design principles and rules generally to be adopted.

It is important to understand that not all of the documentation covering geotechnical engineering in the EU Member States is included in the two Parts of BS EN 1997. Additionally, as shown in Fig. 1.2, there are or will be Standards for the field investigation and testing, and laboratory testing, of the ground and for the ‘execution’\textsuperscript{15} of special geotechnical works; these are being produced by different CEN Technical Committees from those that wrote EN 1997. There are also Standards for ground identification and classification written by an ISO Technical Committee which are being brought into the CEN system. These other Standards are discussed in Sections 3 and 6.

\textsuperscript{14} ‘Prescription’ is a recognised design method in BS EN 1997-1; see Sections 2.3.4 and 4.3.2.

\textsuperscript{15} ‘Execution’ is the European word for ‘construction’.
Figure 1.1 — The links between the Structural Eurocodes
Figure 1.2: Diagrammatic representation of the suite of EU geotechnical and structural Codes and Standards

- ISO/CEN Standards
- Test Standards
- Identification and classification

- EN 1993-5: Geotechnical Standards
- Other Structural Standards
- European Works Standards

- Geotechnical Projects

- BS EN 1990: Basis of Structural Design
- BS EN 1991-1-1: Actions on Structures
- BS EN 1997-1: Geotechnical Design
- BS EN 1997-2: Geotechnical Design
- BS EN 1997-1-1: Structural Stands

Other Structural Eurocodes: e.g. EN 1993-5
Geotechnical Projects
Geotechnical Design

European Standards for the Execution of Special Geotechnical Works

Geotechnical Projects
Geotechnical Design
2. **A BRIEF OVERVIEW OF BS EN 1997-1 - GEOTECHNICAL DESIGN, GENERAL RULES.**

2.1. Introduction

BS EN 1997 has two Parts: BS EN 1997-1 which covers ‘Geotechnical design – general rules’ and prEN 1997-2 which covers ‘Ground investigation and testing’. Section 2 of this Guide discusses BS EN 1997-1 while Section 7 covers prEN 1997-2.

It is important to appreciate that BS EN 1997-1 is not a detailed geotechnical design manual but is intended to provide a framework for design and for checking that a design will perform satisfactorily; that is, that the structure will not reach a ‘limiting condition’ in prescribed ‘design situations’. The Code therefore provides, in outline, all the general requirements for conducting and checking design. It provides only limited assistance or information on how to perform design calculations and further detail may be required from other texts, such as standard soil mechanics books and industry publications.

BS EN 1997-1 describes the general ‘Principles’ and ‘Application Rules’ for geotechnical design, primarily to ensure ‘safety’ (adequate strength and stability), ‘serviceability’ (acceptable movement and deformation) and ‘durability’ of supported structures, that is of buildings and civil engineering works, founded on soil or rock.

BS EN 1997-1 contains the following Sections:

- Section 1 General
- Section 2 Basis of geotechnical design
- Section 3 Geotechnical data
- Section 4 Supervision of construction, monitoring and maintenance
- Section 5 Fill, dewatering, ground improvement and reinforcement
- Section 6 Spread foundations
- Section 7 Pile foundations
- Section 8 Anchorages
- Section 9 Retaining structures
- Section 10 Hydraulic failure

---

16 ‘Principles’ are mandatory (‘Normative’) requirements; ‘Principle’ clauses in the Code are identified by a ‘P’ after the clause number and contain the word ‘shall’. All other clauses are ‘Application Rules’ that indicate the manner in which the design may be shown to comply with the Principles. Application Rules are ‘Informative’ (i.e. not mandatory and for information only) and use words such as ‘should’ and ‘may’.

17 BS EN 1997-1 may also serve as a reference document for other projects such as the design of bridges, dams, tunnels and slope stabilisation, or the design of foundations for special construction works such as nuclear power plants and offshore structures, which require rules additional to those provided by the Eurocodes.
- Section 11 Site stability
- Section 12 Embankments.

BS EN 1997-1 also contains the nine Annexes described in Table 2.1.

<table>
<thead>
<tr>
<th>Annex</th>
<th>Status</th>
<th>Title</th>
<th>Notes</th>
</tr>
</thead>
</table>
| A     | Normative | Partial and correlation factors for ultimate limit states and recommended values  

Annex A is used with Sections 6 to 12, as it gives the relevant partial and correlation factors, and their recommended values, for ultimate limit state design. Annex A is normative, which means that it is an integral part of the standard and the factors in it must be used, although their values are informative and may therefore be modified in the National Annex (see Section 8). |
| B     | Informative | Background information on partial factors for Design Approaches 1, 2 and 3. | Annex B gives some background information on the three alternative Design Approaches permitted by EN 1990 and given in EN 1997-1 (see Appendix 9.2). |
| C     | Informative | Sample procedures to determine limit values of earth pressures on vertical walls |
| D     | Informative | A sample analytical method for bearing resistance calculation. |
| E     | Informative | A sample semi-empirical method for bearing resistance estimation. |
| F     | Informative | Sample methods for settlement evaluation |
| G     | Informative | A sample method for deriving presumed bearing resistance for spread foundations on rock |
| H     | Informative | Limiting values for structural deformation and foundation movement. |
| J     | Informative | Checklist for construction supervision and performance monitoring. |

Annexes C to G provide examples of internationally recognised calculation methods for the design of foundations or retaining structures; Annexes C to J are informative, which means that in principle, they may be superseded in the National Annex (see Section 8).

The scope of BS EN 1997-1 is compared in Table 2.2 with the current BS geotechnical Codes and Standards that will be withdrawn when the Eurocode suite is fully implemented; Table 2.2 also indicates correspondence with prEN 1997-2 and the so-called ‘execution’ BS ENs that are discussed in Section 6 of this Guide.

---

18 The Annex is ‘informative’ which means that the partial factors listed must be used; however, the values of these factors are a matter for national determination and the values shown in the Annex are thus only ‘recommended’.
### TABLE 2.2 – The content of the European documents and their correspondence with BS Codes

<table>
<thead>
<tr>
<th>BS Code</th>
<th>New European Documents</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS EN 1997-1</td>
<td>prEN 1997-2</td>
</tr>
<tr>
<td>Section - Title</td>
<td>General issues covered</td>
</tr>
<tr>
<td>1. General</td>
<td>Overall Approach</td>
</tr>
<tr>
<td>2. Basis of geotechnical design</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>BS 5930 1999 - Site investigation</strong></td>
<td></td>
</tr>
<tr>
<td>3. Geotechnical data</td>
<td>Ground investigation</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>
2.2. The design philosophy of BS EN 1997-1

In this Section we shall examine the essential features of the design philosophy used in the Eurocode and how it differs from the UK design practices represented in BS Codes. As BS EN 1997-1 does not always follow a logical sequence, this Section does not follow exactly the order of presentation of text in the Code.\(^\text{19}\)

2.2.1 Limit states

BS EN 1997-1 is a ‘limit state design’ code; this means that a design that complies with it will prevent the occurrence of a limit state.\(^\text{20}\) A limit state could, for example, be:

- an unsafe situation
- damage to the structure
- economic loss.

While there are, in theory, many limit states that can be envisaged, it has been found convenient to identify two fundamentally different types of limit state, each of them having its own design requirements:

- ultimate limit states (ULS);
- serviceability limit states (SLS).

Ultimate Limit States

ULSs are defined as states associated with collapse or with other similar forms of structural failure (e.g. failure of the foundation due to insufficient bearing resistance). In geotechnical design, ULSs 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 a ULS even if the ground itself has not reached the limit of its strength.

Ultimate limit states of full ‘collapse’ or ‘failure’ of geotechnical structures are fortunately quite rare. However, an ultimate state may develop in the supported structure because of large displacement of a foundation, which has itself not ‘failed’. This means, for example, that a foundation may be stable, after initially settling (it hasn’t ‘exceeded a ULS’ or ‘failed’), but part of the supported structure may have failed (for example, a beam has lost its bearing and collapsed owing to substantial deformation in the structure). BS EN 1997-1 requires that both possible states are avoided. BS EN 1997-1 distinguishes between the following five different types of ULS and uses acronyms for them:

\(^{19}\) Paragraph numbers in this Guide do not necessarily correspond with clause numbers in the Code. To assist readers to align Guide text with Code text, the numbers of important Code clauses are shown on the right-hand-side of the page.

\(^{20}\) Limit states are defined in BS EN 1990 (Clause 3.1(1)P) as ‘states beyond which the structure no longer satisfies the design performance requirements’. Strictly, it is the exceeding of a limit state that is to be prevented, though BS EN 1997-1 often refers to checking, preventing and avoiding the occurrence of a limit state.
that are shown (in brackets):

- “loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance (EQU);

- “internal failure or excessive deformation of the structure or structural elements, including footings, piles, basement walls, etc, in which the strength of structural materials is significant in providing resistance (STR);

- “failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance (GEO), (e.g. overall stability, bearing resistance of spread foundations or pile foundations);

- “loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions (UPL);

- “hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients (HYD).

These ULSs are not explicitly identified in BS Codes.

For most of the design problems likely to be encountered by readers of this Guide, the STR and GEO ultimate limit states are the ones that will apply, as they cover the routine design of shallow and pile foundations and other ‘common’ geotechnical structures. The EQU ULS is intended to cater for the rare occasion when, for example, a rigid retaining wall, bearing on a rigid rock foundation, could rotate about one edge of its base. The UPL and HYD ULSs, while more common than EQU, are generally beyond the ‘routine’ considerations of this Guide. Consequently, in Section 2.3.2 we shall concentrate on STR and GEO ULS design calculations while the other ULSs are only briefly discussed in Appendix 9.3.

Serviceability Limit States

SLSs are defined as states that correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met (e.g. settlement that is excessive for the purposes of the structure). Section 2.3.3 deals with SLS design.

2.2.2 General design requirements

The Code requires a designer or someone checking the design to perform a number of tasks:

a) establish the ‘design situations’ (see 2.2.3);

b) identify the relevant ‘limit states’;

---

21 An EQU ULS design check could consist of showing that vertical and horizontal force, and moment, equilibrium are not violated.
c) using a choice of methods\textsuperscript{22}, check that the limit states are not exceeded for the intended design.

In carrying out these tasks, the risks associated with the project and its complexity are to be identified. To assist in this, BS EN 1997-1 recommends dividing design projects into Geotechnical Categories (GC) 1, 2 or 3 depending on:

- the purpose of the structure;
- the complexity of the structure, the loading and the ground;
- the level of risk involved;
- previous experience of the particular ground conditions.

In Fig. 2.1 a flow diagram illustrates the stages of geotechnical design in BS EN 1997-1. If the use of GCs is adopted, the categorisation should be checked at stages in the design and construction of the project. The intention is that a design problem should normally receive a preliminary categorisation prior to the site investigation. This should then be checked and possibly changed at each stage of the design and construction processes, as indicated by the asterisks in Fig. 2.1. The procedures of higher categories may be used to justify more economical designs, or where the designer considers them to be appropriate.

Most projects will fall in GC 2, while very simple design problems may be in GC 1, with complex problems falling in GC 3\textsuperscript{23}; Fig. 2.2 is a flow diagram showing the decisions required for categorisation. Examples are given of the types of design project in the three GCs.

As GCs are ‘recommended’ their use is not compulsory. GCs are also used in BS EN 1997 to help establish the extent of site investigation required and the amount of effort to be put into checking that a design is satisfactory.

\textsuperscript{22} The design should be checked to ensure that the relevant limit states are not exceeded by one or a combination of the following methods:

- use of calculations (described in 2.3.1 and 2.3.2);
- the adoption of prescriptive measures (described in 2.3.4), in which a well-established and proven design is adopted without calculation under well-defined ground and loading conditions;
- tests on models or full scale tests (described in 2.3.5), which are particularly useful in the design of piles and anchors;
- the Observational Method (discussed in 2.3.6).

\textsuperscript{23} The Code acknowledges that GC 3 problems should require design procedures beyond its provisions (\textit{clause 2.1(20)}).

---

\textit{Clauses 2.1(10)}

\textit{to 2.1(21)}
Figure 2.1: The design process of EN 1997-1 (the number in brackets, shown in italics, refer to the relevant sections and clauses in BS EN 1997-1)
A brief overview of BS EN 1997 – 1 – Geotechnical design, general rules

Figure 2.2: Flow chart for geotechnical categorisation (after Simpson & Driscoll, 1998)

Category 1

Is the structure small and relatively simple?

Are ground conditions known from comparable local experience to be sufficiently straightforward for routine methods to be used for foundation design and construction?

Are excavation below the water table involved or do comparable local experience indicate that it will be straightforward?

Is there negligible risk in terms of overall stability or ground movements?

Is the structure in an area of probable site instability or persistent ground movements?

Are there unusual or exceptional loading conditions?

Are there unusual or exceptionally difficult ground conditions?

Does it involve abnormal risks?

Is the structure very large or unusual?

Category 2

Yes

No

Yes

No

Yes

No

Yes

No

Yes

No

Yes

No

Yes

No

Yes

No

Category 3

Structures or parts of structures which do not fall within the limits of Geotechnical Categories 1 and 2:

- Spread foundations, raft foundations, pile foundations, walls and other structures retaining or supporting soil or water
- Excavations, bridge piers and abutments, embankments and earthworks, ground anchorages and other structures retaining soil or water
- Tunnels in hard, non-fractured rock not subjected to special water tightness or other requirements.

Is there unusual or exceptionally difficult ground?

Are there unusual or exceptional load conditions?
2.2.3 Design Situations

Design situations are described as:
- persistent (long-term),
- transient (short-term) and
- accidental.

The selected design situation must be sufficiently severe and must cover all conditions which can reasonably be foreseeable during temporary (transient) construction works and the (persistent) use of the structure. Different design situations may involve different limit states.

BS EN 1997-1 presents a list of items for consideration when identifying design situations.

2.3. Geotechnical design methods

BS EN 1997-1 offers one method or a combination of four methods for design and design checking;
- Using calculations for ULS (see 2.3.1) and SLS (see 2.3.3);
- Using ‘prescriptive’ measures (see 2.3.4);
- Using tests (see 2.3.5);
- Using the Observational Method (see 2.3.6).

Each of these is discussed in turn.

2.3.1 Ultimate Limit State design by calculation

The following principles apply:
- the explicit identification of limit states;
- calculation of the destabilising (unfavourable) actions (or their effects) and of stabilising (favourable) actions and resistances using ‘design values’ of variables, these values generally being calculated as the product of a ‘partial factor’ and a ‘characteristic’ value;
- for a particular ULS, the sum of destabilising actions (or their effects) must not exceed the sum of stabilising actions and resistances.

The manner in which these principles apply differs somewhat from those in many of the BS Codes (see Appendix 9.1).

---

24 For an accidental situation involving exceptional circumstances, the structure may be required merely to survive without collapsing; in this case serviceability limit states would not be relevant.
Design calculations in BS EN 1997-1 involve the following processes:

- establishing design values of actions (see Box 2.1);
- establishing design values of ground properties and ground resistances (see Box 2.2);
- defining limits that must not be exceeded (e.g. bearing capacity or values of differential settlement);
- setting up calculation models \(^{25}\) for the relevant ultimate and serviceability limit states;
- showing in a calculation that these limits will not be exceeded.

**BOX 2.1 — Actions**

Actions may be forces (loads applied to the structure or to the ground) and displacements (or accelerations) that are imposed by the ground on the structure, or by the structure on the ground. Actions may be permanent (e.g. self-weight of structures or ground), variable (e.g. imposed loads on building floors) or accidental (e.g. impact loads) \(^{26}\).

Design values of actions \((F_d)\) are calculated using the general equation:

\[
F_d = \gamma_F \times F_{rep}
\]

where

- \(F_{rep}\) is the representative \(^{27}\) (or characteristic \(F_k\)) value of an action (or of the effect of an action);
- \(\gamma_F\) is the partial factor for an action (or \(\gamma_E\), for the effect of an action).

BS EN 1997-1 says that the characteristic values of actions must be derived using the principles of BS EN 1990; the values of those actions that come from the supported superstructure must be taken from BS EN 1991 \(^{28}\).

---

\(^{25}\) For the design situations pertaining, the calculation model is required to predict the resistance (e.g. the bearing capacity of a footing), the effects of the action (e.g. bending moments and shear forces in the footing) and/or the deformations of the ground.

\(^{26}\) The values of the actions from the structure must be taken from BS EN 1991 while BS EN 1997-1 deals with the forces and displacements imposed on structures by the ground and with the resistances offered by the ground. Specific examples of geotechnical actions are given later in Section 4.

\(^{27}\) The characteristic value of an action, \(F_k\), is its main representative value. An action, particularly if it is variable, may have other representative values; for example, when the variable structural actions are combined.

\(^{28}\) At present (December 2005), most Parts of BS EN 1991-1 and BS EN 1991-2 have been published.
BOX 2.2 - Ground properties, actions and resistances

Ground properties must be obtained from the results of tests or from other relevant data such as the back-analysis of settlement measurements or of failures of foundations or slopes\footnote{Section 3 describes the acquisition of ground properties.}.

Design values of ground properties shall either be calculated as

\[ X_d = \frac{X_k}{\gamma_M} \]

where

- \( X_k \) is the characteristic value of a material (ground) property (see Box 2.3);
- \( \gamma_M \) is the partial factor for the material property,

or shall be assessed directly by the designer\footnote{The Code implies that when directly assessed, geotechnical parameters should have design values that will provide a level of safety corresponding to that afforded by the combination of characteristic value and partial factor value (\textit{clause 2.4.6.2(3)}).}.

The design values of destabilising geotechnical actions (or their effects) are calculated using design values of variables, these values being either assessed as the product of a partial factor and a characteristic ground property value (see Box 2.3) or assessed directly.

The design values of stabilising resistances are the product of characteristic resistance values, calculated using characteristic ground properties, and partial resistance factors.

---

BOX 2.3 — Characteristic values of ground properties

BS EN 1997-1 defines the characteristic value as being “\textit{selected as a cautious estimate of the value affecting the occurrence of the limit state considered}”. Each word and phrase in this clause is important:

- \textit{Selected}: emphasises the importance of engineering judgement,
- \textit{Cautious estimate}: some conservatism is required,
- \textit{Limit state considered}\footnote{The characteristic value must relate to the limit state. This is discussed further in Section 3.}.

There has been much debate about the determination of characteristic values of ground properties, including the use of statistics\footnote{The use of statistics has been associated with the determination of characteristic values of material properties; this stems from the widespread application to concrete, for example, where many tests on a relatively homogeneous material make the technique meaningful. Such an approach for the ground is almost always inappropriate since the volume of data will be limited and the material is highly variable. In BS EN 1997-1 the use of statistical methods is not mandatory, is almost always inappropriate and will therefore not be discussed further in this Guide.}. The subject is discussed in greater depth in Section 3.3.
Design values are used in calculations for both ULS and SLS, though the values will usually be different for the two states. The design values required for SLS 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.

2.3.2 Calculations for STR and GEO Ultimate Limit State design
In contrast to eqn. A1 in Appendix 9.1, the basis of ULS design in BS EN 1997-1 lies in the following inequality:

\[ E_d \leq R_d \] (2.5)

where

- \( E_d \) is the design value of the sum of the actions and of the effects of them;
- \( R_d \) is the design value of the corresponding resistance of the ground and/or structure.\(^{33}\)

In BS EN 1997-1, \( E_d \) is obtained from the general expression:

\[ E_d = E \{ \gamma_F F_{rep}; X_k/\gamma_M; a_d \} \] (2.6a)

and \( R_d \) from:

\[ R_d = R \{ \gamma_F F_{rep}; X_k/\gamma_M; a_d \} \] (2.7a)

where:

- \( a_d \) is the design value of a geometrical property (e.g. the depth to a water table).\(^{34}\)

The expressions \( E \{ \ldots \} \) and \( R \{ \ldots \} \), which mean ‘functions involving the terms listed’, may be interpreted in several ways (see Box 2.4).

It can be seen that the term \( X_k/\gamma_M \) introduces into both expressions the influences of ground properties such as strength and weight density\(^{35}\) on geotechnical actions such as earth pressures, and on geotechnical resistances such as the bearing pressure from the ground.\(^{36}\)

\(^{33}\) It is in determining the design values of these two terms that the most fundamental departure from ‘traditional’ geotechnical design practice occurs, at least as embodied in most BS Codes of Practice. This is because, in the Eurocodes, design values are calculated from characteristic values and partial factors, as shown in Boxes 2.1 and 2.2.

\(^{34}\) BS EN 1997-1 introduces the term ‘geometrical data’, with the symbol \( a_d \). Such data would include, for example, the level and slope of the ground surface, water levels, levels of the interface between ground strata, excavation levels and the dimensions of the geotechnical structure.

\(^{35}\) BS EN 1997-1 introduces the term ‘weight density’ in place of ‘unit weight’.

\(^{36}\) Note that, if \( \gamma_F \) is greater than 1.0, the design value of the action exceeds its characteristic value and, if \( \gamma_M \) is greater that 1.0, the design value of the material property will be less than the characteristic value.
BOX 2.4 — Different ways of expressing the effects of actions and resistances

During the development of EN 1997-1 there was much debate about the precise forms of the expressions (2.6a) and (2.7a). In order to reach consensus among the Member States of the EU, the Eurocode adopted three alternative ‘Design Approaches’ (DAs) which use slightly different forms of the expressions; in BS EN 1997-1 only one of these alternatives, DA-1, is likely to be permitted in the National Annex. A brief explanation of the differences in the three Design Approaches is given in Appendix 9.2 while more detailed explanations of the application of DA-1 are given in Section 4.2, with Examples of its use.

2.3.3 Performing Serviceability Limit State design checks

Limit state design requires that the occurrence of a serviceability limit state is sufficiently improbable. Serviceability limit states may be checked in two ways:

- by calculating the design values of the effects of the actions \( E_d \) (e.g. deformations, differential settlements\(^{37}\), vibrations etc.) and comparing them with limiting values, \( C_{d,\text{lim}} \).\(^{38}\)

- by a simplified method, based on comparable experience.

In calculating design values of the effects of actions, forces and the ground resistance properties will normally be equal to their characteristic values, that is the \( \gamma_F \) and \( \gamma_M \) values will be equal to 1.0.\(^{2.4.8(1)}\)

As an alternative to performing serviceability checks using calculations, a simplified method may be used to show that a sufficiently low fraction of the ground strength is mobilised, keeping deformations within the required serviceability limit\(^{39}\). The simplified method is applied in BS EN 1997-1 to spread foundations, to pile foundations and to retaining structures. BS EN 1997-1 gives no indication of what is a ‘sufficiently low fraction of ground strength’.\(^{40}\)

---

\(^{37}\) In cases where differential settlements are calculated, a combination of upper and lower characteristic values of deformation moduli should be considered, to account for any local variations in the ground properties.

\(^{38}\) Ideally, limiting values of deformations, \( C_{d,\text{lim}} \), should be specified as design requirements for each supported structure and BS EN 1997-1 lists (clause 2.4.9(3)) a series of items to take into account when establishing limiting values of movement. It is important that limiting values are established as realistically as possible and in close co-operation with the designer of the supported structure. Unnecessarily severe values usually lead to uneconomic design.

\(^{39}\) This simplified method requires the existence of comparable experience with similar soil and structure. These requirements clearly restrict the circumstances in which the simplified method may be applied to conventional structures and foundations in familiar ground conditions.

\(^{40}\) BS 8002 adopts mobilisation factors ranging from 1.2, for effective stress design calculations, to 3.0 to limit the movement of retaining walls in total stress calculations.
2.3.4 **Design by Prescriptive Measures** 41

Prescriptive measures involve *conventional and generally conservative rules* in the design, and attention to specification and control of materials, workmanship, protection and maintenance procedures.

Prescriptive measures usually involve the application of charts, tables and procedures that have been established from comparable experience; they implicitly contain their own safety factor. Very often, the concept of ‘allowable stress on the soil’ is used in these charts or tables. An example would be the table of minimum widths of strip foundations given in Approved Document A to the Building Regulations. These widths are based on allowable bearing pressures for the different ground conditions cited. The calculations of allowable pressure have an implicit ‘factor of safety’ based on ‘comparable experience’.

Prescriptive measures may be applied, for example, to deal with problems of durability, by specifying additional thickness to prevent the adverse effects of corrosion loss, and to the local prescription of depth of footings to avoid seasonal volume change in clay soil. Prescriptive, conservative procedures are quite common for the routine design of piles in familiar ground conditions.

2.3.5 **Design using load tests and tests on experimental models**

BS EN 1997-1 permits design based on the results of load tests; indeed, as we shall see in Section 4.4, load testing is actively encouraged for the design of piles.

2.3.6 **Design using the Observational Method**

BS EN 1997-1 introduces use of the Observational Method in which a design for the ‘most likely’ circumstances is reviewed in a planned manner during the course of construction and in response to the monitored performance of the work 42.

The Method has a number of important ingredients one of which is a precise plan of monitoring and of any actions required as a result of the monitored performance. The advantage of this method is that it facilitates design where a precise prediction of the geotechnical behaviour is difficult, e.g. where the ground conditions are complex or not sufficiently well known or where time or cost savings may be obtained by reducing temporary works such as the propping of retaining structures in excavations. The Observational Method allows ‘optimistic’ or ‘pessimistic’ assumptions to be checked by monitoring the actual behaviour. A comprehensive explanation of the Observational Method has been provided by Nicholson et al (1999).

---

41 Prescriptive measures can be applied in cases where calculation models are not available or not appropriate. Partial factors are not intended to be used with prescriptive measures.

42 The Observational Method has been defined as ‘a continuous, managed, integrated process of design, construction control, monitoring and review that enables previously defined modifications to be incorporated during or after construction as appropriate’. An essential ingredient is the pre-definition of modifications should they prove necessary.
2.4. The Geotechnical Design Report

Unlike current BS Codes, BS EN 1997-1 requires that assumptions, data, calculations and results of the verification of safety and serviceability must be recorded in a Geotechnical Design Report, relevant parts of which must be provided to the client. The level of detail of Geotechnical Design Reports will vary greatly, depending on the type of design. For simple designs, a single sheet such as that shown in Fig. 2.3 may be sufficient. The report may include a plan of supervision and monitoring, as appropriate.

The two most important parts of the GDR are:

- the Ground Investigation Report (see Section 3);
- a plan for any requisite supervision and monitoring during and after construction\(^\text{43}\) (see Section 5.7).

\(^{43}\) A checklist of items that should normally be included in the GDR is given in BS EN 1997-1 (Clauses 2.8(1)\(P\), 2.8(2), 2.8(3) and 2.8(4)\(P\)).
A brief overview of BS EN 1997 – 1 – Geotechnical design, general rules

Figure 2.3: Simple Geotechnical Design Report (after Simpson & Driscoll, 1998)
3. OBTAINING GEOTECHNICAL DESIGN INFORMATION

3.1 Introduction

The processes for obtaining ground parameters for design using BS EN 1997 are specified in several parts of the suite of European codes and standards. BS EN 1997-1 describes in only general terms the methods to be used for obtaining ground design parameters and it is prEN 1997-2\textsuperscript{44} and its associated Testing Standards\textsuperscript{45} and Test Specifications\textsuperscript{46} that itemise and discuss the methods that are to be used to obtain the basic parameters from which design values are acquired. The lists of published and anticipated Test Standards and Specifications are shown in Tables A4.1, A4.2 and A4.3 of Appendix 9.4 which also shows how these BS ENs and TSs compare in content with BS 5930:1999 and BS 1377:1990.

3.2 Using prEN 1997-2 to obtain ground parameter values from tests

3.2.1 Introduction:

prEN 1997-2 will be used with BS EN 1997-1 to provide requirements and recommendations for geotechnical investigation and testing. These investigations are to include:

- the history of development on and around the site;
- an appraisal of existing constructions;
- the ground investigation.

prEN 1997-2’s primary role is ‘the acquisition, evaluation, assessment and reporting of geotechnical data required for design’ using BS EN 1997-1 and it concentrates on ground investigation.

prEN 1997-2 covers:

- the planning and reporting of ground investigations;
- the obtaining of soil and rock samples;
- the general requirements for a number of commonly used laboratory and field tests,

\textsuperscript{44} Elsewhere in this Guide, reference to specific clauses in BS EN 1997-1 are shown, in italics, in the right-hand margin of the page. In this Section, reference to specific clauses in prEN 1997-2 are shown, again in the right-hand margin, in underlined font.

\textsuperscript{45} Testing Standards specify how tests ‘shall’ be performed.

\textsuperscript{46} Test Specifications give ‘guidance’ on how tests ‘should’ be performed. It is expected that many of the Test Specifications will eventually be converted into Testing Standards.
- the evaluation and interpretation of test results,
- the conversion of test results into derived values of geotechnical parameters and coefficients. A derived value is defined as the value of a geotechnical parameter obtained from test results by theory, correlation or empiricism. An example would be the shear strength obtained through correlation with a $q_c$ value measured in a cone penetration test. Correlations may also use a theoretical relationship to link a geotechnical parameter with a test result, for example when obtaining a value of the angle of shearing resistance $\varphi'$ from pressuremeter test results or from plasticity index.

prEN 1997-2 does not specifically cover environmental investigations and only covers commonly-used field and laboratory tests. The Code states clearly that personnel involved in data collection and processing should be suitably qualified. This topic is also covered in BS EN ISO 22475-2 (see Appendix 9.4, Table A4.2).

It is a principle in prEN 1997-2 that ground investigations must ensure that all relevant data are collected\(^{47}\). Despite these ‘requirements’, most of what is contained in prEN 1997-2 is simply ‘good practice’ and should not put onerous burdens on site investigation practice.

Specific aspects of prEN 1997-2 are now separately discussed.

### 3.2.2 Geotechnical Investigations

BS EN 1997-1 states that *geotechnical investigations shall provide sufficient data concerning the ground and groundwater conditions at and around the site to enable a proper description of the essential ground properties and a reliable assessment of the characteristic values of the geotechnical parameters to be used in design calculations*. Geotechnical investigations shall be performed.

prEN 1997-2 defines the following hierarchy of investigations:
- Geotechnical investigations, which are ground investigations and other information about the site,
- Ground investigations, which are field investigations, laboratory testing and desk studies of geotechnical and geological information, and
- Field investigations, which are direct investigations (drilling, sampling and trial pits) and indirect investigations (in situ tests, such as the CPT).

prEN 1997-2 states that *Ground investigations shall provide a description of ground conditions relevant to the proposed works and establish a basis for the assessment of the geotechnical parameters relevant for all construction stages.*

\(^{47}\)The Code states that ‘Geotechnical investigations shall be planned in such a way as to ensure that relevant geotechnical information and data are available at the various stages of the project. Geotechnical information shall be adequate to manage identified and anticipated project risks. For intermediate and final building stages, information and data shall be provided to cover risks of accidents, delays and damage’.
Within the broad remit of ‘geotechnical investigations’ prEN 1997-2 advocates a phased approach for ground investigation comprising:

- preliminary investigations for the positioning and preliminary design of the structure,
- design investigations,
- controlling and monitoring activities.

prEN 1997-2 states that the provisions in the document are based on the premise that the results from investigations recommended in one phase are available before the next phase is started. However it does not preclude preliminary and design investigations being carried out at the same time.

Preliminary investigations
The preliminary investigation should be planned in such a way that adequate data are obtained to:
- assess the overall stability and general suitability of the site;
- assess the suitability of the site in comparison with alternative sites;
- assess the suitable positioning of the structure;
- evaluate the possible effects of the proposed works on surroundings, such as neighbouring buildings, structures and sites;
- identify borrow areas;
- consider the possible foundation methods and any ground improvements;
- plan the design and control investigations, including identification of the extent of ground which may have significant influence on the behaviour of the structure.

A preliminary investigation would normally include:
- desk studies of geotechnical and geological information about the ground conditions, including reports of previous investigations in the vicinity,
- field reconnaissance (walk-over surveys), and
- consideration of construction experience in the vicinity,

all, as the Code states, leading to:

the following estimates of soil data concerning, if relevant:
- the type of soil or rock and their stratification;
- the groundwater table or pore pressure profile;
- the preliminary strength and deformation properties for soil and rock;
- the potential occurrence of contaminated ground or groundwater that might be hazardous to the durability of construction material.

Design Investigations
prEN 1997-2 states that .....where the preliminary investigations do not provide the necessary information to assess the aspects mentioned .........., complementary investigations shall be performed during the design
**Investigation Phase.** Design investigations are the main geotechnical investigations carried out to obtain the data required for design of both the temporary and permanent works. They are also carried out to provide the information required for construction and to identify any difficulties that may arise during it.

Design investigations normally include field investigations and laboratory testing. Field investigations normally comprise:
- drilling and/or excavations (test pits including shafts and headings) for sampling;
- groundwater measurements;
- field tests, usually in situ.

prEN 1997-2 gives guidance on the planning of the field and laboratory testing. Guidance of the applicability of commonly used field tests is given. Guidance is provided on the numbers and types of samples to be tested in the laboratory.

Design investigations should be carried out at least through all the ground formations that are considered likely to be relevant to the particular design.

**Control Investigations**
Control investigations are inspections carried out during the construction phase to check and compare the actual ground conditions encountered with those assumed in the design.

The general requirements for inspections during construction are presented in BS EN 1997-1 and complemented in prEN 1997-2. They include the need to inspect the ground on a continuous basis and the need to record the results of the inspections.

The results of the investigations must be reported to all relevant personnel (see 3.2.6).

**3.2.3 Soil and rock sampling and groundwater measurements**
Section 3 of prEN 1997-2 covers the taking of samples of soil, rock and groundwater. It requires conformity with BS EN ISO 22475-1 (see Appendix 9.4, Table A4.2) regarding drilling and sampling methods.

**Soil Sampling**
The quality class and numbers of samples to be recovered must be based on the aims of the geotechnical investigation, on the geology of the site and on the complexity of the project.

---

48 Some guidance on the spacing and depth of investigations is provided in prEN 1997-2.
49 Soil samples for laboratory testing are divided into five quality classes with respect to the soil properties that are assumed to remain unchanged during sampling and handling, transport and storage; these quality classes are defined in prEN 1997-2.
Three categories of sampling method are defined in BS EN ISO 22475-1\(^{50}\) and are linked to the sample quality classes. The required sample quality is associated with the laboratory test to be performed on the sample. It is clearly stated that use of a specific sampling method does not guarantee samples of the highest quality class associated with that method. For laboratory strength tests, the highest sample quality is required and this can only be achieved with one of the sampling methods. In effect, prEN 1997-2 controls the methods of drilling and sampling to be adopted to achieve the requisite quality of soil parameter determination in the laboratory.

**Groundwater**

prEN 1997-2 covers only the measurement of pore water pressures that are positive relative to atmospheric pressure. It requires that measurements and any sampling are be conducted in accordance with BS EN ISO 22475-1.

Measurements must be made at a frequency that ensures that variations are properly detected and equipment must be selected and installed to allow this to be done. The Code has specific reporting requirements over and above those in BS EN ISO 22475-1.

### 3.2.4 Field tests in soil and rock

prEN 1997-2 covers in situ testing (see Appendix 9.4, Table A4.1) and how to obtain test results. The tests must be undertaken and reported in accordance with the corresponding Testing Standard of EN ISO 22476 (see Appendix 9.4, Table A4.2)\(^{51}\). The evaluation of test results must consider all available additional ground information.

It is also a requirement that the effects must be considered of all possible geotechnical and equipment influences on the measured parameters. If correlations are used to derive geotechnical parameters or coefficients, the validity and the suitability of any relationships must be evaluated critically. In effect, the evaluation of test results requires justification.

Specific requirements and comments on test result evaluation are given for each test method, followed by examples of the use of the test results, their conversion (if applicable) into derived values and their use for design in BS EN 1997-1\(^{52}\).

### 3.2.5 Laboratory tests in soil and rock samples

prEN 1997-2 imposes requirements for the laboratory storage of samples additional to those in BS EN ISO 22475-1.

For each test or group of tests covered (see Appendix 9.4, Table A4.2),

---

\(^{50}\) It is a requirement that handling, transport and storing of samples shall be carried out in accordance with BS EN ISO 22475-1.

\(^{51}\) If a Technical Specification, rather than a Test Specification, applies then the test procedure should fulfill its general requirements.

\(^{52}\) Examples of correlations for obtaining derived values are contained in Informative Annexes (see Appendix 9.4, Table A4.1).
specific requirements are made and discussed. These may apply to test measurements, their interpretation into test results and any processing into derived values.

3.2.6 The Ground Investigation Report

prEN 1997-2 makes compulsory the provision of some form of Ground Investigation Report (GIR) as part of the Geotechnical Design Report (GDR). The Code specifies that the GIR must include:

- a presentation of all available geotechnical information including geological features and relevant data;
- a factual account of all field and laboratory investigations;
- a geotechnical evaluation of the information, stating the assumptions made in the interpretation of the test results;
- a statement of methods adopted (citing the relevant standards);
- all relevant information on how derived values were determined, including any correlations used;
- any known limitations in the results.

The GIR should propose any necessary further field and laboratory investigations, with comments justifying the need for the work. Such proposals should be accompanied by a detailed programme for the further investigations to be carried out.

Once the values of ground parameters required for the design of the project have been acquired using prEN 1997-2 and its associated documents, attention turns back to BS EN 1997-1 in which this basic ground information is turned into values for use in the design itself (see Fig. 3.1)

3.3 Characteristic values of geotechnical parameters

One of the biggest changes for UK practice entails the adoption in BS EN 1997-1 of 'characteristic values' of ground properties; for this reason, the selection and use of characteristic values requires further explanation.

BS EN 1997-1 states that Design values of geotechnical parameters \( (X_d) \) shall either be derived from characteristic values using the following equations:

\[
X_d = \frac{X_k}{\gamma_d}
\]

or shall be assessed directly.

BS EN 1997-1 says only the following about the direct assessment of design values: If design values of geotechnical parameters are assessed directly, the values of the partial factors recommended in Annex A should be used as a guide to the required level of safety.
While the Eurocode permits the direct assessment of a design value it clearly places priority on the determination of design values from characteristic values. This Section of the Guide therefore explains how characteristic values are obtained (using derived values and other relevant information) — see Box 3.1.

**BOX 3.1 — Characteristic value**

The introduction of *characteristic value* has been one of the most controversial topics in the process of drafting BS EN 1997-1.

The importance and controversy of characteristic values stems from the fact that priority is given to applying partial factors to characteristic values in order to obtain suitably safe but economical design values of parameters. Therefore, and notwithstanding the alternative of direct assessment highlighted above, as the values of the partial factors are specified in the National Annex to BS EN 1997-1, so the selection of the characteristic value is the main point in calculations at which engineers are to apply their skills and judgment, with the possibility of dangerous or uneconomic mistakes (Authors’ italics).

The processes of determining design values for the geotechnical parameters, from basic laboratory and/or field test measurements, through the application of any correlations to produce derived values of test results and then to the selection of characteristic values to which partial factors are applied, is shared by both prEN 1997-2 and BS EN 1997-1, as illustrated in Fig. 3.1a; more detail of this is shown in the two steps depicted in Fig. 3.1b:

— Step 1: establish the values of the appropriate ground properties;
— Step 2: from Step 1, select the characteristic value, including all relevant, complementary information.

Most of the activities involved in Step 1 are covered by prEN 1997-2 and have been discussed in Section 3.2. In what now follows, we deal with Step 2.

---

53 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 obtain appropriate values from site investigation and other information. In particular, the degree of conservatism necessary in choosing values for design purposes is rarely discussed.
Figure 3.1a - General framework for the selection of derived, and hence design, values of geotechnical properties.
STEP 1
Covered by:
BS EN 1997-1,
Clauses 2.4.3,
3.3 and
prEN 1997-2
and testing
standards

**Measured Values**

Test related correction, independent of any further analysis

**Test Results**

Results of field tests at particular points in the ground or locations on a site or laboratory tests on particular specimens

Selection of relevant test results e.g. peak or constant volume strengths

Theory, empirical relationships or correlations to obtain **Derived values**

- Choice of an appropriate correlation when using standard tables relating parameters to test results
- Assessment of influence of test and design conditions on parameter value. Calibration and correction factors applied to relate the parameter to actual design situation and to account for correlations used to obtain derived values from test results; e.g.
  - factor to convert from axi-symmetric to plane strain conditions
  - Bjerrum’s correction factor for $c_u$ value obtained from a field vane test
- Relevant published data and local and general experience

**Geotechnical Parameter Values**

Quantified for design calculations

Cautious estimate of geotechnical parameter value taking account of:

- Number of test results
- Variability of the ground
- The scatter of the test results, e.g. application of $\xi$ factors to pile test results
- Particular limit state and volume of ground involved
- Nature of the structure, its stiffness and ability to redistribute loads

STEP 2
Covered by
BS EN 1997-1,
Clause 2.4 5.2

**Characteristic Parameter Value**

Figure 3.1b - General procedure for determining characteristic values from measured values
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. Field and laboratory tests are to be used, but they are to be complemented by well-established experience. The resulting value is inevitably subjective to some extent, being influenced by the knowledge and experience of the designer. In many situations, the known geology of a stratum, and existing experience of it, give a fairly good indication of its parameter values, with soil tests being used as a check.

In selecting the characteristic value, account must be taken of a number of matters that are listed in BS EN 1997-1.

Having taken due consideration of all these issues, BS EN 1997-1 defines 'characteristic value' as follows: 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.

Each word and phrase in this definition is important:

- selected: emphasises the importance of engineering judgement,
- cautious estimate: some conservatism is required,
- limit state considered: The selected value must relate to the limit state and mode of possible failure.

3.3.1. Characteristic values depend on failure mode
To illustrate how the words affecting the occurrence of the limit state might apply, consider that the characteristic value of a parameter in one ground stratum is not necessarily the same for two different limit states. It may depend on the extent to which a particular failure mode averages out the variable properties of the stratum, as illustrated in Box 3.2.

3.3.2. Other attempts to express uncertainty in ground parameter values
It is helpful to compare the treatment of ‘conservative’ ground parameters in BS EN 1997-1 with approaches in other geotechnical design codes and guidance.

CIRIA Report 580 (Gaba et al, 2003) suggests that design may be based on moderately conservative values of parameters; ‘moderately conservative’ is defined as ‘a cautious estimate of the value relevant to the occurrence of the limit state’. The report states that ‘it is considered to be equivalent to representative values as defined in BS 8002 (1994) and to characteristic values as defined in EC7’.

In BS 8002, design values of soil strength (i.e. values entered into calculations) are derived by factoring ‘representative’ values. A representative value is defined as ‘a conservative estimate of the mass strength of the soil’. ‘Conservative values’ are further defined 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.

**BOX 3.2 — Characteristic value and failure mode**

Figure 3.2 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 designer could, for designing the footings, assume that all of them are founded on clay, the most adverse circumstance. When the designer considers the overall stability of his building, and hence the possibility of a slope failure along the large slip surface illustrated, it seems inconceivable that this surface will lie entirely, or even mainly in clay. It can therefore be seen that, in this example, there could be more than one characteristic value for strength parameters of the same site, with a selection for the footing design that is different from that for the slip surface.

### 3.3.3. Significance of statistical methods

BS EN 1997-1 alludes to the employment of statistical approaches to the selection of characteristic values (see Appendix 9.5). However, for the majority of projects the use of statistics will not be appropriate. The exception may be where a large amount of high-quality ground investigation data is available.

### 3.3.4. Characteristic values of stiffness and weight density

BS EN 1990 states that *The structural stiffness parameters (e.g. moduli of elasticity ……) should be represented by a mean value.*

The context of this definition is 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

---

54 Alternatively, he could require an inspection and probing at each footing location, so avoiding this most conservative assumption.
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 margin for uncertainty. In design practice, engineers rarely take this approach, preferring to make a more pessimistic estimate when there is significant uncertainty.

BS EN 1997-1 therefore uses the same definition of characteristic value for stiffness as for strength. That is, a ‘cautious estimate’, not a mean value.

BS EN 1997-1’s definition of characteristic value also applies to the weight density of soil and rock. However, the uncertainty about weight density 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.
4. DESIGN CALCULATIONS FOR FOUNDATIONS AND RETAINING STRUCTURES

4.1. Introduction

BS EN 1997-1 permits several methods of design, not all of which entail performing detailed calculations, such as prescriptive measures, the use of model test results and the application of the Observational Method (see Section 2.3.6). In this Section, we examine the basis for design calculation as set out in the Code and then how it is applied in straightforward foundation design (for the GEO and STR ultimate limit states) and for SLS design.

4.2. Using Design Approach 1 for GEO and STR ULS calculations

We have seen in Section 2.3.2 that BS EN 1997-1 will adopt DA-1 in which the two fundamental expressions

\[ E_d = E \{\gamma_F F_{rep} ; X_k / \gamma_M ; a_d\} \]  \hspace{1cm} (2.6a)  

and

\[ R_d = R \{\gamma_F F_{rep} ; X_k / \gamma_M ; a_d\} \]  \hspace{1cm} (2.7a)

are used to satisfy the inequality \( E_d \geq R_d \) \hspace{1cm} (2.5)

(Note that a further expression (2.7b)

\[ R_d = R \{\gamma_F F_{rep} ; X_k ; a_d\}/\gamma_R \]

is used in DA-1 for piles and anchorages. This applies a resistance factor \( \gamma_R \) rather than a material factor).

In principle, DA-1 requires two separate calculations to be performed in which the GEO and STR ULSs are examined using two different combinations of sets of partial factors\(^{55}\). These combinations are now discussed, using the symbolic notation explained in Appendix 9.2. The values recommended for all the partial and other factors used in EN 1997-1 are contained in the Tables in Annex A. As many of the values in these 17 Tables have no relevance for DA-1, the factor values for DA-1, for the GEO and STR ULSs, have been assembled in the single Table 4.1 below, for simplicity and clarity.

---

\(^{55}\) The requirement for two separate calculations, in principle, to be performed for all designs involving the ground and man-made structures has its origins in the early stages of development of the structural Eurocodes and the integration of geotechnics and structural engineering into a common design methodology. A fuller explanation may be found in Simpson & Driscoll, 1998.
### Table 4.1: Values of partial factors recommended in BS EN 1997-1

#### Annex A

<table>
<thead>
<tr>
<th>Design Approach 1</th>
<th>Combination 1</th>
<th>Combination 2</th>
<th>Piles &amp; Anchors, Combination 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A1</td>
<td>M1</td>
<td>R1</td>
</tr>
<tr>
<td></td>
<td>A2</td>
<td>M1</td>
<td>R1</td>
</tr>
<tr>
<td></td>
<td>A2</td>
<td>M1</td>
<td>or M2</td>
</tr>
</tbody>
</table>

#### Design Approach

- **Combination 1**
  - Permanent
  - Variables:
    - \( \gamma_G \)
    - \( \gamma_Q \)
    - \( \gamma_{\phi} \)
    - \( \gamma_c' \)
    - \( \gamma_{qu} \)
  - Factors:
    - Weight density
    - Unit weight in tension (compression)
    - Total/combined (compression)
    - Shaft in tension (compression)
    - Base
- **Combination 2**
  - Variables:
    - \( \gamma_R \)
  - Factors:
    - Permanent
    - Temporary

#### Table Notes

- Values of partial factors recommended in BS EN 1997-1
- \( \gamma \) is the factor of safety in the equation: \( F = \gamma \cdot R \)

#### Design Approach 1

- **Combination 1**
  - Permanent
  - Variables:
    - \( \gamma_G \)
    - \( \gamma_Q \)
    - \( \gamma_{\phi} \)
    - \( \gamma_c' \)
    - \( \gamma_{qu} \)
  - Factors:
    - Weight density
    - Unit weight in tension (compression)
    - Total/combined (compression)
    - Shaft in tension (compression)
    - Base
- **Combination 2**
  - Variables:
    - \( \gamma_R \)
  - Factors:
    - Permanent
    - Temporary

### Table 4.1

Values of partial factors recommended in BS EN 1997-1. Appendix A
4.2.1. **Combination 1**
This is expressed symbolically as:

\[ A_1 \text{“+” } M_1 \text{“+” } R_1 \]

where:
- the symbol ‘A’ represents the sets of partial factors for the actions \( \gamma_F \) or for the effects of actions \( \gamma_E \);
- the symbol ‘M’ represents the sets of partial factors \( \gamma_M \) for strength (material) parameters of the ground.
- the symbol ‘R’ represents the sets of partial factors for resistance \( \gamma_R \) (expression 2.6b).
- the symbol “+” means ‘used in combination with’.

Table 4.1 (Set A1) indicates \( \gamma_G = 1.35 \) and \( \gamma_Q = 1.5 \). Calculations of actions and resistances from the ground are performed using design values of ground properties equal to their characteristic values since Table 4.1 (Set M1) indicates \( \gamma_{\phi} = \gamma_c = \gamma_{cu} = \gamma_{qu} = 1.0 \) while it can be seen that, in Combination 1, caution is applied primarily to the structural actions.

4.2.2. **Combination 2**
This Combination covers the case in which caution is applied primarily to ground properties.

Combination 2 is expressed symbolically as:

\[ A_2 \text{“+” } M_2 \text{“+” } R_1 \]

in which the permanent actions from the structure are at their representative values \( (\gamma_G = 1.0, \text{ Set A2, Table 4.1}) \) while any unfavourable, variable actions from the structure are increased, by a relatively small amount, above their representative value \( (\gamma_Q = 1.3, \text{ Set A2, Table 4.1}) \). Calculations of actions and resistances from the ground are performed using design values of ground properties below their characteristic values \( (\gamma_{\phi} = 1.25, \gamma_c = \gamma_{cu} = 1.4, \text{ Set M2, Table 4.1}) \), while any contribution from the weight of the ground is at its characteristic value \( (\gamma_G = 1.0, \text{ Set M2, Table 4.1}) \), since there is less uncertainty about ground weight density than there is about ground strength.

Since in DA-1 actions and resistances from the ground are generally calculated using factored ground material properties (set M) and not from geotechnical

---

56 Note that, while the generic symbol for partial factors for actions and the effects of actions are, respectively, \( \gamma_F \) and \( \gamma_E \), \( \gamma_G \) and \( \gamma_Q \) are the specific symbols for permanent and variable actions, respectively. Further, the factors for favourable, stabilising actions are represented thus: \( \gamma_{G;stb} \), while the factors for unfavourable, destabilising actions have the symbol \( \gamma_{G;dst} \).

57 Uncertainty about how well the calculated ground resistance models real behaviour is also assumed to be covered by the partial factor values of Combination 2.
element resistances, the resistance factors, $\gamma_R$, (set R1) are not used. However, as we shall see later, the design of piles and anchorages is a notable exception to this general rule.

Where it is obvious that one Combination will govern the design, it is not necessary to perform calculations for the other — see Box 4.1.

**BOX 4.1 — Application of Combinations 1 & 2**

In many circumstances, the *dimensions* of a foundation are determined from the Combination 2 calculation, while the structural design (i.e. the bending moments and shear forces) is determined using the Combination 1 calculation with the dimensions found from Combination 2. A suggested, initial procedure is firstly to perform the Combination 2 calculation to find the size of the sub-structure and then to check that the strength of the resulting structural element (e.g. square pad footing, or cast-in-situ concrete pile) is satisfactory to carry the internal forces and moments found using Combination 1. Obviously, where the *structural* strength of the foundation is not in question, this second step will be unnecessary.

Which of the two Combinations will prove critical may not always be obvious. In both Examples 4.1 and 4.2, in which a square pad is subjected firstly to a vertical load and secondly to vertical and horizontal loads, Combination 2 proves to be critical to the dimensioning of the pad. However, an increase in the eccentricity of loading as the horizontal, variable load increases relative to the vertical load would be found to result in Combination 1 becoming critical.

For this reason, *in principle*, BOTH Combinations require checking, though a designer will, with experience, know that calculation using only one Combination will suffice for a straightforward design problem.

The manner in which DA-1 is used in STR and GEO ULS calculations is now described for simple foundation design problems. Also discussed are means of satisfying the SLS requirements.

### 4.3. Spread foundations

#### 4.3.1. Overall Stability

BS EN 1997-1 requires a check of the overall stability of the ground mass both beneath and adjacent to the foundation itself. Figure 4.1 shows that the potentially unstable ground may contain the foundation (failure surface A-B) or may pass close to it (C-D). Since failure along C-D may substantially affect the bearing capacity of the foundation, failure along C-D should be as ‘sufficiently improbable’ as bearing failure of the footing itself (see Section 5.2 for a discussion of slope stability).

#### 4.3.2. Design of the foundation

The Code requires either of two calculation methods to be adopted:

- a) A *direct* method which involves two separate processes:
  - firstly, a ULS calculation using ground properties;
  - secondly, a settlement calculation to check the SLS requirements;

- b) An *indirect* method in which a single calculation is based on comparable experience (an essential prerequisite), and which uses the results of field or

---

*Clause 2.4.3.2(2)*

The manner in which DA-1 is used in STR and GEO ULS calculations is now described for simple foundation design problems. Also discussed are means of satisfying the SLS requirements.

### 4.3. Spread foundations

#### 4.3.1. Overall Stability

BS EN 1997-1 requires a check of the overall stability of the ground mass both beneath and adjacent to the foundation itself. Figure 4.1 shows that the potentially unstable ground may contain the foundation (failure surface A-B) or may pass close to it (C-D). Since failure along C-D may substantially affect the bearing capacity of the foundation, failure along C-D should be as ‘sufficiently improbable’ as bearing failure of the footing itself (see Section 5.2 for a discussion of slope stability).

#### 4.3.2. Design of the foundation

The Code requires either of two calculation methods to be adopted:

- a) A *direct* method which involves two separate processes:
  - firstly, a ULS calculation using ground properties;
  - secondly, a settlement calculation to check the SLS requirements;

- b) An *indirect* method in which a single calculation is based on comparable experience (an essential prerequisite), and which uses the results of field or

---

*Clause 6.5.1*

The manner in which DA-1 is used in STR and GEO ULS calculations is now described for simple foundation design problems. Also discussed are means of satisfying the SLS requirements.

### 4.3. Spread foundations

#### 4.3.1. Overall Stability

BS EN 1997-1 requires a check of the overall stability of the ground mass both beneath and adjacent to the foundation itself. Figure 4.1 shows that the potentially unstable ground may contain the foundation (failure surface A-B) or may pass close to it (C-D). Since failure along C-D may substantially affect the bearing capacity of the foundation, failure along C-D should be as ‘sufficiently improbable’ as bearing failure of the footing itself (see Section 5.2 for a discussion of slope stability).

#### 4.3.2. Design of the foundation

The Code requires either of two calculation methods to be adopted:

- a) A *direct* method which involves two separate processes:
  - firstly, a ULS calculation using ground properties;
  - secondly, a settlement calculation to check the SLS requirements;

- b) An *indirect* method in which a single calculation is based on comparable experience (an essential prerequisite), and which uses the results of field or
laboratory measurements or other observations and SLS loads. This method implicitly covers the ULS provided that the comparable experience involves similar structures and that the loads are not exceptional.

Figure 4.1 - Potentially unstable slope supporting a foundation

Direct calculation method for ULS design.
The fundamental ULS requirement is represented by the inequality:

\[ E_d \leq R_d \quad (2.5) \]

In the simple case illustrated in Fig. 4.2,

\[ E_d = V_d, \text{ the ULS design load normal to the foundation} \]

and

\[ R_d \quad \text{is the design bearing resistance of the foundation to loads normal to it.} \]

\[ V_d \text{ includes the weight of the foundation and of any backfill material (regarded as a 'structural action') placed on it.} \]

\[ R_d \text{ may be calculated using analytical or semi-empirical formulae. } Annex \ D \text{ of BS EN 1997-1 provides widely-recognised formulae for bearing resistance}\] 

\[ 58 \text{ Note that } Annex \ D \text{ is 'informative' and could be superseded in the National Annex; in fact, the NA to BS EN 1997-1 is not expected to recommend that alternative formulae are used.} \]
Figure 4.2 — Simple pad footing loaded vertically at its centroid.

**Indirect Method for combined ULS and SLS design.**
A typical indirect, empirical method would be based on the results of a field test. While EN 1997-1 adopts an approach using a pressuremeter test not commonly encountered in the UK (see Annex E of BS EN 1997-1), other, more common indirect methods could be based on the results of the SPT or CPT\(^5\)\(^9\).

**ULS Design - Prescriptive Method**
As has been mentioned, presumed bearing pressures are prescribed, for simple strip footings, for example, in Tables in the Building Regulations. These pressures also meet the SLS requirements.

**SLS Design and Settlement**
As has been said, BS EN 1997-1 says little about how to calculate foundation and ground movements beyond offering a sample method for calculating immediate and consolidation settlements. It also offers some limited guidance on acceptable levels of structural deformation.

**Structural Design**
Similarly, BS EN 1997-1 says little about the determination of shear forces and bending moments for use in section design using the BS EN for the relevant structural material (e.g. BS EN 1992 for reinforced concrete).

In the following examples of the use of BS EN 1997-1 for the design of shallow foundations, Example 4.1 illustrates how the calculations are performed for a pad foundation subject to a vertical, external load, while Example 4.2 illustrates the same footing with vertical and horizontal external loads.

---

\(^5\) Results from the standard penetration test (SPT) have been related to the allowable bearing pressure (assuming a 25mm settlement) for footings on sand (see Tomlinson, 2001). Results from the cone penetration test (CPT) have been similarly used (see BRE, 2003).
Spread Foundations
Pad Foundation – Clay Soil
Vertical Load Only

\[ γ_{s,k}, \text{ characteristic weight density of soil} – 20kN/m^3 \]
\[ γ_{c,k}, \text{ characteristic weight density of concrete} – 24kN/m^3 \]
\[ c_{uk,k}, \text{ characteristic value of undrained shear strength} – 60kPa \]

**Bearing Resistance**

**Purpose:** To determine a width of footing, B, that meets the Bearing Resistance requirements

**Required:**
\[ V_d ≤ R_d \]

\[ V_d – \text{Design value of the vertical load (action)} \]
\[ R_d – \text{Design value of resistance to vertical load (action)} \]
**1st Trial** Assume 2m x 2m pad footing

**Permanent, vertical characteristic Load (actions)**

1. Imposed vertical load on column \(500\text{kN}\)
2. Weight of Foundation:
   - Wt. of rising column \((1 \times 0.5 \times 0.5 \times 24)\) \(6\text{kN}\)
   - Wt. of foundation pad \((2 \times 2 \times 0.5 \times 24)\) \(48\text{kN}\)
   - Wt. of backfill \(((2 \times 2 \times 1) - (0.5 \times 0.5 \times 1)) \times 20\) \(75\text{kN}\)

Total Characteristic (actual) Permanent Load
\[V_k = 629\text{kN}\]

**Design Approach 1**

Comment: The code offers three alternative procedures to assess bearing resistance. The most usual approach is the analytical method.

**Analytical Method**

Undrained Conditions
\[R/A = (\pi + 2) c_u s_c + q\]

Note: Simplified for the case of a vertical load action at the centre of pad with no inclination.

Shape factor
- Square footing, \(s_c = 1.2\)

Area of footing
- Pad 2m x 2m, \(A = 4\text{m}^2\)
Bearing Resistance
\[ R = 4[(\pi + 2) \times 1.2cu + q] \] (2)

Combination 1  A1 “+” M1 “+” R1

Design Load (A1)
\[ V_{d1} = \gamma_G \times V_k \]
\[ V_{d1} = 1.35 \times 629 \]
\[ V_{d1} = 849kN \] (3)

Design Strength (M1)
\[ c_{ud1} = \frac{c_{uk}}{\gamma_{cu}} \]
\[ c_{ud1} = 60/1.0 \]
\[ c_{ud1} = 60kPa \] (4)

Soil Surcharge, Design value adjacent to footing (q_{d1}) (A1)
\[ q_{d1} = q_k \times \gamma_G \]
\[ q_{d1} = (1.5 \times 20) \times 1.0 \]
\[ q_{d1} = 30kPa \] (5)

Design Bearing Resistance (R1)
\[ R_{d1} = R_k/\gamma_{R,V} \]
\[ R_{d1} = R_k/1.0 \]
\[ R_{d1} = R_k \] (6)

From Eqn 2
\[ R_{d1} = 4[(\pi+2) \times 1.2 \times 60 + 30] \]
\[ R_{d1} = 1601kN \] (7)

Check if \( V_{d1} \leq R_{d1} \)
From Eqn 3 and Eqn 7
849kN < 1601kN

Footing 2m x 2m acceptable for Design Approach 1, Combination 1
### Combination 2 A2 “+” M2 “+” R1

**Design Load (A2)**

\[
V_{d2} = 1.0 \times 629
\]

\[
V_{d2} = 629
\]

**Design Strength (M2)**

\[
c_{ud2} = \frac{60}{1.4}
\]

\[
c_{ud2} = 43\text{kPa}
\]

**Soil Surcharge, Design Value (A2)**

\[
q_{d2} = q_k \times \gamma_G
\]

\[
= (1.5 \times 20) \times 1.0
\]

\[
q_{d2} = 30\text{kPa}
\]

**Design Bearing Resistance (R1)**

\[
R_{d2} = \frac{R}{R\gamma_V}
\]

\[
= \frac{R}{1.0}
\]

From Eqn 2 and above (Eqns 9 & 10)

\[
R_{d2} = 4[\pi \times 1.2 \times 43 + 30]
\]

\[
R_{d2} = 1181\text{kN}
\]

**Check** if \( V_{d2} \leq R_{d2} \)

From Eqn 8 and Eqn 11

\[
629 < 1181
\]

Footing 2m×2m acceptable for Design Approach 1, Combination 2.

**Comment:**

1. It is apparent that the pad could be made smaller and still comply with the code requirements for bearing resistance, however consider the serviceability condition (acceptable settlement)

2. Note that Comb. 2 is very slightly more critical than Comb. 1 since \( \frac{R_{d2}}{V_{d2}} \) (1.88) is less than \( \frac{R_{d1}}{V_{d1}} \) (1.9)

---

**Table A3**

<table>
<thead>
<tr>
<th>Permanent Unfavourable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table A3 (Undrained shearing strength)</td>
</tr>
<tr>
<td>Table A3 (Permanent Favourable)</td>
</tr>
<tr>
<td>Table A5 (RI Bearing)</td>
</tr>
</tbody>
</table>

---

2.4.7.3.4.2
**Settlement**

Comment: For a ‘straightforward’ design case two options are open to the designer:

(a) Develop a safe design without the need for settlement analysis, using the following equation:

\[ V_k \leq \frac{R_k}{3} \]

(b) Undertake a settlement analysis considering both immediate and consolidation settlement.

**Settlement Option (a)**

Note: ‘Classic’ bearing capacity analysis

From Eqn 1

\[ V_k = 629kN \]

From Eqn 6

Bearing Capacity, \( R_{d1} \) is effectively unfactored

\[ \gamma_{R\cdot V} = 1.0 \] (bearing resistance)

\[ \gamma_G = 1.0 \] (surcharge)

\[ \gamma_{cu} = 1.0 \] (shear strength)

\[ R_k = R_{d1} \]

\[ R_k = 1601kN \]

Check

\[ \frac{R_k}{V_k} = \frac{1601}{629} = 2.55 \text{ (<3)} \]

Increase footing size. Try pad 2.5m × 2.5m

**Revision of:**

**Vertical Characteristic Load (actions)**

1. Imposed vertical load on column 500kN
2. Weight of Foundation:
   - Wt. of rising column 6kN
   - Wt. of foundation pad 75kN
Wt. of backfill

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>120kN</td>
<td>201kN</td>
</tr>
</tbody>
</table>

Revised Total Characteristic (actual) Permanent Load

\( V_k = 701kN \)

Bearing Resistance

From Eqn 2

\[ R_k = 2.5 \times 2.5 \left( \pi + 2 \right) \times 1.2 \times 60 \times 30 \]

\[ R_k = 2501 \text{kN} \]

Check, using Eqn.12

\[ \frac{R_k}{V_k} = \frac{2501}{701} = 3.56 (>3) \]

Conclusion: Use a footing 2.5m \( \times \) 2.5m without settlement analysis being undertaken.

Note: This option could be refined so that \( R_k / V_k \approx 3 \)

Settlement Option (b)

Undertake settlement analysis, using a pad footing 2m \( \times \) 2m.

(i) Adjusted elasticity method

Parameters

Drained modulus, \( E' = 12 \text{MN/m}^2 \)  
\( \text{Eqn F.1} \)

Poisson's Ratio, \( \nu = 0.2 \) (stiff clay)

Clay layer (z) is 10m thick below the base of the footing.

Method

Note: The engineer may choose any suitable method. This example uses: Butler F.G (1974). Heavily overconsolidated clays.

COSOS. April. Pentech Press.

Formula

\[ \delta = \frac{1}{2} q B / E \]

(Similar to \( s = p.b.f/E_m \))
Butler’s charts are used to determine settlement at the corner of a footing

\[ \delta_{\text{centre}} = 4 \times \delta_{\text{corner}} \]

where \( B_{\text{corner}} = \frac{1}{2} B_{\text{centre}} \)

From charts

For \( z/b = 10, L/B = 1, \nu = 0.5 \)

\[ I = 0.37 \]

For \( z/b = 10, L/B = 1, \nu = 0.1 \)

\[ I = 0.5 \]

For \( \nu = 0.2, \) by interpolation

\[ I = 0.47 \]

**Settlement**

From Eqn 14

\[ \delta_{\text{corner}} = 0.47 \times \left( \frac{683}{4} \right) \times \frac{1}{12} \]

\[ = 6.7 \text{ mm} \]

\[ \delta_{\text{centre}} \text{ (settlement of pad)} \]

\[ = 4 \times 6.7 = 27 \text{ mm} \]

**Depth Correction Factor**

As the footing is 1.5 m below ground level a depth correction factor is used. The engineer may choose any suitable method. The method chosen for this example is:

Fox E.N (1948). The mean elastic settlement of a uniformly loaded area at a depth below the ground surface. Proc. 2\textsuperscript{nd} I CSM. Rotterdam. Vol.1.p.129–132.

\[ D = \text{depth of footing} = 1.5 \text{m} \]

\[ a = \text{width of footing} = 2 \text{m} \]

\[ b = \text{length of footing} = 2 \text{m} \]

\[ D/\sqrt{ab} = \frac{1.5}{2} = 0.75 \]

\[ a/b = 1 \]
From chart

Depth correction factor, $\mu = 0.78$

Settlement = $\mu \times \delta_{\text{centre}}$

$= 0.78 \times 27$

Settlement = 21mm.

Note: Overall settlement is often not of direct relevance. Frequently the concern is focused on displacements subsequent to the application of finishes and facades. However a figure less than 25mm is often used as a fail safe value for overall movement.

Pad 2m x 2m is acceptable

(ii) Settlements caused by consolidation

Parameters

Coefficient of volume compressibility, $M_v = 0.1 \text{ m}^2 / \text{MN}$

Clay layer is 10 m thick.

Formula

Settlement $\delta_{\text{oed}} = M_v \int_0^H \Delta p \cdot h$

$\Delta p$ – Increase in vertical stress of a soil layer

$h$ – Thickness of the each soil layer

$H$ – Overall depth of soil beneath footing

Examine 5 layers, each 2m thick, below the footing.

Evaluation of increase in vertical stress at the centre of each layer

Using chart contained in:


Note: The engineer may choose any other suitable method.

$$Z = 1 \text{m}$$

$Z/B = 1/2 = 0.5 \quad \rightarrow \quad \Delta p / q = 0.7$

$Z/B = 3/2 = 1.5 \quad \rightarrow \quad \Delta p / q = 0.18$

$Z/B = 5/2 = 2.5 \quad \rightarrow \quad \Delta p / q = 0.07$
Comment: As the calculation is based on increases in stress, the analysis can be approximated to only consider the structural load being applied i.e. 500 kN

Settlement
\[ \delta_{\text{oed}} = 0.1 \times \frac{500}{4} \times (0.7 + 0.18 + 0.07 + 0.04 + 0.02) \times 2 \]
Settlement = 25mm

Comment. Commonly accepted practice is to adopt the calculated oedometer settlement as the sum of both immediate and consolidation settlement for a heavily overconsolidated clay e.g. London Clay.

Applying the same logic as Option (b.1)

Pad 2m × 2m is acceptable
**Spread Foundations**

Pad Footing – Clay Soils

Vertical and Horizontal Loading

\[ \gamma_{s,k} \text{ characteristic weight density of soil} = 20\text{kN/m}^3 \]
\[ \gamma_{c,k} \text{ characteristic weight density of concrete} = 24\text{kN/m}^3 \]
\[ c_{u,k} \text{ characteristic value of undrained shear strength} = 60\text{kPa} \]
\[ H_k \text{ characteristic variable horizontal load} = 75\text{kN} \]

Note: For clarity in the calculations a footing size of 2.2m x 2.2m is adopted. A 2m x 2m footing does not work for sliding resistance.

Bearing Resistance

Permanent, vertical characteristic load
(1) Imposed vertical load on column \( 500\text{kN} \)
(2) Weight of Foundation
- Wt. of rising column \((1 \times 0.5 \times 0.5 \times 24)\) \(6\text{kN}\)
- Wt. of foundation pad \((2.2 \times 2.2 \times 0.5 \times 24)\) \(58\text{kN}\)
- Wt. of backfill \((2.2 \times 2.2 \times 1) - (0.5 \times 0.5 \times 1)\) \(92\text{kN}\)

Total Characteristic Load \( V_k = 656\text{kN} \)

**Combination 1** A1 “+” M1 “+” R1
Applied Loads (Action A1)

Vertical Design Load
\[ V_{d1} = \gamma_0 \times V_k \]
\[ V_{d1} = 1.35 \times 656 \]
\[ V_{d1} = 886 \text{ kN} \]

Variable, horizontal design load
\[ H_{d1} = \gamma_Q \times H_k \]
\[ H_{d1} = 1.5 \times 75 \]
\[ H_{d1} = 112.5 \text{ kN} \]

Moment, due to \( H_{d1} \)
\[ M_{d1} = 112.5 \times 2.5 \]
\[ M_{d1} = 282 \text{ kNm} \]

Commentary: The moment introduces an eccentric loading of the footing. For calculation of allowable bearing capacity a revised footing width \( (B') \) is derived based on the centre of action of the applied loads. The engineer may choose any suitable method to determine \( B' \). The method chosen for this example is: Meyerhof G.G (1953). The bearing capacity of foundations under eccentric and inclined loads. 3rd ICSMFE. Zurich. Vol.1p.440-445

Eccentricity
\[ e = \frac{281}{886} = 0.32 \]

Effective width
\[ B_1' = B - 2e = 2.2 - (2 \times 0.32) \]
\[ B_1' = 1.56 \text{ m} \]

Effective area of footing:
\[ A_1' = 1.56 \times 2.2 = 3.43 \text{ m}^2 \]

Shape factor
\[ s_{c1} = 1 + 0.2(B'/L') \]
Design calculations for foundations and retaining structures

Project Number
ODPM EC7 8200

Example 4.2

Made by/date
SF 8/2004

Checked/date
PDS 9/2004

\[ s_{c1} = 1 + 0.2 \left( \frac{1.56}{2.2} \right) \]
\[ s_{c1} = 1.14 \]  

\( i_{c1} = \frac{1}{2} \left( 1 + \sqrt{1 - \frac{112.5}{3.43 \times 60}} \right) \)

\[ i_{c1} = 0.84 \]  

**Inclination factor**

**Design bearing resistance**

Using the analytical method as with example 1

\[ R/A = (\pi + Z) c_u \cdot s_c \cdot i_c + q \]  

Note: Except for \( A \), \( i_c \), and \( s_{c1} \) above the values used are as given in Example 4.1 for Combination 1.

\[ R_{d1} = 3.43 \left[ (\pi + 2) \times 60 \times 1.14 \times 0.84 + 30 \right] \]
\[ R_{d1} = 1116 \text{ kN} \]  

**Check**

\[ V_{d1} \leq R_{d1} \]
From Eqn (2) and (10)
\[ 886 < 1116 \]

For Combination 1,
2.2m × 2.2m footing has sufficient bearing resistance

**Combination 2**  \( A_2 \) “+” \( M_2 \) “+” \( R_1 \)

**Applied loads (Action \( A_2 \))**

**Vertical Design Load**

\[ V_{d2} = \gamma_G \times V_k \]
\[ V_{d2} = 1.0 \times 656 \]
\[ V_{d2} = 656\text{kN} \]  

**Horizontal Design Load**

\[ V_{h2} = \gamma_G \times V_k \]
H_{d2} = 1.3 \times 75 = 98 \text{kN} \quad \text{(12)}

Moment, due to H_{d2}
M_{d2} = 98 \times 2.5 = 245 \text{kN} \quad \text{(13)}

Eccentricity
e = M/V = 245/656 = 0.37 \quad \text{(14)}

Effective width
B_{12} = B - 2e = 2.2 - (2 \times 0.37) = 1.5 \text{m} \quad \text{(15)}

Effective area of footing:
A_{12} = 1.5 \times 2.2 = 3.3 \text{ m}^2 \quad \text{(16)}

Inclination factor
i_{c2} = \frac{1}{2} \left( 1 + \sqrt{\frac{1 - \frac{98}{3.3 \times 43}}{}} \right) \quad \text{(17)}

Shape factor
S_{c2} = 1 + 0.2(1.5/2.2) = 1.14 \quad \text{(18)}

Design bearing resistance
R_{d2} = 3.3 \times [(5.14 \times 43 \times 1.14 \times 0.78) + 30] = 748 \text{ kN} \quad \text{(19)}

Check
V_{d2} \leq R_{d2}
From Eqn (11) and (19)
656 < 748

For both combination 1 and combination 2, 2.2 \times 2.2 footing has sufficient bearing resistance

Comment:
Again, Comb. 2 is critical since R_{d2} / V_{d2} (1.14) is less than R_{d1} / V_{d1} (1.23)
Sliding Resistance

Require

\[ H_d \leq R_d \]  
Eqn 6.2

Design Resistance, based on factored ground properties

\[ R_d = A_c \cdot c_{ud} \]  
Eqn 6.4

Note: As the analysis is based on factored ground properties, Combination 2 values are used.

Area of base under compressive load

\[ A_c = A_j = 3.3 \text{ m}^2 \]

Design Shear Resistance

from Example 4.1, Eqn. (9)

\[ c_{ud} = c_{d2} = 43 \text{ kPa} \]

Sliding Resistance

\[ R_d = 3.3 \times 43 = 142 \text{ kN} \]

Check:

\[ H_d < R_d \]

From Eqn (12)

Applied Horizontal Load \( H_{d2} = 98 \text{ kN} \)

\[ 98 \text{ kN} < 142 \text{ kN} \]

Sliding is OK

Use a 2.2m × 2.2m pad footing

Note: A settlement check is required (see Example 4.1).
Chapter 4. Piles

4.4 General

Section 7 of BSEN 1997-1 is one of the most comprehensive. Most of it is devoted to vertical piles subjected to vertical loads\(^1\).

The words *ground resistance*\(^2\) are used throughout Section 7. In the context of piles, they mean ‘bearing capacity’ (whether in compression or in tension).

The Code leans heavily towards pile design based on load testing\(^3\), with the tacit assumption that the piles are installed in accordance with the corresponding BSEN ‘execution’ standards (see Section 6).

In a clear departure from common practice, the Code introduces the use of ‘correlation factors’, \(\xi\), for deriving the characteristic bearing capacity of piles from static pile load tests or from profiles\(^4\) of ground strength test results – see Box 4.2.

**BOX 4.2 – Correlation factors**

| Correlation factors have been introduced from some European practice in which piles are designed using the results of load test or several profiles of, for example, CPT tests. Their function is to make allowance for the quantity of data known about test pile performance and the ground, and for the variability of the data. Thus, the more data are available the lower the correlation factor value, leading to a smaller pile, while the more variable the data the more pessimistic is the value; there is a clear design benefit from carrying out more testing. Some background information on the values for the \(\xi\) factors provided in Annex A of BSEN 1997-1 can be found in Bauduin (2001). |

**Limit states.**

A list is provided of the most common limit states to be considered for pile foundations. The first seven of these are the usual ULS failure modes, either geotechnical or structural; the list does not imply that calculations or other explicit checks should be made for all of them.

---

\(^1\) The Section applies to all piles, regardless of the installation method (driving, jacking, screwing or boring with or without grouting) or their expected behaviour (by end-bearing or by friction), which is catered for by different partial factor values.

\(^2\) The word ‘ground’ is rightly used to indicate that the failure examined is failure in the ground. The word ‘resistance’ emphasises that the aim is to determine the maximum reaction from the ground (maximum shaft friction and, for piles in compression, any base resistance).

\(^3\) The Eurocode seeks to encourage greater use of pile (and ground) testing as a means to reduce the conservatism felt to exist in much design that is based on ground property values from ground investigations and pile installation procedures that may lack the requisite extent and quality.

\(^4\) In the context of BSEN 1997-1, a ‘profile’ may be thought of as a vertical sequence of information, from ground surface to required maximum depth, on such items as undrained shear strength or blow count in a Standard Penetration Test.
Actions.
In addition to the actions already mentioned in Section 2 of this Guide, BSEN 1997-1 identifies the specific actions on piles that result from ground movements¹.

Design methods and design considerations
BSEN 1997-1 requires that piles are designed using one of the following (the italics are the Authors¹):

- the results of relevant pile load tests;
- calculations that are based on valid load tests²;
- the results of dynamic tests (again based on valid load tests);
- observations of the performance of comparable piles.

Given the emphasis on pile load testing, BSEN 1997-1 has quite a bit to say about both static and dynamic pile load tests.

One of the ULS design requirements is that failure does not occur in the supported superstructure from undue displacement of the foundation. In these circumstances, and as it is not common practice to calculate pile displacements, the Code requires that a cautious range of settlements be adopted³.

Compressive or tensile resistance failure is defined as the state in which the pile foundation displaces significantly downwards or upwards with negligible increase or decrease of resistance. During load tests of piles in compression it is often difficult to reach this state, or to deduce it from a plot of load versus settlement. In common with much current practice, BSEN 1997-1 therefore

---

¹ Actions arise from:
- vertical ground settlements causing downdrag (or negative shaft friction);
- upward ground displacement causing heave forces on the pile;
- horizontal ground displacement transversely loading the pile.

Vertical settlements causing downdrag: Adopting the maximum (long-term) downdrag load for the design can lead to very conservative or even unrealistic pile size (see clause 7.3.2.2), in particular when the settlements of the ground are small and/or the compressible layer is very thick. To avoid this, a careful soil-pile interaction analysis may be carried out, for which a geotechnical specialist should be consulted.

Heave: For heave, BSEN 1997 - Part 1 requires the upward movement to be treated as an action in an interaction analysis (see clause 7.3.2.3(1)P); again, a geotechnical specialist should be consulted.

Transverse loading: Many situations are encountered, such as in piled bridge abutments, where ground movements subject piles to transverse loading. The relevant design situations are listed in the Code (see clauses 7.3.2.4(2) and 7.3.2.4(3)) which recommends soil-pile interaction analyses, such as using beams on linear or non-linear supports, using the horizontal modulus of sub-grade reaction or p-y curves.

² Such a calculation would use the well-known ‘α method’ in which the shaft resistance of a pile in a clay soil is a proportion (α) of the undrained shear stress $c_u$. Values of α range between 0.3 and 0.9 (depending on the type of clay and the manner in which $c_u$ has been determined) and are based on the back-analysis of pile tests in comparable situations.

³ It should be noted that no further indication is given in Section 7 of the Eurocode on how to check displacements corresponding to ULS in the supported structure. The guidance given for spread foundations in Section 4.3.2 in this Guide are also relevant for pile settlements.
defines failure as a pile head settlement of 10% of the effective diameter

Design of piles exhibiting compressive ground resistance
All ultimate limit states for an axially-loaded, vertical pile or group of vertical piles are avoided if the following basic inequality is satisfied:

\[ F_{c,d} \leq R_{c,d} \] (7.1)

where

- \( F_{c,d} \) is the design value of the axial compressive load;
- \( R_{c,d} \) is the design value of the ultimate compressive ground resistance.

\( R_{c,d} \) can be determined from:
- static pile load tests;
- ground test results;
- dynamic pile load tests.

As this is intended to be a simple guide to BSEN 1997-1 and because, for the run-of-the-mill project using piles, comprehensive pile testing is rarely performed, the Guide focuses, in Section 4.4.2, on design using ground test rather than pile load test data. The Code requirements for design using pile load test data have been summarised in Appendix 9.6.

4.4.2 Calculating ultimate compressive resistance using ground parameters from tests
Any calculations that use ground test results to predict pile bearing capacity must have been ‘established from pile load tests and from comparable experience’.

Any uncertainty in the calculation method may be dealt with by introducing a model factor to ensure a sufficiently ‘safe’ result. Nothing more is said about this factor in BSEN 1997-1.

---

1 This adoption of a well-known definition of ‘failure’ is important, since the ground resistance calculation methods used are based on failure loads measured during static load tests, and these loads can vary significantly depending on the chosen failure criterion.

2 It should be remembered that much routine design, using the \( \alpha \)-method for example, is based on correlation with pile test results.
Two calculation methods are given:

- An 'alternative procedure', where the ground test results (e.g. shear strength, cone resistance, etc) for all relevant test locations are first combined, adding all other relevant information, to obtain the 'characteristic values of base resistance and shaft friction in the various strata'; this is a common method for routine design in the UK.

- A procedure in which bearing resistance are calculated, for each profile location, using the results from one or more profiles of ground test results.

Alternative procedure

The characteristic values $R_{b,k}$ and $R_{s,k}$ may be calculated directly from values of ground parameters that do not have appreciable variation across a site, or when a series of ground profiles is not available, thus:

\[
R_{b,k} = A_b \cdot q_{b,k}
\]

and

\[
R_{s,k} = \sum A_{s,i} \cdot q_{s,k,j}
\]

where $q_{b,k}$ and $q_{s,k,j}$ are characteristic values of unit base resistance and unit shaft friction obtained from the values of ground parameters in stratum $i$. This more traditional way of calculating the pile compressive resistance from limited ground test results then follows steps 3 & 4, shown below for use with profiles of ground tests.

In this alternative procedure, $\xi$ factors are not used explicitly to cater for ground variability. Consequently, the values of $q_{b,k}$ and $q_{s,k,j}$ should implicitly take account of:

- the variability of the ground,
- the volume of soil involved in the failure mechanism considered,
- the spatial variability of the pile resistance,
- the variability due to the effect of pile installation and
- the stiffness of the structure.

As the values of the partial factors $\gamma_b$, $\gamma_s$ and $\gamma_t$ in Table 4.1 were developed for application in BSEN 1997-1 with the $\xi$-factors, they are not in themselves sufficiently large to be used alone in the alternative procedure and an additional model factor is required. Example 4.3 adopts a value of 1.4 for this model factor but it remains to be seen what value will be required by the National Annex.

---

1 Any doubts about the quality of test data or standards of workmanship in the piling operations may be reflected in the choice of characteristic value.

2 The procedure is similar to the one using the results of static pile load tests, that is by using $\xi$ factors to account for variability between the results for the different profiles of ground test data.
Using profiles of ground tests.
The steps in the procedure are:

1. Calculate the compressive pile resistance, \( R_{c;cal} \), separately for each profile of ground tests from:

\[
R_{c;cal} = R_{b;cal} + R_{s;cal}
\]

where \( R_{b;cal} \) and \( R_{s;cal} \) are the calculated base and shaft resistance respectively. The result is the predicted resistance of a pile if it was at the location of the profile of ground tests.

2. Calculate the characteristic values from:

\[
R_{c;k} = (R_{b;k} + R_{s;k}) = \frac{R_{b;cal} + R_{s;cal}}{\xi} = \min\left\{\frac{(R_{c;cal})_{mean}}{\xi_3}, \frac{(R_{c;cal})_{min}}{\xi_4}\right\}
\]

where \( R_{b;cal} \) is the calculated compressive base resistance, \( R_{s;cal} \) is the calculated compressive shaft resistance and where \( \xi_3 \) and \( \xi_4 \) are correlation factors that depend on the number of profiles of ground tests, \( n \), and are applied respectively to
- the mean values \( (R_{c;cal})_{mean} = (R_{b;cal} + R_{s;cal})_{mean} = (R_{b;cal})_{mean} + (R_{s;cal})_{mean} \)
- and to the lowest values \( (R_{c;cal})_{min} = (R_{b;cal} + R_{s;cal})_{min} \).

Values for \( \xi_3 \) and \( \xi_4 \) are given in Table A.10 of Annex A of BSEN 1997-1 and are reproduced in Table 4.2 below for convenience.

<table>
<thead>
<tr>
<th>( \xi ) for ( n ) =</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>7</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \xi_3 ) (mean)</td>
<td>1.4</td>
<td>1.35</td>
<td>1.33</td>
<td>1.31</td>
<td>1.29</td>
<td>1.27</td>
<td>1.25</td>
</tr>
<tr>
<td>( \xi_4 ) (min.)</td>
<td>1.4</td>
<td>1.27</td>
<td>1.23</td>
<td>1.2</td>
<td>1.15</td>
<td>1.12</td>
<td>1.08</td>
</tr>
</tbody>
</table>

Table 4.2 - Values of correlation factors for ground test results
(n is number of profiles of tests)

As for the procedure using pile load test results (see Appendix 9.6), this process of calculating the characteristic value of the pile compressive resistance as the minimum of the two (differently factored) lowest and mean values is intended to provide a suitably conservative result that takes account of (a) the number of profiles of ground tests commissioned in the ground investigation and (b) the scatter of the results between profile locations.

\[\text{Note that, if } \gamma_T > 1.0, \text{ the design value of the action exceeds its characteristic value}
\]

\[\text{and, if } \gamma_M > 1.0, \text{ the design value of the material property will be less than the characteristic value.}\]
3. The design resistance is then obtained by applying the partial factor $\gamma_t$ to the total characteristic resistance $R_{c,k}$ or by applying the partial factors $\gamma_s$ and $\gamma_b$ respectively to the characteristic shaft frictional resistance and to the characteristic base resistance, thus:

$$R_{c:d} = \frac{R_{c,k}}{\gamma_t}$$

or

$$R_{c:d} = \frac{R_{b,k}}{\gamma_b} + \frac{R_{s,k}}{\gamma_s}$$

Tables A.6 to A.8 of Annex A in BSEN 1997-1 list the values for these factors; for convenience, they have been drawn together in Table 4.1.

4. Account may be taken of the ability of the structure connecting the piles to transfer loads from weaker piles to stronger piles. If the structure is able to do so, the values of $\xi_3$ and $\xi_4$ may be divided by 1.1 provided $\xi_3$ does not fall below 1.0.

BSEN 1997-1 also includes material for pile design that is beyond the scope of this simple Guide. Consequently, brief discussion on the following material has been relegated to the Appendices or Footnotes indicated:

- calculation of ultimate compressive resistance using the results of dynamic pile testing (Appendix 9.7);
- the design of piles in tension (Appendix 9.8);
- the design of transversely-loaded piles.

Using DA-1 for piles in compression
We have seen that, in general, DA-1 requires the two Combinations to be applied separately:

Combination 1 : $A_1 + M_1 + R_1$
Combination 2 : $A_2 + M_2 + R_1$

---

1 As with axially loaded piles, the following considerations apply:
- the ultimate capacities should be checked for the rotation or translation of short, stiff piles and, in the case of long, slender piles, for bending failure of the pile and yield of the soil near ground surface (see clauses 7.7.1.(2)P and 7.7.1(3));
- account for any group effects (see clause 7.7.1(4)P);
- determine the transverse pile resistance using static load test results (see clause 7.7.2) or using ground test results and pile strength parameters (see clause 7.7.3);
- check for failure of the pile structure itself;
- assess the transverse pile displacement.

Specific requirements concern the variability of the ground near the surface (see clause 7.7.2(3)P) and the pile head fixity with the structure (see clauses 7.7.2(4) and 7.7.3(4)P).

The use of beam theory with horizontal modulus of subgrade reaction is explicitly recognised in BSEN 1997-1 for analysing the behaviour of long, slender piles (see clause 7.7.3(3)).
noting that, since the geotechnical resistance is normally calculated using factored values of material properties (strength), a resistance factor (R1) is then not additionally applied.

However, the design of piles (and anchorages) differs from this since much of pile design has derived from the results of resistance measurements in load testing. For piles, Combination 2 therefore becomes:

\[ A2 + (M1 \text{ or } M2) + R4 \]

This means that, for piles in compression, \( M1 (\gamma_M = 1.0) \) has no significance and factors \( R4 (\gamma_R > 1.0) \) are applied to resistances calculated using characteristic values of ground properties or characteristic values of load test resistance measurements. In specific cases of piles subjected to unfavourable actions from the ground (e.g. downdrag or lateral earth pressure), factor values \( M2 (\gamma_M > 1.0) \) are applied to the ground properties used to calculate the design value of this unfavourable action, while \( R4 \) factor values are used to calculate the design value of the (favourable) resistance to downdrag and external loading.

4.4.3 Vertical displacements of pile foundations (serviceability of supported structure)

As for many other foundations, it is necessary to assess the settlements of piles in order to prevent either ultimate or serviceability limit states being exceeded in the supported structure\(^1\).

It is understood that most often this assessment will be only approximate because of all the uncertainties in predicting settlements\(^2\). Again, BSEN 1997-1 permits settlement calculations to be replaced by bearing capacity calculations with a high factor of safety so that ‘a sufficiently low fraction of the ground strength is mobilised’; however, advice on this high factor value is not given in the Code.

Examples 4.3 and 4.4 illustrate aspects of the design of the compressive resistance of piles using BSEN 1997-1.

---

\(^1\) The values of allowable movements in the structure should be selected during the design. Some indicative limiting values of structural deformation are given in Annex H of BSEN 1997-1. Further guidance may be found in Burland et al (1977).

\(^2\) Methods for calculating the displacement of pile foundations include the well known linear elastic approach of Poulos and Davis (1980), as well as finite element calculations and methods based on the mobilisation of shaft friction.
4.4.4 The structural design of piles
BSEN 1997-1 requires that piles are checked for structural failure\(^1\); this is done using the appropriate structural Eurocode, depending on the material of the pile\(^2\).

Slender piles passing through water or through a thick layer of soft soil must be checked for buckling failure. This check is not required if the undrained shear strength, \(c_u\), of the soil exceeds 10 kPa.

4.4.5 Aspects of the construction of piles
The way piles are installed not only influences their design\(^3\), but also how they subsequently behave in supporting the superstructure. Therefore, BSEN 1997-1 requires a detailed plan of the piling operations, with a record of pile monitoring. Full-scale testing, re-driving or the installation of new piles is required if doubt exists about a pile's quality.

Pile integrity may need to be checked and this may require a number of different tests.

4.5 Anchorages
BSEN 1997-1 contains a relatively short section\(^4\) dealing with the design of anchorages. As this is a rather specialised topic it is not discussed further in this Guide; the interested reader is referred to Frank et al (2004).

4.6 Retaining structures
4.6.1 Introduction
Section 9 of BSEN 1997-1 gives only the basic requirements for the design of retaining structures without describing or specifying particular calculation methods. This Section of the Guide describes some general principles and presents simple gravity wall design calculations\(^5\) for ULS limit states.

BSEN 1997-1 identifies the following three main types of retaining structure:

- Gravity walls\(^6\);
- Embedded walls\(^7\);

\(^1\) Combination 1 usually governs the pile structural design.
\(^2\) BSENs 1992, 1993 and 1995 apply, respectively, for reinforced concrete, steel and timber piles.
\(^3\) In Clause 7.9(4), BSEN 1997-1 repeats some of the pile construction provisions, covered in the relevant execution standards issued by CEN/TC 288, that influence design (see Section 7 of this Guide).
\(^4\) Section 8 is only 6 pages in length compared with the ~170 pages of BS 8081:1989. Bear in mind, though, that BSEN 1537:1999 (the 'execution' standard for ground anchors, ~55 pages) covers material in BS 8081).
\(^5\) The design of gravity walls in Section 9 of BSEN 1997-1 relies on Section 6 for the design of the footing against bearing failure.
\(^6\) In these walls the weight of the wall, sometimes including stabilising masses of ground (e.g. in stem walls), plays a significant role in supporting the retained material.
\(^7\) These walls are usually relatively thin and made of steel, concrete or, occasionally, timber. For stability they rely either solely on the resistance provided by the passive earth pressure in front of the wall (these are 'cantilever walls') or on a combination of passive resistance near their toe and the support of anchorages or struts (these are 'supported walls'). In contrast to gravity walls, the bending resistance of an embedded wall plays a significant role in supporting the retained material.
- Composite retaining structures\(^1\).

As the design of embedded walls and composite retaining structures is rather specialised it is not considered further in this Guide. Readers can find guidance in Frank et al (2004) and Simpson & Driscoll (1998).

4.6.2. Limit States

Ultimate Limit States

The Code requires all possible limit states to be listed with, as a minimum, the following explicitly considered (note that this list is deliberately restricted to gravity walls):

1. Loss of overall stability\(^2\);
2. Foundation failure\(^3\);
3. Failure by toppling\(^4\);
4. Failure of the structural element\(^5\).

In addition to these, uplift (the UPL limit state) and problems associated with water movement (the HYD limit state) must also be considered for retaining structures (usually of the embedded wall variety) supporting excavations below the water table. These more complex, specialist issues are not discussed in this Guide and interested readers are again referred to Frank et al (2004).

Serviceability Limit States

These include:

- Unacceptable movement of the retaining structure which may affect its appearance or functionality, or other structures or utilities influenced by these movements.
- Unacceptable change in the groundwater regime.

\(^1\) These include a combination of the two walls described above. A typical example is a cofferdam. They also include reinforced earth and nailed retaining structures.

\(^2\) Figure 9.1 in BSEN 1997-1 illustrates the modes of overall stability failure that should be considered; these are GEO ultimate limit states since, as illustrated, failure would occur on a surface through the ground and would be governed by the ground strength.

\(^3\) Figure 9.2 in BSEN 1997-1 illustrates the modes of foundation failure that should be considered. Again, these are GEO ultimate limit states since failure could occur as a bearing capacity and/or sliding failure of the wall footing, in which the ground strength would govern; the principles of Section 6 would apply.

\(^4\) This 'EQU' failure would entail a loss of equilibrium of the wall acting as a rigid structure founded on rock, such that the strengths of the ground and of the structure play no part in the equilibrium calculation.

\(^5\) This failure involves breaking of a structural element or connection between elements. Examples are illustrated in Figure 9.5 a. to f. in BSEN 1997-1.
4.6.3. Actions, geometrical data and wall friction

Actions
We have seen that a clear distinction needs to be made between those actions that are favourable (that is they act to stabilise the structure) and those that are unfavourable (that is they act to destabilise the structure).

In BS EN 1997-1 earth pressures acting on retaining walls are regarded as being both geotechnical actions and geotechnical resistances, with little if any guidance on when to use which of them; groundwater pressures acting on the wall are regarded as geotechnical actions\(^1\).

For relatively rigid retaining structures such as gravity walls, the distinction between favourable and unfavourable geotechnical actions (or resistances) is quite straightforward: earth pressures on the active side are unfavourable geotechnical actions, while those on the passive side are favourable actions or resistances\(^2\).

Geometrical data
For retaining structures, geometrical data include excavation and water levels. In most cases, small variations in them are taken to be included in calculations by the selection of their characteristic values and the partial factor values applied to them. However, because the designs of retaining structures can be very sensitive to ground and water levels\(^3\), BSEN 1997-1 has specific requirements for unforeseen overdig\(^4\) in front of the wall and for groundwater levels on both sides of it.

Earth pressures
Determining the appropriate earth pressures is a major aspect of the design of earth retaining structures, since earth pressures depend not only on the strength and weight of the retained ground but also on the amount of permitted movement and deformation of the retaining wall\(^5\). As the wall moves and/or deforms, the earth pressure on the passive side of the wall increases until a maximum value is reached, usually at failure; on the active side, the earth pressure decreases as the

---

\(^1\) The distinction between a geotechnical action and a geotechnical resistance may be important since unfavourable (active pressure) forces and favourable (passive pressure) forces attract different partial action factors in Combination 1 of Design Approach 1; treating a passive earth force as a geotechnical resistance means that it attracts a different value of (resistance) factor. Care is required when factoring water pressures on both the active and passive sides of an embedded retaining structure.

\(^2\) The situation for flexible embedded walls, however, may be far more complex as it is frequently not clear where the active and passive pressures occur. In these circumstances, a calculation that includes interaction between the ground and the structure is required. Discussion of this is outside the scope of this Guide and the reader is referred to Frank et al (2004) or Simpson & Driscoll (1998). Driscoll et al (2008?) have argued that passive earth force on an embedded wall should be regarded as a resistance, not a (favourable) action.

\(^3\) This is especially so for embedded walls in soils with high undrained shear strength or high angles of shearing resistance.

\(^4\) The ULS design requirements for unforeseen overdig in front of the wall apply for embedded cantilever and supported walls.

\(^5\) Annex C of BSEN 1997-1 gives guidance on the amount of wall movement required to mobilise the limiting active and passive earth pressures.
wall moves. The magnitudes of earth pressure mobilised in SLS and ULS designs will depend on the stiffness of the wall, the stiffness of any supporting elements (struts, anchorages, etc) and the stress-strain behaviour of the ground; they will generally be different in the ULS and SLS calculations.

BSEN 1997-1 is conservative in providing over-estimates of the earth pressure on the active sides of the wall and under-estimates on the passive sides of the wall.

**Determining the appropriate earth pressures for use in design.**
For ULS design and since large wall deformations or deflections occur as collapse approaches, limiting values of the active and passive earth pressures are estimated using standard equations for plastic behaviour\(^1\).

Where walls are restrained from moving sufficiently to mobilise limiting earth pressures, calculations should be performed using intermediate values of earth pressures\(^2\).

For SLS design, numerical procedures are usually adopted.

**4.6.4. Design and construction considerations**

**ULS design**
As before for DA-1, two separate calculations are, in principle, performed for all relevant design situations:

a) A Combination 2 calculation in which a limit equilibrium check determines the height of the wall and the width of its base foundation\(^3\). This calculation should examine the horizontal and vertical force equilibrium and the moment equilibrium of the wall under all envisaged actions; Fig. 4.3 illustrates the types of actions and resistances that could apply for a typical gravity wall.

---

\(^1\) *Annex C* of BSEN 1997-1 provides equations that may be applied in terms either of total or of effective stresses, as appropriate. The values of the limiting earth pressure coefficients for \(\phi\) ranging from 10° to 45° and for different wall-ground interface parameters and ground inclinations, are presented graphically in *Annex C*; the graphs are based on Caquot, Kérisel and Absi (1973). Account should be taken of the effect that any compaction and water-filled tension or shrinkage cracks can have in increasing pressures.

\(^2\) Different types of retaining wall are subject to different degrees of movement and hence different earth pressures.

\(^3\) In principle, both Combination 1 AND Combination 2 calculations should be performed to determine the critical dimensions, though experience will usually indicate that Combination 2 governs.
Figure 4.3 – Actions and resistances for a simple gravity retaining wall

$G_w$ is the weight of the wall;
$P_A$ is the active earth force, a destabilising geotechnical action, that includes the active, variable pressure from the surcharge, q, and the active, permanent earth pressure, $\sigma_\text{act}$;
$P_p$ is the passive earth force, a stabilising geotechnical resistance;
$R_H$ is the earth resistance to sliding, a stabilising geotechnical action;
$R_V$ is the bearing resistance of the ground, a stabilising geotechnical action.

b) A Combination 1\(^1\) calculation to determine, for the critical geometry, the shear forces and bending moments from which the structural sections are determined using the appropriate structural Eurocode, such as BSEN 1992 in the case of a reinforced concrete wall or BSEN 1993 in the case of a steel sheet pile wall.

ULS designs are carried out using the appropriate earth pressures (see Section 4.6.3) and by ensuring that the design values of the actions and their effects ($E_d$) do not exceed the corresponding design values of the resistances ($R_d$) in the familiar expression:

$$E_d \leq R_d$$

1. In Combination 2, design values of non-geotechnical actions (e.g. the self-weight of the wall) are calculated from the equation $F_d = \gamma_F F_k$, while design values of geotechnical actions and resistances (e.g. earth pressure) are

\(^1\) In principle, both Combination 1 AND Combination 2 should be checked for the greater forces and moments in the critical geometry, though Combination 1 will almost always govern.
calculated using factored values of ground parameters, i.e.
\[ F_d = \gamma_F F(\chi_m) \] In both cases, \( \gamma_F = \gamma_G = 1.0 \), except for unfavourable variable actions (\( \gamma_Q = 1.3 \)).

2. In Comb. 1, by applying appropriate partial factors to:
   - the characteristic values of the actions, i.e., \( F_d = \gamma_F F_k \), for non-geotechnical actions;
   - the values of the actions calculated using the characteristic values of the ground parameters (because \( \gamma_M = 1.0 \)), i.e., \( F_d = \gamma_F F(X_a) \) for geotechnical actions.

Alternatively, design values of actions may be calculated by applying appropriate partial factors to the calculated effects of the actions, i.e.

\[ E_d = \gamma_E E \{ F_{rep}; X_k; a_d \} \]

for example to the bending moments (M) in the wall, which are calculated using the characteristic values of actions and ground parameters (i.e. \( \gamma_F = \gamma_M = 1.0 \) but \( \gamma_E > 1.0 \)).

The design values of actions and of their effects are calculated as follows, using the values of partial factors reproduced from Annex A and shown below in Table 4.4. These partial factors are taken from the following tables in that Annex:

- **Table A.3**: Partial factors on actions or the effects of actions.
- **Table A.4**: Partial factors on ground parameters.
- **Table A.13**: Partial resistance factors for retaining structures\(^1\).

<table>
<thead>
<tr>
<th>Design Approach (Sets of partial factors)</th>
<th>Partial factors on actions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unfavourable permanent ( \gamma_Q )</td>
</tr>
<tr>
<td>DA-1, Combination 1 (A1 ++ M1 ++ R1)</td>
<td>1.35</td>
</tr>
<tr>
<td>DA-1, Combination 2 (A2 ++ M2 ++ R1)</td>
<td>1.0</td>
</tr>
</tbody>
</table>

\(^{(1)}\) for favourable permanent action : \( \gamma_Q = 1.0 \)
\(^{(2)}\) for favourable variable action : \( \gamma_Q = 0.0 \)

<table>
<thead>
<tr>
<th>Design Approach (sets of partial factors)</th>
<th>Ground parameters (( \gamma_M ))</th>
<th>Resistances</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \gamma_R ) ( \gamma_D ) ( \gamma_C ) ( \gamma_{CU} )</td>
<td>Passive ( \gamma_{R,\ast} )</td>
</tr>
<tr>
<td>DA-1, Combination 1 (A1 ++ M1 ++ R1)</td>
<td>1.0 1.0 1.0 1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>DA-1, Combination 2 (A2 ++ M2 ++ R1)</td>
<td>1.0 1.25 1.25 1.4</td>
<td>1.0</td>
</tr>
</tbody>
</table>

**Table 4.4** - Recommended values of the partial factors on actions and the effects of actions, in persistent and transient situations, and on ground parameters and resistances.

\(^1\) Not shown here is **Table A.12**: Partial resistance factors for pre-stressed anchorages that are very often used to tie back retaining structures.
As shown in Fig. 4.3, the bases of gravity walls are usually spread foundations, often subjected to highly eccentric loads. BSEN 1997-1 does not include the familiar ‘middle third rule’ to ensure rotational stability, but instead requires that the design is explicitly checked for bearing failure under eccentric loading. For walls founded below the water table, uplift water pressures on the foundation should be included in the design. 

**SLS design**

Tolerable wall movements are usually not large enough to generate limiting earth pressures, particularly when the retained ground is over-consolidated or heavily compacted and when the retaining structure and its foundation are fairly rigid (e.g. a gravity wall). Appropriate earth pressures are then usually larger than the limiting active earth pressures and lower than the limiting passive earth pressures.

BSEN 1997-1 recommends a cautious approach to the assessment of tolerable wall movements and any effects of these movements on supported structures and services; ‘comparable experience’ is required.

Displacement calculations should be considered when walls retain more than 6m of cohesive soil of low plasticity or 3m of high plasticity soils, or when the wall is supported by soft clay within its height or beneath its base.

In SLS calculations\(^1\) the values of earth pressures should normally be obtained using design values of all ground parameters equal to their characteristic values (i.e. with \(\gamma_M = 1.0\)).

**Examples of gravity retaining wall design**

Example 4.5 illustrates the use of BSEN 1997-1 for the ULS design of a gravity wall where water pressures do not affect the design. Example 4.6 examines the problem with water pressures added and adopting a partial factor treatment of them. Example 4.7 addresses the same problem but uses the safety margin approach to deal with uncertainty in the water pressures.

---

\(^1\) Wall deflections and ground displacements can be calculated using numerical analyses (e.g. finite element models) of the ground-structure system including the complete wall construction with any support system. Alternatively, the wall can be modelled as a beam supported on elasto-plastic springs; this method does not predict supported ground movements.

When performing detailed deflection calculations, characteristic earth resistance and ground parameter values should be selected as cautious estimates of the values governing the SLS in question. Clause 4.2(8) of BSEN 1990 states that characteristic values of structural stiffness parameters are equal to their mean values while BSEN 1997-1 requires that the characteristic value of the ground stiffness is selected as a **cautious estimate of the mean value**.
Purpose of calculation: To determine the length of pile required.

**Design Approach 1. (Axially loaded piles)**

**Combination 1: A1 “+” M1 “+” R1**

Design Action (Load) (A1)

Partial Factor, $\gamma_G = 1.35$

$F_{c;d1} = 1500 \times 1.35$

$F_{o;d1} = 2025kN$

Note: For transparency in the calculation any difference in the weight of the pile and the displaced overburden load is not included.
Basic Pile Resistance Factors

Pile Base: \( 9 \, c_u \)

Pile Shaft : \( \alpha \, c_u \)
For this case, adopt \( \alpha = 0.5 \)

Pile Shaft : \( 0.5 \, c_u \)

Material Factors (M1)

\[ \gamma_{cu} = 1.0 \]

So, \( c_u = c_{u,k} \)

Note: No modification to adopted soil parameters is required for the design of axially loaded piles.

Design Resistance (R1)

Base Resistance: \( R_b = 9 \cdot c_u \cdot A_b \) \hspace{1cm} (2)

Shaft resistance: \( R_s = 0.5 \cdot c_u \cdot A_s \) \hspace{1cm} (3)

Compressive Resistance, \( R_{c;d} = R_{b;d} + R_{s;d} \) \hspace{1cm} 7.6.2.3(3)

Partial factors for Cfa piles

\[ \gamma_b = 1.1 \]

\[ \gamma_s = 1.0 \]

Note: When deriving characteristics values for pile design from ground parameters partial factors have to be corrected by a Model Factor. 7.6.2.3 (8)

Presumed Model Factor = 1.4

Partial factors for pile resistance for CFA piles:

\[ \gamma_{b;d} = 1.1 \times 1.4 = 1.54 \] \hspace{1cm} (5)

\[ \gamma_{s;d} = 1.0 \times 1.4 = 1.4 \] \hspace{1cm} (6)

Option (1)

Base Resistance: \( R_{b;d} = (9/1.54) \cdot c_u \cdot A_b = 5.85 \cdot c_u \cdot A_b \)
Shaft Resistance: \[ R_{s,d} = (0.5/1.4) \times C_u \times A_s = 0.36 \times C_u \times A_s \]  

Note: \( C_u \) is the average undrained shear strength of the clay along the pile shaft.

Compressive Resistance, \( R_{c,d} = R_{b,d} + R_{e,d} \)  

**Determination of length of pile to carry prescribed load [using Eqn (9)]**

Try 19m long pile

\[
R_{c,d1} = 5.84 \times (60 + 19 \times 8) \times \frac{\pi \times (0.6)^2}{4} + 0.36 \times 60 \times (60 + 19 \times 8) \times (19 \times \pi \times 0.6) \times 2
\]

\[= 350 \text{ kN} + 1753 \text{ kN} \]

\[ R_{c,d1} = 2153 \text{ kN} \]

From Eqn (1) \( F_{c,d1} = 2025 \text{ kN} \)

\[ R_{c,d1} > F_{c,d1} \]

**Conclusion:**

A pile 19m long, 600mm diameter can carry the load of 1500 kN under Combination 1.
Combination 2: A2 “+” (M1 or M2) “+” R4

Design Action (A2)

Partial Factor, $\gamma_Q = 1.0$

$$F_{c.d2} = 1500 \times 1.0$$

$$F_{c.d2} = 1500$$

Material Factors (M1) for Combination 1 and Combination 2 are the same i.e. M1, so $c_u$ is unchanged.

Design Resistance (R4)

Partial factor for Cfa piles

$\gamma_b = 1.45$

$\gamma_s = 1.3$

Presumed Model Factor = 1.4

Partial factors for pile resistance

$$\gamma_{b.d} = 1.45 \times 1.4 = 2.03$$

$$\gamma_{s.d} = 1.3 \times 1.4 = 1.82$$

Try 19m long pile

$$R_{b.d2} = (9/2.03) c_u A_b = 4.43 c_u A_b$$

$$= 4.43 \times (60 + 19 \times 8) \times \pi \times (0.6)^2 / 4$$

$$= 266 \text{kN}$$

(13)

$$R_{s.d2} = (0.5/1.82) c_u A_s = 0.27 c_u A_s$$

$$= 0.27 \times 60 + (60 + 19 \times 8) \times 19 \times \pi \times 0.6 / 2$$

$$= 1315 \text{kN}$$

(14)

$$R_{c.d2} = 266 + 1315$$
From Eqn (10) \( F_{c,dz} = 1500 \text{kN} \)

\[ R_{c,dz} > F_{c,dz} \]

**Conclusions:**

1. A pile 19 m long, 600mm diameter can carry the load of 1500 kN under Combination 1 and 2.

2. Combination 2 is marginally critical as \( (R_{c,d2}/F_{c,d2}) [1.05] < (R_{c,d1}/F_{c,d1}) [1.06] \)

**NOTES**

1. A pile settlement analysis is required to finally prove the pile dimensions

2. Traditional pile analysis has been based broadly on varying the overall factor of safety dependent on testing proposed.

   Eg

   \( \text{FofS} = 3 \) – No pile tests

   \( \text{FofS} = 2.5 \) – Test 1% of working piles

   \( \text{FofS} = 2 \) – Undertake a preliminary pile test

   Using this approach it has been expected that settlement criteria would be met, without the need for explicit analyses.

3. The equivalent factors of safety for the above analyses are:

   **Combination 1**
   \[ \gamma_0 \times \text{Model Factor} \times \gamma_t \]
   \[ = 1.35 \times 1.4 \times 1.1 \]

   \( \text{FofS} = 2.08 \)
Combination 2
\[ \gamma_G \times \text{Model Factor} \times \gamma_t \]

\[ = 1.0 \times 1.4 \times 1.4 \]

\[ \text{FoS} = 1.96 \]

The implication is that by the introduction of EC7 increased on site pile testing appears necessary to adequately verify pile design using ground parameters.
Note: This example demonstrates how a pile design should be approached using EC7. As for example 4.3, the calculation to determine the length of pile required is shown below. As before, Section 7 of the Code should be consulted when undertaking a complete design to determine which other limit states should be considered.

**Purpose of calculation:** To determine length of pile required.

**Design Approach 1. (Axially loaded piles)**

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

**Design Action (Load) (A1)**

Partial Factor, \( \gamma_a \) = 1.35

\[
F_{c,d1} = \frac{2000}{1.35} = 2700\, kN
\]

Note: For transparency in the calculation any difference in the weight of the pile and the displaced overburden load is not included.
Basic Pile Resistance Factors

Material Factors (M1)

All partial factors = 1.0

Note: No modification to adopted soil parameters is required for the design of axially loaded piles.

Resistance (R1)

Base resistance formula:

\[ N_q \cdot \sigma_v' \cdot A_b \]

\( N_q \) – bearing capacity factor
\( \sigma_v' \) – vertical effective stress at the pile toe
\( A_b \) – area of the base of pile

Bearing capacity factor:

\[ \phi' = 35 \]

\[ N_q = 55 \ (D/B = 20) \]

Area of base:

\[ A_b = \pi \times (0.3)^2 = 0.28 \text{m}^2 \]

Shaft resistance formula:

\[ K_s \cdot \bar{\sigma}_v' \cdot \tan \delta \cdot A_s \]

\( K_s \) – lateral load factor
\( \bar{\sigma}_v' \) – average effective stress on the pile shaft
\( \tan \delta \) - mobilised friction at the pile-soil interface
\( A_s \) – area of the pile shaft

\[ K_s = 1 \ (\text{Driven pile, large displacement}) \]

\[ \tan \delta = \tan (0.8\phi') \ (\text{Precast concrete}) \]

\[ \tan \delta = 0.53 \]
Design Resistance (R1)

Partial factors for driven piles in compression, $\gamma_b$, $\gamma_s$ & $\gamma_t = 1.0$

Model Factor = 1.4

Base resistance:

$$R_{bd1} = \frac{(55 \times \sigma_v' b \times 0.28)}{(\gamma_b \times 1.4)}$$

$$R_{bd1} = 11 \sigma_v' b$$  \hspace{1cm} (4)

Shaft resistance:

$$R_{sd1} = \frac{(1 \times \sigma_v' s \times 0.53 \times (0.6 \times \pi \times L))}{(\gamma_s \times 1.4)}$$

$$R_{sd1} = 0.71 \sigma_v' s L$$  \hspace{1cm} (5)

Compressive Resistance, $R_{cd} = R_{bd} + R_{sd}$

Determination of length of pile to carry prescribed load

Try 15m long pile

$$R_{b1} = 11 \times (2 \times 20 + 13 \times 10) = 1870kN$$

$$R_{s1} = 0.71 \times (0 + 180) \times 15 = 958kN \div 2$$

$$R_{c1} = 1870 + 958 = 2828kN (> F_{cd1} (2700kN))$$

Conclusion: A pile 15m long, 600mm diameter can carry the load of 2000kN under Combination 1.
Combination 2: A2 “+” (M1 or M2) “+” R4

Design Action (A2)

Partial Factor, \( \gamma_G = 1.0 \)

\[
F_{c;d2} = 2000 \times 1.0
\]

\[
F_{c;d2} = 2000
\]

Material Factors (M1) for Combination 1 and Combination 2 are the same i.e. M1

Design Resistance (R4)

Model Factor = 1.4

As for Combination 1, \( \gamma_b \) & \( \gamma_s \) are equal (For R4, \( \gamma_b = \gamma_s = 1.3 \))

Therefore \( R_{c;d2} = R_{c;d1} / 1.3 \)

For 15m long pile

\[
R_{c;d2} = \frac{1870 + 958}{1.3} = 2175\text{kN} \ (> F_{c;d2} \ (2000\text{kN}))
\]

Conclusion:

1. A pile 15m long, 600m diameter can carry the load of 2000kN under Combination 1 and Combination 2.
2. Combination 1 is marginally critical as \( \frac{R_{c;d1}}{F_{c;d1}} \) [1.05] < \( \frac{R_{c;d2}}{F_{c;d2}} \) [1.09]
Diagram Notes: The wall section is split into 3 (A, B, C) for analysis. The point O is 1/3 along the base. Moments will be taken about this point to prove whether the resultant line of action of forces lies within the middle third i.e. no tension in the base.

This example demonstrates how a retaining wall design should be approached using EC7. Stability and sliding calculations are included. However for a complete design, it will be necessary to consider other limit states discussed in the code such as bearing resistance and wall movement (Section 9).

**Trial width, B = 2.5m**

Position of point 0 = 2.5/3 = 0.83 m from 'a'

**Characteristic Vertical Load (Weight of Wall)**

A. \[ W_t = 4 \times 0.5 \times 24 = 48 \text{kN} \]
   Point of Action from O = \( 2.5 - 0.5/2 - 0.83 = 1.42 \text{m} \)

B. \[ W_t = 0.5 \times 2 \times 24 = 24 \text{kN} \]
   PoA from O = \( 2/2 - 0.83 = 0.17 \text{m} \)

C. \[ W_t = [(3.5 \times 2)/2 \times 24] = 84 \text{kN} \]
   PoA from O = \( (2 \times 2/3) - 0.83 = 0.50 \text{m} \)

\[ V_k = 48 + 24 + 84 = 156 \text{kN} \] (1)

Note: Design Approach 1 is used, as for Example 4.1

**Clauses**

2.4.2(4) & 9.3.1

2.4.7.3.4.2
## Stability

Combination 1 A1 “+” M1 “+” R1

### Design Loads (A1)

#### Design Wall Load

The weight of the wall is a favourable action as increasing the weight of the wall increases its stability.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_g$</td>
<td>1.0</td>
</tr>
<tr>
<td>$V_{d1W}$</td>
<td>$\gamma_g \times V_k$</td>
</tr>
<tr>
<td>$V_{d1W}$</td>
<td>1.0 $\times$ 156</td>
</tr>
<tr>
<td>$V_{d1W}$</td>
<td>156kN</td>
</tr>
</tbody>
</table>

#### Design Surcharge Load

Surcharges to the rear of retaining structures act in both a favourable and unfavourable manner at the same time.

For lateral load calculations, surcharge is an unfavourable action as it increases the tendency of the wall to overturn.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_Q$</td>
<td>1.5</td>
</tr>
<tr>
<td>$V_{d1s}$</td>
<td>$\gamma_Q \times V_k$</td>
</tr>
<tr>
<td>$V_{d1s}$</td>
<td>1.5 $\times$ 10</td>
</tr>
<tr>
<td>$V_{d1s}$</td>
<td>15kPa</td>
</tr>
</tbody>
</table>

For wall shear calculations, surcharge increases the shear force at the rear of the wall and is therefore a favourable action.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_Q$</td>
<td>0</td>
</tr>
<tr>
<td>$V_{d1s}$</td>
<td>10 $\times$ 0 = 0kPa</td>
</tr>
</tbody>
</table>

### Design Soil Parameters (M1)

#### Weight density

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma$</td>
<td>1</td>
</tr>
<tr>
<td>Design $\gamma_d$</td>
<td>20kN/m³</td>
</tr>
</tbody>
</table>

#### Angle of shearing resistance

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_\phi$</td>
<td>1</td>
</tr>
<tr>
<td>Design $\tan\phi_d = (\tan32^\circ) / 1 = \tan32^\circ$</td>
<td></td>
</tr>
</tbody>
</table>
Soil/Wall interface, $\delta$

$\delta = k \phi'_{cVD}$ with $k \leq 2/3$ (precast concrete)

In this case $\phi'_{cVD} = \phi_k' \\
\delta = 2/3 \times \phi' = 2/3 \times 32^\circ = 21.3^\circ$

Coefficient of horizontal active earth pressure

$K_a = 0.27$

Active pressure on the back of the gravity wall (A1)

$\sigma_o(z) = K_a (\gamma z + q)$

Lateral Design Soil Pressures (A1)

$\sigma_{pd1}(z) = K_a (\gamma_d z \times gG + V_{d1s})$

A lateral earth load is an unfavourable action as it causes the wall to have a greater tendency to overturn.

$\gamma_G = 1.35$

$V_{d1s} = 15\text{kPa (Eqn 3)}$

$\sigma_{pd1}(0) = 0.27 (20 \times 0 \times 1.35 + 15) = 4.1\text{kPa}$

$\sigma_{pd1}(4) = 0.27 (20 \times 4 \times 1.35 + 15) = 33.2\text{kPa}$

Lateral Earth Load (A1)

$H_{d1s} = (4.1 \times 4) + ((33.2 - 4.1) \times 4/2) \\
= 16.4 + 58.2$

$H_{d1s} = 74.6\text{kN/m of wall}$

Design Shear Force (A1)

Downward shear force on the back of the wall due to friction at the wall-soil interface.

$\tau_o(z) = K_a \tan \delta (\gamma z)$

Design Shear Load Due to Surcharge (A1)

From Eqn (4)

$V_{d1s,\tau} = 0$

Design Soil Shear Force

$\tau_{de}(z) = K_a \tan \delta (\gamma_d z \times \gamma_G)$
The soil mass retained behind the wall acts in a favourable manner as it causes a stabilising shear force. (Previously, the soil acted in an unfavourable manner when the lateral load was calculated)

\[
\gamma_G = 1.0 \\
\tau_{dl,e}(0) = 0.27 \cdot \tan 21.3^\circ \times (20 \times 0 \times 1.0) = 0 \\
\tau_{dl,e}(4) = 0.27 \cdot \tan 21.3^\circ \times (20 \times 4 \times 1.0) = 8.4\text{kN/mrun}
\]

Design Soil Shear Load (A1)

\[
V_{d1,e} = 8.4 \times 4/2 \\
V_{d1,e} = 16.8\text{kN/mrun}
\]

Passive Pressures

The depth of excavation to allow for in front of the wall (\(\Delta a\)) should strictly be limited to 10% of 3.5m i.e. 350mm (clause 9.3.2.2(2)). However in this example, the maximum excavation of 500mm is taken. (The effect of this is marginal). Therefore there are no passive pressures.

Design Criteria: Unreinforced Wall

Note – The wall is unreinforced, therefore the line of action of the resultant force must fall within the middle 1/3 of the base. The consequence of possible overturning is precluded by adopting this principle within the design.

Taking moments about 0:

\[
\Sigma M \text{ about } O = \text{[Wall]} + \text{[Wall/soil shear]} + \text{[Lateral earth pressure]} \\
= [(48 \times 1.42) + (24 \times 0.17) + (84 \times 0.5)] \\
+ [(16.8 \times (2.5 - 0.83)] \\
+ \left[-(16.4 \times 2) - (58.2 \times 4/3)\right] \\
= [114.2] + [28.1] + [-110.4] \\
= +32\text{kNm}
\]

Conclusion:

The line of action falls within the middle 1/3
Wall of base width 2.5m is acceptable for Combination 1
Combination 2 $A_2$ “+” (M1 or M2) “+” R1

Design Loads (Action $A_2$)

Wall:
\[ \gamma_G = 1.0 \]
\[ V_{d2W} = V_{d1W} = 156 \text{kN}, \text{(Eqn 2)} \] \hspace{1cm} (10)

Surcharge:
Lateral load calculations
\[ \gamma_Q = 1.3 \]
\[ V_{d2s} = 10 \times 1.3 = 13 \text{kPa} \] \hspace{1cm} \text{(11)}

Shear calculations
\[ \gamma_Q = 0 \]
\[ V_{d2s,\tau} = 10 \times 0 = 0 \text{kPa} \] \hspace{1cm} \text{(12)}

Design Soil Parameters (M2)

Weight density
\[ \gamma_v = 1 \]
- Design $\gamma_d = 20 \text{kN/m}^3$

Angle of shearing resistance
\[ \gamma_\phi = 1.25 \]
- Design $\tan \phi'_{d2} = (\tan 32^o) / 1.25 = 0.5$
\[ \phi'_{d2} = \tan^{-1}(0.5) = 26.6^o \] \hspace{1cm} \text{(13)}

Soil/Wall interface, $\delta$
- Take $\phi'_{\delta,v/d} = \phi'$
\[ \delta_d = 2/3 \times \phi' = 2/3 \times 26.6^o = 17.7^o \]

Coefficient of horizontal active earth pressure
\[ K_a = 0.33 \]

Active pressure on the back of the gravity wall (A2)

Lateral Design Pressures (A2)
\[ \sigma_{p,ax}(z) = K_a (\gamma_d \times z \times \gamma_G + V_{d2s}) \]
\[ \gamma_G = 1.0 \]

\[ \sigma_{p,ax}(0) = 0.33 (20 \times 0 \times 1.0 + 13) = 4.3 \text{kPa} \]
\[ \sigma_{p,ax}(4) = 0.33 (20 \times 4 \times 1.0 + 13) = 30.7 \text{kPa} \]
### Lateral Earth Load (A2)

\[
H_{d2s} = (4.3 \times 4) + (30.7 - 4.3) \times 4/2 \\
= 17.2 + 52.8 \\
H_{d2s} = 70 \text{kN/mrun}
\]

### Shear force (A2)

**Design Shear Force (A2)**

**Design shear load due to surcharge (A2)**

\[
V_{d2s} = 0 \quad (\text{Eqn 12})
\]

**Design soil shear pressures**

\[
\tau_{d2e}(z) = K_a \cdot \tan \delta_2 (\gamma_d \times z \times \gamma_0) \\
\gamma_0 = 1.0 \\
\tau_{d2e}(0) = 0 \\
\tau_{d2e}(4) = 0.33 \times \tan 17.7^\circ (20 \times 4 \times 1.0) \\
= 8.4 \text{kN/m}^2/\text{mrun}
\]

**Design Soil Shear Load (A2)**

\[
V_{d2e} = 8.4 \times 4/2 \\
V_{d2e} = 16.8 \text{kN/mrun}
\]

### Check Line of action of resultant force

**Take moments about 0**

\[
\Sigma M \text{ about } O = [\text{Wall}] + [\text{Wall/soil shear}] + [\text{Lateral earth pressure}] \\
= [(48 \times 1.42) + (24 \times 0.17) + (84 \times 0.5)] \\
+ [(16.8 \times (2.5-0.83)] \\
+ [- (17.2 \times 2) - (52.8 \times 4/3)] \\
= [114.2] + [28.1] + [-104.8] \\
= +37.5 \text{kNm}
\]

### Conclusion:

The line of action falls within the middle 1/3
Wall of base width 2.5m is acceptable for Combination 1 and 2

---

**Annex C**

Table A3 (Permanent Favourable) (14)

**Table A3**

(Permanen Unfavourable) (14)
Compare Combination 1 and 2

Stabilising Forces

A1. Wall Load = A2. Wall Load
Eqn 2 (156kN) = Eqn 10 (156kN)

Eqn 8 (16.8kN) = Eqn 15 (16.8kN)

Destabilising Forces

A1. Lateral Earth Load > A2. Lateral Earth Load
Eqn 7 (74.6kN) > Eqn 14 (70kN)

Conclusions:

1) Combination 1 governs overturning
2) Wall base of 2.5m is acceptable

Sliding

Require: \( H_d \leq R_d \)

Design Resistance to Sliding by Factoring Ground Properties

Combination 2 – Factored Ground Properties

Lateral Load, \( H_{d2} = H_{d2s} = 70kN/mrun \) (Eqn 14)

Design Resistance to Sliding

\[
R_{d2} = V_{d2} \tan \delta_{d2}
\]

\[
V_{d2} = [\text{Wall Load}] + [\text{Wall Shear}] = 156 + 16.8
\]

\[
V_{d2} = 172.8kN/mrun \quad \text{(17)}
\]

Basal angle of shearing resistance:

\[
\tan \phi_{d2} = 0.5 \quad \text{(Eqn13)}
\]

\[
\tan \delta_{dB2} = 0.5
\]

\[
R_{d2} = 172.8 \times 0.5
\]

\[
R_{d2} = 86.4kN/m \text{ run} \quad \text{(18)}
\]
Check: \( H_d \leq R_d \)

70kN (Eqn 14) \( < \) 86.4kN (Eqn 18)

Gravity wall of 2.5m width is acceptable to resist sliding and overturning and no tension is developed in its base.

**Conclusion**

(1) Wall Base of 2.5m is acceptable  
(2) Combination 1 governs overturning  
(3) Combination 2 governs sliding.
Retaining Walls

Gravity Wall with Water Pressures

[Clause 2.4.6.1(8) Partial Factor Approach]

Diagram Notes: See Example 4.5

This example demonstrates how a retaining wall design with water pressures should be approached using EC7. As for Example 4.5, only stability and sliding limit states are considered below. Section 9 of the code should be consulted when undertaking a complete design to determine which other limit states should be considered.

NOTE: Clause 9.6 (3) requires, for fine grained soils, that a reliable drainage system is installed behind the wall. Also consider clauses 2.4.6.1 (6) to (11).

1st trial, B = 2.7m

Position of point 0 = 2.7/3 = 0.9 m from ‘a’

Characteristic Vertical Load (Weight of Wall)

A. \[ Wt = 4 \times 0.5 \times 24 = 48 \text{kN} \]
### Design calculations for foundations and retaining structures

#### Example 4.6

<table>
<thead>
<tr>
<th>Project Number</th>
<th>ODPM EC7 8200</th>
</tr>
</thead>
<tbody>
<tr>
<td>Made by/date</td>
<td>SF 8/2004</td>
</tr>
<tr>
<td>Checked/date</td>
<td>PDS 9/2004</td>
</tr>
</tbody>
</table>

**Point of Action from O = 2.7 - 0.5/2 – 0.9 = 1.55 m**

**B.** \( W_t = 0.5 \times 2.2 \times 24 = 26.4\) kN  
PoA from O = 2.2 /2 – 0.9 = 0.2 m

**C.** \( W_t = [(2.2 \times 3.5) \times 24]/2 = 92.4\) kN  
PoA from O = (2.2 \_ 2/3) – 0.9= 0.57m

**Characteristic Vertical Load**

\[
V_k = 48 + 26.4 + 92.4 = 166.8\text{kN}
\]  

**Characteristic Base Water Pressure (Uplift Pressure)**

\[
U_{b,k} = (9.81 \times 2) \times 2.7/2 = 26.5 \text{ kN}
\]

PoA from O = (2.7 \times 2/3) – 0.9 = 0.9m

### Stability

#### Combination 1 A1 “+” M1 “+” R1

**Design Loads (A1)**

**Design Effective Wall Load**

**NOTE.** To prevent unrealistic situations arising, where different partial factors are applied to water pressures and the weight of the wall, a factor is applied to the net (effective) load.

\[
\gamma_G = 1.0
\]

\[
V_{d1w} = \gamma_G \times (V_k - U_{b,k})
\]

\[
V_{d1w} = 1.0 \times (166.8 - 26.5)
\]

\[
V_{d1w} = 140.3\text{kN}
\]

**Design Surcharge Load**

Lateral load calculations: \( V_{d1s} = 15\text{kPa} \) (Example 2b p2)

Shear calculations: \( V_{d1s,c} = 15 \times 0 = 0\text{kPa} \) (Example 2b p2)

**Design Soil Parameters (M1) (Example 4.5 p3)**

Design \( \tan \phi'_d = 32^\circ \)  
\( \gamma_{d1} = 20 \text{kN/m}^3 \)  
\( \delta_{d1} = 21.3^\circ \)
Ka1 = 0.27

Active pressure on the back of the gravity wall (A1)

\[ \sigma_a(z) = K_a (\gamma_z + q) \]

Lateral Design Soil Pressures (A1)

\[ \sigma_{p,d1}(z) = K_a (\gamma_z \times z \times \gamma_G + V_{d1s}) \]

\[ \gamma_G = 1.35 \]

\[ \sigma_{p,d1}'(0) = 0.27 (20 \times 0 \times 1.35 + 15) = 4.1 \text{kPa} \]

\[ \sigma_{p,d1}'(2) = 0.27 (20 \times 2 \times 1.35 + 15) = 18.6 \text{kPa} \]

\[ \sigma_{p,d1}'(4) = 0.27 ([20 \times 4 - 9.81 \times 2] \times 1.35 + 15) = 26.1 \text{kPa} \]

Lateral Earth Load (A1)

\[ H_{d1s} = (4.1 \times 2) + ((18.6 - 4.1) \times 2/2) + (18.6 \times 2) + ((26.1 - 18.6) \times 2/2) \]

\[ = 8.2 + 14.5 + 37.2 + 7.5 \]

\[ H_{d1s} = 67.4 \text{kN/m run of wall} \] (6)

Lateral Design Water Pressure (A1)

Note: Clause 2.4.2(4) includes groundwater pressures as actions

\[ U_w(z) = \gamma_G \times \gamma_W \times z \]

\[ \gamma_G = 1.35 \]

Lateral Water Load (A1)

\[ U_{d1} = 1.35 \times 9.8 \times 2 \times 2/2 \]

\[ U_{d1} = 26.5 \text{kN/m run of wall} \] (7)

Design Shear Force (A1)

\[ \tau_a(z) = K_a \tan \delta (\gamma z) \]
From Example 2b.p4, the contribution of surcharge to friction at the wall-soil interface is zero.

**Design Soil Shear Pressures**

\[
\tau_{dl,e}(z) = K_d \tan \delta (\gamma_d \times z \times \gamma_G)
\]

\[G = 1.0\]

\[\tau_{dl,e}(0) = 0.27 \tan 21.3 (20 \times 0 \times 1.0) = 0\]

\[\tau_{dl,e}(2) = 0.27 \tan 21.3 (20 \times 2 \times 1.0) = 4.2 \text{kN/m run}\]

\[\tau_{dl,e}(4) = 0.27 \tan 21.3 \left[ ((20 \times 4) - (9.81 \times 2)) \times 1.0 \right] = 6.4 \text{kN/m run}\]

**Design Soil Shear Load (A1)**

\[V_{d1,e} = 4.2 \times 2/2 + (4.2 + 6.4)/2 \times 2\]

\[V_{d1,e} = 14.8 \text{kN/m run}\]

**Note:**
As for Example 4.5, the depth of excavation allowed for in front of the wall is 500mm. Therefore there are no passive pressures to consider.

**Check**
Line of action of resultant force must fall within the middle 1/3 of the base

**Take moments about 0**

\[\Sigma M, \text{wall} = (48 \times 1.55) + (26.4 \times 0.2) + (92.4 \times 0.57) = 74.4 + 5.3 + 52.7 = 132.4\]

\[\Sigma M, \text{base water pressure} = -(26.5 \times 0.9) = -23.9\]

\[\Sigma M, \text{wall/soil shear} = 14.8 \times 1.8 = 26.6\]

\[\Sigma M, \text{lateral earth pressure} = -(8.2 \times 3) - (14.5 \times 2.67) - (37.2 \times 1) - (7.5 \times 0.67) = -105.5\]

\[\Sigma M, \text{lateral water pressure} = -(26.5 \times 0.67) = -17.8\]
\[ \Sigma M \text{ about } O = 132.4 - 23.9 + 26.6 - 105.5 - 17.8 = +11.8 \text{ kNm/m run} \]

Line of action falls within the middle 1/3
Wall of base width 2.7m is acceptable for Combination 1

**Combination 2** (A2 “+” M2 “+” R1)

**Design Loads (A2)**

**Design Effective Wall Load**

\[ \gamma_G = 1 \]
\[ V_{d_{2W}} = V_{d_{1W}} = 140.3\text{kN} \text{ from (Eqn 3)} \]

**Design Surcharge Load**

Lateral load calculations: \( V_{d_{2s}} = 13\text{kPa} \) (Example 4.5 p5)
Shear calculations: \( V_{d_{2s,c}} = 10 \times 0 = 0\text{kPa} \) (Example 4.5 p5)

**Design Soil Parameters (M2)** (Example 4.5 p5)

Weight density,
\[ \gamma = 1 \]
\[ \gamma_{d2} = 20\text{kN/m}^3 \]
Design tan \( \phi'_{d2} = 0.5 \)
\[ \gamma_{\alpha2} = 20 \text{kN/m}^3 \]
\[ \delta_{d2} = 17.7^\circ \]
\[ K_{a2} = 0.33 \]

Active pressure on the back of the gravity wall (A2)

**Lateral Design Pressures (A2)**

\[ \gamma_G = 1.0 \]
\[ \sigma_{p,d2}'(0) = 0.33 \times (20 \times 0 \times 1.0 + 13) = 4.3\text{kPa} \]
\[ \sigma_{p,d2}'(2) = 0.33 \times (20 \times 2 \times 1.0 + 13) = 17.5\text{Pa} \]
\[ \sigma_{p,d2}'(4) = 0.33 \times [(20 \times 4) - (9.81 \times 2)] \times 1.0 + 13] \]
\[ = 24.2\text{kPa} \]

**Lateral Earth Load (A2)**
H_{d2s} = (4.3 \times 2) + ((17.5 - 4.3) \times 2/2) + (17.5 \times 2) +
((24.2 - 17.5) \times 2/2)
= 8.6 + 13.2 + 35 + 6.7
H_{d2s} = 63.5 \text{ kN/m run of wall} \hspace{1cm} (14)

**Lateral Water Load (A2)**

\[ \gamma_G = 1.0 \]

\[ U_{d2s;d} = 1.0 \times 9.81 \times 2 \times 2/2 = 19.6 \text{ kN/m run of wall} \]

**Design Shear Force (A2)**

Shear due to surcharge, \( V_{d2s;e} = 0 \) (Eqn 6)

**Design Soil Shear Pressures**

\[ \tau_{d2,e}(z) = K_a \tan \delta \ (\gamma_d \times z \times \gamma_G) \]

\[ \gamma_G = 1.0 \]

\[ \tau_{d2,e}(0) = 0 \]

\[ \tau_{d2,e}(2) = 0.33 \times \tan 17.7^\circ \ (20 \times 2 \times 1) = 4.2 \text{ kN/m run} \]

\[ \tau_{d2,e}(4) = 0.33 \times \tan 17.7^\circ \ \left[\left(20 \times 4\right) - (9.81 \times 2)\right] \times 1.0]) \]

\[ = 6.4 \text{ kN/m run} \]

**Design Soil Shear Load (A2)**

\[ V_{d2,e} = 4.2 \ - \ 2/2 + (4.2 + 6.4)/2 \times 2 \]

\[ V_{d2,e} = 14.8 \text{ kN/m run} \hspace{1cm} (16) \]

**Check Line of action of resultant force**

Take moments about 0

\[ \Sigma M, \text{ wall} = 132.4 \text{ (Eqn 9)} \]

\[ \Sigma M, \text{ base water pressure} = -23.9 \text{ (Eqn 10)} \]
\[ \Sigma M, \text{wall/soil shear} = 14.8 \times 1.8 = 26.6 \]

\[ \Sigma M, \text{lateral earth pressure} = - (8.6 \times 3) - (13.2 \times 2.67) - (35 \times 1) - (6.7 \times 0.67) = -100.5 \]

\[ \Sigma M, \text{lateral water pressure} = - (19.6 \times 0.67) = -13.13 \]

\[ \Sigma M \text{ about } O = +11.8 \]

Line of action falls within the middle 1/3

**Compare Combination 1 and 2**

**Stabilising Forces**

<table>
<thead>
<tr>
<th>A1. Effective Wall Load Moment</th>
<th>=</th>
<th>A2. Effective Wall Load Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>(132.4)</td>
<td></td>
<td>(132.4)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>(26.6)</td>
<td></td>
<td>(26.6)</td>
</tr>
</tbody>
</table>

**Destabilising Forces**

<table>
<thead>
<tr>
<th>A1. Lateral Earth Load Moment</th>
<th>&gt;</th>
<th>A2. Lateral Earth Load Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>(105.5)</td>
<td></td>
<td>(100.5)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>A1. Lateral Water Load Moment</th>
<th>&gt;</th>
<th>A2. Lateral Water Load Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>(17.8)</td>
<td></td>
<td>(13.13)</td>
</tr>
</tbody>
</table>

**Conclusion**

1. Wall Base of 2.7m is acceptable
2. Combination 1 governs overturning

**Sliding**

Require: \( H_d \leq R_d \)

Establish design resistance to sliding by factoring Ground Properties

**Combination 2 (Factored Ground Properties)**
Lateral Load: \( H_d = H_{d2} + U_{d2} = 63.5 + 19.6 = 83.1 \text{kN/m run} \) \hspace{1cm} (18)

Design Resistance to Sliding by Factoring Ground Properties

\[ R_d = V_d \tan \delta \]

\[ V_d = \text{[Effective Wall Load]} + \text{[Wall Shear]} = [140.3] + [14.8] \]

\[ V_{di} = 155.1 \text{kN/m run} \]

Basal angle of shearing resistance:

\[ \tan \delta_{B2} = \tan \phi'_d \]

\[ \tan \phi'_d = 0.5 \text{ (Eq 2b p5)} \]

\[ R_d = 155.1 \times 0.5 = 77.6 \text{ kN/m run} \] \hspace{1cm} (20)

Check if \( H_d < R_d \)

From Eqn (19) and (20) \( 83.1 > 77.6 \)

So, Wall Base of 2.7m is not acceptable for sliding

Try gravity wall with 3.2m wide base

\[ V_d = 48 + (2.7 \times 0.5 \times 24) + (3.5 \times 2.7/2 \times 24) - (19.6 \times 3.2)/2 + 14.8 \]

\[ V_d = 177.2 \text{kN} \]

\[ R_d = 177.2 \times 0.5 = 88.6 \text{kN} \] \hspace{1cm} (21)

Check: \( H_{d2s} < R_d \)

From Eqn (19) and (21) \( 83.1 < 88.6 \)

Gravity wall 3.2m wide is acceptable to resist sliding and overturning, with no tension developing in its base.

Conclusion

(1) Wall Base of 3.2m is acceptable
(2) Combination 1 governs overturning
(3) Combination 2 governs sliding.
Retaining Walls

Gravity Wall with Water Pressures

[Clause 2.4.6.1(8) Safety Margin Approach]

Diagram Notes: See Example 4.5

This example demonstrates how a retaining wall design with water pressures (safety margin approach) should be approached using EC7. As for examples 4.5 and 4.5, only stability and sliding limit states are considered below. Section 9 of the code should be consulted when undertaking a complete design to determine which other limit states should be considered.

NOTE: Clause 9.6 (3) requires, for fine grained soils, that a reliable drainage system is installed behind the wall. Also consider clauses 2.4.6.1 (6) to (11).

1st trial, \( B = 2.7\,\text{m} \)

Position of point 0 = \( 2.7/3 = 0.9 \,\text{m} \) from ‘a’
Characteristics of Vertical Load (Weight of Wall)

\[
\begin{align*}
A. & \quad W_t = 4 \times 0.5 \times 24 = 48 \text{kN} \\
& \quad \text{Point of Action from O} = 2.7 - 0.5/2 - 0.9 = 1.55 \text{ m} \\
B. & \quad W_t = 0.5 \times 2.2 \times 24 = 26.4 \text{kN} \\
& \quad \text{P.O.A from O} = 2.2/2 - 0.9 = 0.2 \text{ m} \\
C. & \quad W_t = [(2.2 \times 3.5) \times 24]/2 = 92.4 \text{kN} \\
& \quad \text{P.O.A from O} = (2.2 \times 2/3) - 0.9 = 0.57 \text{ m}
\end{align*}
\]

Characteristic Vertical Load:
\[V_k = 48 + 26.4 + 92.4 = 166.8 \text{kN}\] (1)

Characteristic Base Water Pressure (Uplift Pressure):
\[U_{b;k} = (9.81 \times 2.5) \times 2.7/2 = 33.1 \text{ kN}\] (2)

Stability

Combination 1 A1 “+” M1 “+” R1

Design Loads (A1)

Design Wall Load
\[
\begin{align*}
\gamma_G &= 1.0 \\
V_{d1} &= \gamma_G \times V_k \\
V_{d1} &= 1.0 \times 166.8 \\
V_{d1} &= 166.8 \text{kN}
\end{align*}
\] (3)

Design Uplift Pressure

Note: The characteristic pressure by safety margin approach is unfactored
\[U_{b;d} = 33.1 \text{kN}\] (4)

Design Surcharge Load

Lateral load calculations: \[V_{d1s} = 15 \text{kPa}\] (Example 4.5 p2) (5)

Shear calculations: \[V_{d1s,c} = 15 \times 0 = 0 \text{kPa}\] (Example 4.5 p2)
Design Soil Parameters (M1) (Example 4.5 p3)

- Design tan $\phi'_d = 32^\circ$
- $\gamma_{d1} = 20 \text{kN/m}^3$
- $\delta_{d1} = 21.3^\circ$
- $K_{a1} = 0.27$

Active pressure on the back of the gravity wall (A1)

$$\sigma_a(z) = K_a (\gamma z + q)$$

Lateral Design Soil Pressures (A1)

$$\sigma_{a,d1}(z) = K_a (\gamma_d \times z \times \gamma_G + V_{d1s})$$

- $\gamma_0 = 1.35$
- $\sigma_{a,d1}(0) = 0.27 (20 \times 0 \times 1.35 + 15) = 4.1\text{kPa}$
- $\sigma_{a,d1}(1.5) = 0.27 (20 \times 1.5 \times 1.35 + 15) = 15.0\text{kPa}$
- $\sigma_{a,d1}(4) = 0.27 ([20 \times 4 - 9.81 \times 2.5] \times 1.35 + 15)$
  $$= 24.3\text{kPa}$$

Lateral Earth Load (A1)

$$H_{d1s} = (4.1 \times 1.5) + ((15.0 - 4.1) \times 1.5/2) + (15 \times 2.5)$$

$$+ ((24.3 - 15) \times 2.5)$$

$$= 6.2 + 8.2 + 37.5 + 11.6$$

$$H_{d1s} = 63.5\text{kN/m run of wall} \quad (6)$$

Lateral Design Water Pressure (A1)

Note: Clause 2.4.6.1(8) safety margin approach uses unfactored water pressures

$$U_w(z) = \gamma_w \times z$$
Lateral Water Load (A1)

\[ U_{d1} = 9.8 \times 2.5 \times 2.5/2 \]
\[ U_{d1} = 30.7\text{kN/m run of wall} \]  

Design Shear Force (A1)

\[ \tau a(z) = K_a \tan(\delta(z)) \]

From Example 4.5 p4, the contribution of surcharge to friction at the wall-soil interface is zero.

Design Soil Shear Pressures

\[ \tau_{dl,e}(z) = K_a \tan(gd z \times gG) \]
\[ g_G = 1.0 \]
\[ \tau_{dl,e}(0) = 0.27 \tan 21.3 (20 \times 0 \times 1.0) = 0 \]
\[ \tau_{dl,e}(1.5) = 0.27 \tan 21.3 (20 \times 1.5 \times 1.0) = 3.2\text{kN/m run} \]
\[ \tau_{dl,e}(4) = 0.27 \tan 21.3 [((20 \times 4) - (9.81 \times 2.5)) \times 1.0)] \]
\[ = 5.8\text{kN/m run} \]

Design Soil Shear Load (A1)

\[ V_{d1,e} = 4.2 \times 1.5/2 + (3.2 + 5.8)/2 \times 2.5 \]
\[ V_{d1,e} = 14.5\text{kN/m run} \]

Note:
As for Example 4.5, the depth of excavation allowed for in front of the wall is 500mm. Therefore there are no passive pressures to consider.

Check
Line of action of resultant force must fall within the middle 1/3 of the base

Take moments about 0

\[ \Sigma M, \text{wall} = (48 \times 1.55) + (26.4 \times 0.2) + (92.4 \times 0.57) \]
\[ = 74.4 + 5.3 + 52.7 = 132.4 \]
\[ \sum M, \text{ base water pressure} = -(33.1 \times 0.9) = -29.8 \]
\[ \sum M, \text{ wall/soil shear} = 14.5 \times 1.8 = 26.1 \]
\[ \sum M, \text{ lateral earth pressure} = -(6.2 \times 3.25) - (8.2 \times 3) - (37.5 \times 1.25) - (11.6 \times 0.83) = -101.2 \]
\[ \sum M, \text{ lateral water pressure} = -(30.7 \times 2.5/3) = -25.6 \]
\[ \sum M \text{ about } O = 132.4 - 29.8 + 26.1 - 101.2 - 25.6 = +1.9 \text{ kNm/m run} \]

Line of action falls within the middle 1/3
Wall of base width 2.7m is acceptable for Combination 1

**Combination 2** (A2 “+” M2 “+” R1)

**Design Loads (A2)**

**Design Wall Load**

\[ \gamma_g = 1 \]
\[ V_{d2w} = V_{d1w} = 166.8 \text{kN} \text{ from (Eqn 3)} \]

**Design Uplift Pressure**

From (4) \( U_{ud2} = 33.1 \)

**Design Surcharge Load**

Lateral load calculations: \( V_{d2s} = 13 \text{kPa} \) (Example 4.5 p5)
Shear calculations: \( V_{d2s,t} = 10 \times 0 = 0 \text{kPa} \) (Example 4.5 p5)

**Design Soil Parameters (M2)** (Example 4.5 p5)

Weight density,
\[ \gamma_i = 1 \]
\[ \gamma_d = 20 \text{kN/m}^3 \]
\[ \text{Design } \tan \phi_d = 0.5 \]
\[ \gamma_d = 20 \text{ kN/m}^3 \]
\[ \delta_{d2} = 17.7^\circ \]
\[ K_{s2} = 0.33 \]
Active pressure on the back of the gravity wall (A2)

Lateral Design Pressures (A2)

\[ \gamma_G = 1.0 \]

\[ \sigma_{a,d_2} = Ka (\gamma_d \times 2 \times \gamma_G + V_{d_2s}) \]

\[ \sigma_{a,d_2}(0) = 0.33 (20 \times 0 \times 1.0 + 13) = 4.3 \text{kPa} \]

\[ \sigma_{a,d_2}(1.52) = 0.33 (20 \times 1.5 \times 1.0 + 13) = 14.2 \text{Pa} \]

\[ \sigma_{a,d_2}(4) = 0.33 \left[ \left(20 \times 4 \right) \times \left(9.81 \times 2.5\right) \right] \times 1.0 + 13 \] = 22.5 kPa

Lateral Earth Load (A2)

\[ H_{d_2s} = (4.3 \times 1.5) + ((14.2 - 4.3) \times 1.5/2) + (14.2 \times 2.5) \]
\[ + ((22.5 - 14.2) \times 2.5/2) \]
\[ = 6.5 + 7.4 + 35.5 + 10.4 \]

\[ H_{d_2s} = 59.8 \text{ kN/m run of wall} \]

Lateral Water Load (A2)

\[ U_{d_2s;d} = 1.0 \times 9.81 \times 2.5 \times 2.5/2 = 30.7 \text{ kN/m run of wall} \]

Design Shear Force (A2)

Shear due to surcharge, \( V_{d_2s,t} = 0 \) (Eqn 6)

Design Soil Shear Pressures

\[ \tau_{a,d_2,e}(z) = K_a \tan \delta \left( \gamma_d \times z \times \gamma_G \right) \]

\[ \gamma_G = 1.0 \]

\[ \tau_{a,d_2,e}(0) = 0 \]

\[ \tau_{a,d_2,e}(1.5) = 0.33 \times \tan 17.7\degree \times (20 \times 1.5 \times 1) = 3.2 \text{kN/m run} \]
\[ \tau_{d2,e}(4) = 0.33 \tan 17.7^\circ \times \left( (20 \times 4) - (9.81 \times 2.5) \right) \times 1.0 \] = 5.8 kN/m run

Design Soil Shear Load (A2)

\[ V_{d2,e} = 3.2 \times 1.5/2 + (3.2 + 5.8)/2 \times 2.5 \]
\[ V_{d2,e} = 16.1 \text{ kN/m run} \]

Check Line of action of resultant force

Take moments about 0

\[ \sum M, \text{wall} = 132.4 \text{ (Eqn 9)} \]
\[ \sum M, \text{base water pressure} = -29.8 \text{ (Eqn 10)} \]
\[ \sum M, \text{wall/soil shear} = 16.1 \times 1.8 = 29.0 \]
\[ \sum M, \text{lateral earth pressure} = 6.5 \times 3.25 - 7.4 \times 3 - 35.5 \times 1.25 - 10.4 \times 0.83 = -96.3 \]
\[ \sum M, \text{lateral water pressure} = -30.7 \times 0.83 = -25.5 \]
\[ \sum M \text{ about O} = +9.8 \]

Line of action falls within the middle 1/3

**Conclusion**

1. Wall Base of 2.7m is acceptable
2. Combination 1 governs overturning
Design calculations for foundations and retaining structures

---

**Sliding**

**Require**: $H_d \leq R_d$

Establish design resistance to sliding by factoring Ground Properties

**Combination 2** (Factored Ground Properties)

Lateral Load, $H_{d2} = H_{d2s} + U_{d2s} = 59.8 + 30.7 = 90.5\text{kN/m run}$ \hfill (18)

**Design Resistance to Sliding by Factoring Ground Properties**

$$R_{d2} = V_{d2} \tan \delta_{d2}$$

$$V_{d2} = \text{[Wall Load]} + \text{[Wall Shear]} - \text{[Uplift Pressure]}$$

$$= [166.8] + [16.1] - [33.1]$$

$$V_{d1} = 149.8\text{kN/m run}$$

Basal angle of shearing resistance:

$$\tan \delta_{d2} = \tan \delta_{d2} \\text{tan} \delta_{d2} = 0.5 \text{ (Eg 2b p5)}$$

$$R_{d2} = 149.8 \times 0.5 = 74.9 \text{kN/m run} \hfill (19)$$

**Check** if $H_{d2} < R_{d2}$

From Eqn (19) and (20) \hfill (90.5 > 74.9)

So, Wall Base of 2.7m is not acceptable for sliding

**Try gravity wall with 3.5m wide base**

$$V_{d2} = 48 + (3 \times 0.5 \times 24) + (3.5 \times 3)/2 \times 24) + 16.1 - (9.8 \times 2.5 \times 3.5)/2$$

$$V_{d2} = 48 + 36 + 126 + 16.1 - 42.9 = 183.2$$

$$R_{d2} = 183.2 \times 0.5 = 91.6 \text{kN} \hfill (20)$$

**Check**: $H_{d2s} < R_{d2}$

From Eqn (19) and (21) \hfill (90.5 < 91.6)

Gravity wall 3.5m wide is acceptable to resist sliding and overturning, with no tension developing in its base.

**Conclusion**

1. Wall Base of 3.5m is acceptable
2. Combination 1 governs overturning
3. Combination 2 governs sliding.
5. OTHER GEOTECHNICAL DESIGN AND CONSTRUCTION MATTERS

5.1 General

BSEN 1997-1 has the attraction of bringing together in one document a set of design principles for all basic geotechnical problems. In contrast, the current BS geotechnical Codes are largely 'problem-related'\(^1\).

Section 4 dealt with the design of specific elements such as foundations and retaining walls. This Section brings together some general design issues, including those design decisions that bear upon construction activities, that are presented in separate Sections of BSEN 1997-1.

Section 5 deals firstly with the most fundamental requirement that the site itself must be stable. It then briefly discusses aspects of the design of embankments.

One of the important elements of geotechnical design is that concerning groundwater, both static and moving; the subject is not addressed separately and explicitly in most current BS Codes. BSEN 1997-1 introduces two limit states (UPL and HYD) that largely concern water, both static and flowing; as these matters are beyond the scope of a ‘simple’ Guide, they are not discussed further.

Section 5 then explains how BSEN 1997-1 addresses the issues of supervision, monitoring and maintenance before concluding with a short sub-section on fill, dewatering, and ground improvement and reinforcement.

5.2 Overall Stability

Section 11 of BSEN 1997-1 deals with designing and checking the overall stability of natural slopes, embankments, excavations and retained ground, and with ground movement around foundations on sloping ground and near excavations or coasts.

5.2.1 Limit States

Overall stability ultimate limit states cover:

- GEO limit states, in which failure occurs only in the ground; examples are the failure of a natural slope, or of an embankment on soft clay,

- STR limit states, in which collapse or excessive movement in the ground combines with failure of a structural element such as a sheet-pile wall, or an anchorage, supporting an excavation.

Serviceability limit states cover excessive movements\(^1\) in the ground due to, for example:

\(^1\) For example, BS8004 deals with foundations while BS8002 deals separately with retaining walls, many of which have spread foundations to be designed using BS8004; similarly, BS6031 deals separately with the design and construction of earthworks.

\(^1\) The term ‘excessive movements’ includes movements causing loss of serviceability or damage in associated structures, roads or services.
shear deformations in natural and artificial slopes;
- settlements in retained ground beside excavations;
- vibrations generated by machinery used to densify the ground;
- the swelling of desiccated clay.

5.2.2. Actions and design situations

Lists of possible actions and situations are provided.

5.2.3. Design and construction considerations

BSEN 1997-1 states that overall stability should be checked by calculations, taking into account ‘comparable experience’. If checking indicates instability or unacceptable movements, the design should provide stabilising measures. In cases where overall stability is not certain, additional investigations, monitoring and analysis are recommended.

5.2.4. Ultimate limit state design

Natural slopes, including any existing or planned structures, must be checked to ensure that they are stable. The partial factor values for the actions, material properties and resistances used in a stability analysis are given in Tables A.3, A.4 and A.14 of Annex A of BSEN 1997-1. The same requirements apply for slopes and cuts in rock and for the slopes of excavations and man-made embankments.

Slope stability is usually checked by one of the following analyses:
- assume a failure surface in a numerical analysis that adopts, for example, the ‘method of slices’;
- closed-form, limit analysis solutions for simple geometrical problems such as vertical cuts or infinite slopes;
- advanced numerical methods such as involving finite elements.

Whatever analytical method is used, the Code recommends that overall stability checks include:

- establishing the problem geometry including ground profiles and hydrogeological conditions;
- making separate calculations using upper and lower characteristic values of ground weight density, since it is not possible to distinguish readily between favourable and unfavourable gravity loads when calculating the most adverse slip surface.

The partial factor values for geotechnical actions are obtained from Table A.3 of Annex A of BSEN 1997-1.
5.3 Design of embankments

Section 12 of BSEN 1997-1 is concerned with the design requirements for embankments for infrastructure, such as road and railway embankments, and for 'small' dams; these requirements compare closely with those for construction using fill (see Section 5.8). While Section 12 is, in common with the other Sections of the Code, very limited in terms of details of the design, it does lay down the guiding principles that any design must meet.

5.3.1. Limit States

These are given in a checklist and it is should be noted that three items concern water and its movement:

− internal erosion
− failure caused by surface erosion or scour,
− deformations caused by hydraulic actions.

5.3.2. Actions and Design Situations

The most unfavourable groundwater condition within the embankment and free water levels in front of it should be selected when assessing stability.

5.3.3. Design and Construction Considerations

In the extensive list of matters to be considered, we are reminded of the value of relevant experience and of the need to observe and evaluate local conditions.

5.3.4. Ultimate Limit State Design

Overall instability and hydraulic failure must be prevented (see Sections 5.2 and 5.4).

5.3.5. Serviceability Limit State Design

Any deformation of the embankment must not lead to a serviceability limit state in either the embankment or any adjacent structures, roads or services. The SLS design checks are performed using design values of the actions and material deformation (stiffness) parameters that are numerically equal to the characteristic values since, for SLS design, the partial factor values ($\gamma_M$) are normally 1.0.

5.3.6. Supervision and Monitoring

BSEN 1997-1 recommends recording the results of the monitoring of certain items of an embankment during construction and subsequent usage; these will vary according to the particular design circumstances.

In view of the many possible design and construction situations and possible failure modes of embankments, the Observational Method, with its reliance on supervision and monitoring in conjunction with design calculations, is recommended.

---

1 BSEN 1997-1 does not say what ‘small’ means. Since the Code is concerned primarily with the design of Geotechnical Category 2 structures, it is fair to assume that ‘small’ means ‘to a height of approximately 10m.

2 The list reflects concern for the possible adverse influence of the embankment on adjacent structures, roads and services by including limit states involving them. To this can be added the importance of water in overall instability, deformation and the effects of climate.
5.4 Supervision of construction, monitoring and maintenance

5.4.1 Introduction
In Section 4, BSEN 1997-1 requires that all geotechnical construction processes, including the workmanship applied, must be supervised, that the performance of the structure must be monitored, both during and after construction, and that the finished structure must be adequately maintained. The amount of supervision, monitoring and maintenance will depend on the nature of the project and the ground conditions.

These requirements are fulfilled by:
- Ensuring safety and quality through supervision, monitoring and maintenance. The nature and quality of the supervision and monitoring prescribed for a project must be commensurate with the degree of precision assumed in the design, and in the values of the engineering parameters chosen and the partial factors used in the calculations. If the reliability of the design calculations is in doubt, it may be necessary to prescribe an enhanced regime of construction supervision and monitoring.
- Communicating what must be done formally in contract documents and records such as the Geotechnical Design Report (GDR);
- Supervising, monitoring and maintaining in an orderly, planned manner, with the keeping of records.

It is the designer’s responsibility to specify the requirements. Clearly, it cannot be the designer’s responsibility to ensure that post-construction monitoring and maintenance are carried out on the structure; however, it is the designer’s responsibility to prepare and communicate specifications for any such monitoring and maintenance, so that assumptions made in the design can be confirmed during and after construction.

Section 4 contains no specific details about construction workmanship and the reader is advised to check in the appropriate BSEN ‘execution’ standard for special geotechnical works (see Section 6 of this Guide).

As in other parts of BSEN 1997-1, Section 4 does not specify precise contractual arrangements. It does, however, indicate what tasks must be undertaken and that an appropriate flow of information must be maintained. Section 4 is concerned principally with the actions of the designer and with information others should make available to him.

BSEN 1997-1 specifically requires that extracts from the GDR concerning any post-construction monitoring and maintenance must be provide to the owner/client.

A list of the more important items to be considered for supervision during construction is given in Annex J of BSEN 1997-1. The reference in Annex J to ‘…due consideration of geotechnical limit states’ particularly concerns matters such as the collapse of an excavation in which men are working. Item J.2.1(4) of this Annex refers to the ‘safety of workers’ which is not
specifically covered by the Code as it is a matter for National health and safety regulation. Reference should be made to the Construction (Design and Management) Regulations - the CDM Regulations (Health and Safety Commission, 1994).

5.4.2 Supervision

‘Supervision’ means checking both the design and the construction. A plan is required, to be included in the GDR, that will depend on the scale and complexity of the project, as may be represented by Geotechnical Categories (GCs). In the case of GC1, a visual inspection of the ground, superficial quality controls and a qualitative assessment of the performance of the structure during and immediately after construction, may be all that are required. For GC2, measurements of the properties of the ground or the movement of the structure may be necessary. Tests to check the movement under load and the quality of piles, or tests of the density of fill behind a retaining wall are examples of the monitoring that might be required for GC2 projects.

‘The construction shall be inspected on a continuous basis...’ ‘Continuous’ means that inspection should not be so infrequent that important features of the ground or of the installation of foundations, for example, may be missed.

That inspection records should systematically be kept and made available to the designer is an important requirement of BSEN 1997-1. The following features must be recorded, as appropriate:

- Significant ground and groundwater features;
- The sequence of works;
- The quality of materials;
- Deviation from design;
- As-built drawings;
- Results of measurements and their interpretation;
- Observations of the environmental conditions;
- Unforeseen events.

5.4.3 Checking ground conditions

BSEN 1997-1 describes the manner in which the checking of ground and ground water conditions is carried out by inspection, description and recording.

5.4.4 Checking construction

This must include checks that site operations comply with those specified in the GDR.

5.4.5 Monitoring

The purposes and objectives of monitoring are listed; again, compliance with any requirements stated in the GDR is emphasised.

5.5 Fill, Dewatering, Ground Improvement and Reinforcement

5.5.1 Introduction

This material has been gathered together in Section 5 of BSEN 1997-1
because the subject matter generally concerns improving the properties of the ground. The Section provides few specific requirements except for some aspects of the use of fill. The Section effectively comprises a checklist of items which must be considered in the design.

In BSEN 1997-1, fill placed during construction works is regarded as part of the ‘structure’. BS EN 1997-1 defines a ‘structure’ as ‘an organized combination of connected parts, including fill placed during execution of the construction works, designed to carry loads and provide adequate rigidity’.

5.5.2. Fundamental requirements

Filled, dewatered, improved or reinforced ground has to satisfy the same fundamental requirements as natural ground, namely that it must be capable of sustaining the actions arising from its function and from its environment.

5.5.3. Constructing with Fill

The Code covers the selection, placement, compaction and checking of fill in the following circumstances:

- beneath foundations and floor slabs,
- in excavations,
- behind retaining structures
- for general landfilling, including hydraulic fill,
- for landscape mounds
- for the placement of spoil heaps
- for embankments for small dams and transportation infrastructure.

It provides a checklist of items to be covered in specifications for earthworks.

Design calculations require the assessment of the characteristic values of the fill properties which are cautious estimates of the values affecting the occurrence of the limit state. It should be noted that, at the time the design is made, the fill to be used may not have been identified, though its properties will have been specified.
5.5.4. Dewatering

‘Dewatering’ means the abstraction of water to improve the behaviour of the ground and to facilitate construction in it. Groundwater ‘recharge’ may also be required, as part of a dewatering scheme, to prevent the lowering of the water table.

The importance of the control of groundwater is emphasised in several places in BSEN 1997-1 and the Code provides a checklist of conditions to be met when designing a dewatering scheme. It is particularly important to check that groundwater lowering has no unwanted consequences for structures in the vicinity. The design of dewatering schemes often utilises the Observational Method.

5.5.5. Ground improvement and reinforcement

BSEN 1997-1 discusses the design of ground improvement and ground reinforcement only in very general terms, with some aspects being listed. In many cases, the design of ground improvement and ground reinforcement would be classified as a GC3 problem and, as such, would be covered by the requirements of BSEN 1997-1 only in general terms; design would be carried out by a geotechnical specialist.

In order that the effectiveness of improvement may be checked, the Code requires that an initial geotechnical investigation be carried out to determine ground conditions before the improvement technique is applied.
6. EUROPEAN GEOTECHNICAL CONSTRUCTION STANDARDS

6.1 Introduction

Particularly for geotechnical engineering, design and construction are inextricably linked, in no small way because much of the knowledge of the ground cannot be acquired during the design phase and only comes to light during the construction process itself. However, BSEN 1997-1, being a ‘design’ code, says little about construction matters beyond indicating that:

- European ‘execution’ Standards either exist or will do so, and
- ‘Execution is covered to the extent that is necessary to comply with the assumptions of the design rules’. This simply means that appropriate construction processes and standards of workmanship are implicitly assumed in the design guidance given.

In a clear departure from current geotechnical BS Codes of practice, BSEN 1997-1 does not provide the detailed combination of ‘design’ and ‘construction’ advice with which we are familiar. Instead, we have to address a separate collection of ‘execution’ standards produced by CEN TC288 which were developed, by a separate group of European engineers, with the aim of meeting the design requirements of EN 1997-1; the list of published and anticipated documents is given in Table 6.1.

<table>
<thead>
<tr>
<th>TC 288 Execution Standards</th>
<th>Title</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>BSEN 1536:2000</td>
<td>Bored piles</td>
<td>Published by BSI (see dates in first column).</td>
</tr>
<tr>
<td>BSEN 1537:2000</td>
<td>Ground anchors</td>
<td></td>
</tr>
<tr>
<td>BSEN 1538:2000</td>
<td>Diaphragm walls</td>
<td></td>
</tr>
<tr>
<td>BSEN 12063:1999</td>
<td>Sheet pile walls</td>
<td></td>
</tr>
<tr>
<td>BSEN 12699:2001</td>
<td>Displacement piles</td>
<td></td>
</tr>
<tr>
<td>BSEN 12715:2000</td>
<td>Grouting</td>
<td></td>
</tr>
<tr>
<td>BSEN 12716:2001</td>
<td>Jet grouting</td>
<td></td>
</tr>
<tr>
<td>prEN 14199</td>
<td>Micropiles</td>
<td>prEN dated April 1998; conversion to EN in progress</td>
</tr>
<tr>
<td>prEN 14475</td>
<td>Reinforced fills</td>
<td>prEN dated March 2004; issued for formal CEN vote.</td>
</tr>
<tr>
<td>prEN 14490</td>
<td>Soil nailing</td>
<td></td>
</tr>
<tr>
<td>prEN 14679</td>
<td>Deep mixing</td>
<td>prEN dated March 2004; issued for formal CEN vote.</td>
</tr>
<tr>
<td>prEN 14731</td>
<td>Deep vibration</td>
<td>Editing</td>
</tr>
<tr>
<td>prEN xxxxx</td>
<td>Deep drainage</td>
<td>Awaiting decision to proceed to CEN Enquiry</td>
</tr>
</tbody>
</table>

Table 6.1 — European Geotechnical Execution Standards
6.2 Compatibility between BSEN 1997-1 and Execution Standards

It is evident from Table 6.1 and Section 2.1 of this Guide that many of the specific topics covered in the Execution Standards have no corresponding design guidance in BSEN 1997-1. Where this applies, for example to the design of reinforced fills, engineers may continue to use BS8006:1995 with BSEN 14475 (when it is published).

As has been said, our BS Codes contain both design guidance and quite comprehensive advice about how to construct geotechnical elements. A close examination of the differences between the new Execution Standards and construction aspects of the current BS Codes has, for those elements covered by both, revealed very little in the former that conflicts with the latter. Table 6.2 summarises the comparisons and their outcome and it can be seen that, apart from BS 8081, there are only a few, minor differences. As is discussed in Section 7, any conflicts between the new geotechnical BSEnS and BS Standards will be dealt with by amendments to the latter to make them comply with the Principles of the former.

It may be concluded that, for the geotechnical elements specifically covered by the new BSEN ‘execution’ standards, no significant changes to current practice are envisaged. For all other construction-related guidance, designers will be able to continue to have recourse to the advisory material contained in current BS Codes.
<table>
<thead>
<tr>
<th>BS Code</th>
<th>BS Code</th>
<th>BS Code</th>
<th>BS Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>BS 8004</td>
<td>BS 8004</td>
<td>BS 8004</td>
<td>BS 8004</td>
</tr>
<tr>
<td>Jet grouting</td>
<td>Grouting</td>
<td>Displacement piles</td>
<td>Shear pile walls</td>
</tr>
<tr>
<td>BS 8004</td>
<td>BS 8002</td>
<td>BS 8004</td>
<td>BS 8002</td>
</tr>
<tr>
<td>None</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>BS 8004</td>
<td>BS 8002</td>
<td>BS 8004</td>
<td>BS 8002</td>
</tr>
<tr>
<td>Ground anchors</td>
<td>Ground anchors</td>
<td>Ground anchors</td>
<td>Ground anchors</td>
</tr>
<tr>
<td>BS 8004</td>
<td>BS 8002</td>
<td>BS 8004</td>
<td>BS 8002</td>
</tr>
<tr>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
</tr>
<tr>
<td>BS 8004</td>
<td>BS 8002</td>
<td>BS 8004</td>
<td>BS 8002</td>
</tr>
<tr>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
</tr>
<tr>
<td>BS 8004</td>
<td>BS 8002</td>
<td>BS 8004</td>
<td>BS 8002</td>
</tr>
<tr>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
</tr>
<tr>
<td>BS 8004</td>
<td>BS 8002</td>
<td>BS 8004</td>
<td>BS 8002</td>
</tr>
<tr>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
</tr>
<tr>
<td>BS 8004</td>
<td>BS 8002</td>
<td>BS 8004</td>
<td>BS 8002</td>
</tr>
<tr>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
</tr>
<tr>
<td>BS 8004</td>
<td>BS 8002</td>
<td>BS 8004</td>
<td>BS 8002</td>
</tr>
<tr>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
</tr>
<tr>
<td>BS 8004</td>
<td>BS 8002</td>
<td>BS 8004</td>
<td>BS 8002</td>
</tr>
<tr>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
<td>Small difference in design and specification regarding safe procedures for pile operations</td>
</tr>
<tr>
<td>BS 8004</td>
<td>BS 8002</td>
<td>BS 8004</td>
<td>BS 8002</td>
</tr>
</tbody>
</table>
7. HOW THE NEW GEOTECHNICAL CODES AND STANDARDS WILL BE APPLIED AND MAY HAVE AN IMPACT ON UK PRACTICE

7.1 Introduction
The BS Codes and Standards have been developed and updated over many decades; indeed some of them are now showing their age. While not necessarily using them every day, many design and checking engineers will be familiar with their content. It will, therefore, be a significant task for engineers to replace the BS documents with the new BSENs and to become familiar with the new concepts contained in BSEN 1997-1.

The manner in which the change-over to the new BSENs will take place is not yet fully determined but is expected to be quite close to the procedures that are explained below.

7.2 How the BSENs will be implemented

7.2.1. The National Annexes
As has been said, the setting of levels of safety for buildings and civil engineering works, and parts thereof including aspects of durability and economy, are a matter for individual member states of the EU. Such national choice, as identified by a ‘Note’ in EN 1997, is made in the National Annex (NA). The NA for BSEN 1997-1 is expected to contain the following features:

1. The stipulation that only Design Approach 1 will be used for the GEO and STR ultimate limit state design calculations;
2. The values of the partial factors to use in all design calculations;
3. A statement that none of the informative Annexes in EN 1997-1 are to be withdrawn or replaced;
4. ‘Country specific data’; these include such things as the minimum foundation depth to avoid frost heave in susceptible ground conditions or subsidence in shrinkable clay with trees nearby.
5. With regard to prescriptive measures, reference to the Approved Documents of the Building Regulations for the minimum widths and depths of footings for low-rise buildings;
6. Reference to ‘non-contradictory, complementary information to assist the user in applying the Eurocode’.

In addition, the NA is likely to list the relatively few areas of conflict of principle between current BS codes and the BSENs, so that amendments to the former are clearly identified.

As some national choice is expected to be available in EN 1997-2, an NA for BSEN 1997-2 is likely to be produced.

The Building Regulations Division of ODPM has stipulated that draft NAs be issued for public comment before they are implemented. This consultation process cannot begin until BSEN 1997-1 and BSEN 1997-2 have been published by BSI.
7.2.2. **Withdrawal of BS codes and Standards.**
The intention is that, eventually, all BS geotechnical codes and standards are withdrawn. However, since much of the general, ‘best-practice’, detailed advice given in them is not found in BSEN documents and since this advice does not conflict with any of the principles laid down in the latter, it is anticipated that, once minor amendments have been made to remove the few conflicts mentioned in Table 6.2, the material in these BS codes will be referenced in the NA to ‘assist the user in applying the Eurocode’ (see 7.2.1 item 6 above); however, the status of the material will no longer be that of a Code or Standard. Since most of the familiar BS code material will still exist after the introduction of the BSENs there is scope for some confusion in peoples’ minds about which documents take precedence. A general rule could be that, where there is overlap, the BSEN governs and that the old BS material may be used only to supplement the design and construction activities. This proposal will be tested during the anticipated public consultation process.

7.3 **A timetable for change**
The likely sequence of events leading to full implementation of the suite of BSENs and any eventual withdrawal of (old) BS codes and standards is laid out in Table 7.1.

<table>
<thead>
<tr>
<th>Document</th>
<th>Event</th>
<th>Anticipated date</th>
</tr>
</thead>
<tbody>
<tr>
<td>BSEN 1997-1</td>
<td>− Publication by BSI</td>
<td>Was published December 2004</td>
</tr>
<tr>
<td>BSEN 1997-2</td>
<td>− Publication by BSI</td>
<td>End-2006</td>
</tr>
<tr>
<td>NA for BSEN 1997-1</td>
<td>− Issue for public comment</td>
<td>Mid-2006</td>
</tr>
<tr>
<td>NA for BSEN 1997-1</td>
<td>− Publication by BSI</td>
<td>End-2006</td>
</tr>
<tr>
<td>Remaining Execution Standards (see Table 6.1)</td>
<td>− Publication by BSI</td>
<td>End-2006</td>
</tr>
<tr>
<td>Ground Testing Standards and Technical Specifications (see Table A.4.2)</td>
<td>− Publication by BSI</td>
<td>End-2007</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Document</th>
<th>Event</th>
<th>Anticipated date</th>
</tr>
</thead>
<tbody>
<tr>
<td>All BS codes</td>
<td>− Withdrawal or amendment to remove completely any conflict; retention of BS documents where a BSEN equivalent does not exist</td>
<td>End-2009 or 2010</td>
</tr>
</tbody>
</table>

(Note: The precise timing of events is uncertain. Much depends on (a) the speed at which CEN issues finished documents to BSI and (b) how quickly BSI is able to add national Forewords and proceed to publication. It is understood that NAs will be issued separately from the BSENs or as integral parts.)

**Table 7.1 — Likely implementation timetable**
7.4 How the BSENs may be applied

The Eurocodes are particularly relevant for projects involving EU cooperation or competition, especially on publicly-funded work, where it is likely to become a legal requirement to accept designs which satisfy the Eurocodes. It is anticipated that the BSENs will be fully implemented by such UK public bodies as the Highways Agency and Network Rail. The Office of the Deputy Prime Minister will aim to have all references in the Building Regulations to BSs replaced by references to BSENs.
8. REFERENCES AND BIBLIOGRAPHY


179p.


Bibliography.


APPENDICES

Appendix 9.1 — Contrasting design philosophies

In ‘traditional’ design calculations such as those employed in BS 8004, satisfactory design was achieved by making sure that stresses in materials, e.g. in the soil beneath a foundation, were kept to ‘working’ levels; this was done by applying a ‘global safety factor’ in the design calculation. For example, in:

\[ E \leq \frac{R}{F} \] (A1)

where

- \( E \) is the disturbance or force acting;
- \( R \) is the resistance offered by the structure (e.g. the bearing pressure beneath the footing);
- \( F \) is a factor to make sure that \( E \) is ‘sufficiently’ less than \( R \).

The precise nature and magnitudes of the ingredients of \( E \) and \( R \) are not often clearly specified in current national geotechnical Codes.

Traditionally, \( F \) has been a fairly large number, for example 3 for a simple strip footing and 2 — 3 for a pile. These factor values were found, from experience by testing and from the back-analysis of observations, firstly to prevent ‘failure’ and secondly to ensure that settlements under working loads remained acceptably small. In the global safety factor method, checking that both ULS and SLS requirements were met was performed in the one calculation.

Historically, the ‘limit state’ method became popular at about the time that partial safety factors began to be adopted in structural engineering. The two are therefore often associated, 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 would not occur. The limit state method does not necessarily require calculations as the basis of design.

The Eurocode design philosophy, on the other hand, firstly clearly separates ULS from SLS and secondly deals rather more rigorously with the identification and treatment of the many uncertainties inherent in a design problem. It does this by:

- making a clear distinction between actions from the superstructure (as stipulated in BSEN 1991) and those from the ground;
- by separating the uncertainties in these actions, with the partial factors for structural actions coming from BSEN 1990 and for geotechnical actions from BSEN 1997-1.
- by separating the uncertainty in the reactions from the ground from that of the structural loads.

Since most of BSEN 1997-1 is concerned with checking the avoidance of a ULS, this rigour is applied to the uncertainties in the calculation of, for example, bearing capacity and ground strength rather than to settlement calculations for checking the avoidance of an SLS.
Appendix 9.2 - The three alternative ‘Design Approaches’

Design values of the effect of actions

In the general expression

\[ E_d = E \{ \gamma_F \times F_{\text{rep}} ; X_k/\gamma_M ; a_d \} \]  \hspace{1cm} (2.6a)

the effects of an action (such as the bending moments and shear forces in foundations) are functions of the action itself, of ground properties and of the geometry of the geotechnical structure. In (2.6a) it can be seen that the partial factors are applied to the actions (\( \gamma_F \times F_{\text{rep}} \)) and to the ground properties (\( X_k/\gamma_M \)) that lead to actions (such as earth pressure forces) before the effects (E) of these are determined.

Alternatively, \( E_d \) may be expressed as:

\[ E_d = \gamma_E \times E\{F_{\text{rep}} ; X_k/\gamma_M ; a_d \} \] \hspace{1cm} (2.6b)

Or indeed as

\[ E_d = \gamma_E \times E\{F_{\text{rep}} ; X_k ; a_d \} \]

In these expressions, the partial factor \( \gamma_E \) is applied to the sum of effects of the actions after they are determined, with no factoring of the actions themselves. In effect, the uncertainties about the actions have been subsumed into the uncertainty about their effects in generating shear forces and bending moments in the geotechnical structure.

Design values of resistances

To obtain the design value of the resistance, \( R_d \), partial factors may be applied either to ground properties (\( X_k \)) or to resistance (R), or to both, in three ways:

\[ R_d = R \{ \gamma_F \times F_{\text{rep}} ; X_k/\gamma_M ; a_d \} \] \hspace{1cm} (2.7a)

\[ R_d = R \{ \gamma_F \times F_{\text{rep}} ; X_k ; a_d \} / \gamma_R \] \hspace{1cm} (2.7b)

\[ R_d = R \{ \gamma_F \times F_{\text{rep}} ; X_k/\gamma_M ; a_d \} / \gamma_R \] \hspace{1cm} (2.7c)

where \( \gamma_R \) is a partial factor that caters for uncertainty in the resistance when it is calculated from ground strength or when it is assessed from measurements of resistance, for example from a pile test.

In expression 2.7a, the design value of the resistance is obtained by applying the partial factor \( \gamma_M > 1.0 \) to the characteristic values of the ground strength parameters \( c_k' \) and \( \phi_k' \) or \( c_{uk} \) etc, but with no ‘resistance factor’, \( \gamma_R \). If actions play a role in the resistance, design values of actions (\( \gamma_F \times F_{\text{rep}} \)) are introduced into the calculation of \( R_d \).

In expression 2.7b, the design value of the resistance is obtained by applying the partial resistance factor \( \gamma_R > 1.0 \) to the resistance obtained using
characteristic values of the ground strength parameters. If actions play a role in the resistance, design values of actions ($\gamma_F \times F_{rep}$) are introduced in the calculation of $R_d$, but with $\gamma_F = 1.0$, so that equation 2.7b becomes:

$$R_d = R \left\{ F_{rep} ; X_k ; a_d \right\} / \gamma_R$$

Expression 2.7c is similar to 2.7a, but an (additional) resistance factor $\gamma_R > 1.0$ can be applied to obtain $R_d$.

**The Three Design Approaches**

Because in some European countries there are perceived to be different and equivalent ways to account for the interactions between geotechnical actions and resistances, EN 1997-1 offers three ‘Design Approaches’ (DAs 1, 2 and 3) for use in calculations to check that GEO and STR ULSs will not be exceeded. The 3 DAs adopt different combinations of expressions (2.6a) and (2.6b), and (2.7a), (2.7b) and (2.7c), above.

As mentioned in Section 1, only DA-1 is permitted in BSEN 1997-1. Therefore, the two other Design Approaches are not given any further coverage in this Guide. Some explanation of DAs 2 and 3 is given in the informative Annex B of BSEN 1997-1, while additional discussion and comparisons of results using the 3 DAs may be found in Frank et al (2004).

We have seen that expressions (2.6) and (2.7) can be applied in different ways. These lead to different ‘sets’ of values for the partial factors $\gamma_F$ (or $\gamma_E$), $\gamma_M$ and $\gamma_R$. To help with understanding how to apply these sets in the alternative forms of the expressions, EN 1997-1 simplifies them symbolically as follows:

$$A \; “+” \; M \; “+” \; R$$

where:

- the partial factors for the actions $\gamma_F$ or for the effects of actions $\gamma_E$ are represented by the symbol A;
- the partial factors $\gamma_M$ for (material) parameters of the ground are represented by the symbol M.
- the partial factors for resistance $\gamma_R$ are represented by the symbol R.
- the symbol “+” means ‘used in combination with’.

In EN1997-1, *recommended* values for the different sets of factors are given in Annex A. While Annex A is ‘normative’, which means that the factors and sets of factors must be used, the *values* of the factors are ‘informative’; this means that alternative *values* may be given in National Annexes (hence the use of the word ‘recommended’). Annex A contains 17 tables of sets of factor values to cover the three Design Approaches and the different limit states, EQU, GEO, STR, UPL and HYD.

The National Annex to BSEN 1997-1 will provide the partial factor values to be used in DA-1 for design in Britain. Section 7 contains a discussion of the application of the National Annex.
For the design of piles and anchorages, DA-1 departs from its general approach of always factoring material properties. Since most routine pile (and anchorage) design is either based upon, or has evolved from, the results of test measurements of the resistance, DA-1 adopts a resistance factoring approach for piles (and anchorages). The design value of their resistance is, therefore, calculated using a partial resistance factor ($\gamma_R > 1.0$) from set R4 of Tables A.6 to A.8 or A.12 in Annex A that is applied to a characteristic resistance value. Where a pile will be designed using, say, the $\alpha$ method with $c_u$, the characteristic value of the resistance is first calculated using the characteristic value $c_{uk}$ (in effect not applying $\gamma_M$) and then the design value of the resistance is calculated by applying $\gamma_R$ to this characteristic resistance.

If unfavourable ground forces, such as downdrag, bear on the pile then the design values of these forces are calculated by applying a $\gamma_M$ factor (set M2) to the characteristic value of the friction force applied to the pile, while the resistance of the supporting portion of the pile is calculated using $\gamma_R$, as before.

Where it is obvious that one combination of sets of partial factors governs the design, it is not necessary to perform full calculations for the other combination. Usually, the geotechnical “sizing” is governed by Combination 2 and the structural design is governed by Combination 1. So, it is often obvious in a first step to determine the size of the geotechnical element using Combination 2 and then simply to check in a second step that this size of element is acceptable using Combination 1; similarly, it is often obvious to determine the structural strength of the resulting element using Combination 2 and, when relevant, to check it for Combination 1.

For further details of DA-1, see Simpson (2000).

In all Design Approaches the design values of structural material strength properties are taken from the relevant material code.
Appendix 9.3 - The other Ultimate Limit States

The **EQU limit state**
BSEN 1997-1 stipulates that the following inequality must be satisfied:

\[
E_{dst;d} \leq E_{stb;d} + T_d
\]  \hspace{1cm} (2.4)

This means that the design value of the destabilising action \(E_{dst;d}\) (e.g. the overturning moment from earth or water pressures) must be less than the design value of the stabilising action \(E_{stb;d}\) (e.g. the restoring moment due to the weight of the structure) plus any contribution from shearing resistance, \(T_d\), on the sides of structures in the ground (the contribution from \(T_d\) should be of minor significance).

Partial factors to apply for checking the EQU limit state in persistent and transient design situations are given in *Tables A.1 and A.2 of Annex A of BSEN 1997-1*.

The **UPL limit state**
The UPL limit state applies in circumstances such as where a new building basement will be excavated below the water table so that uplift water pressures may require to be resisted by, say, piling the basement slab.

To check that failure will not occur, BSEN 1997-1 requires the following inequality to be satisfied:

\[
V_{dstd} \leq G_{stb;d} + R_d
\]  \hspace{1cm} (2.8)

where \(V_{dstd}\) is the sum of \(G_{dstd}\) and \(Q_{dstd}\), the design values of the permanent and variable destabilising actions, such as water pressures under the structure and any other upward or pull-out force, and \(G_{stb;d}\) and \(R_d\) are, respectively, the design values of the stabilising permanent actions, such as the weight of the structure and/or of the ground, and the resistance of any additional structures such as holding-down piles or anchorages.

The required values of partial factors for UPL design checks in persistent and transient design situations are given in *Tables A.15 and A.16 of Annex A of BSEN 1997-1*.

The **HYD limit state**
The resistance to failure by heave due to seepage of water in the ground is checked using either stresses or forces as the variables. This limit state is considered to be beyond the scope of this simple Guide. Further information may be found in Frank et al, 2004.
Appendix 9.4 – The correspondence for ground investigation and testing between BS documents and BSEN documents.

Table A4.1 shows the contents of BSEN 1997-2 and equivalent BS documents while Table A4.2 lists the CEN Testing Standards and Test Specifications and their current status.

It should be noted that a suite of documents on the identification and classification of soil and rock, that has been developed by an ISO committee, will be adopted by CEN; these documents are listed in Table A4.3, with their BS equivalents.

<table>
<thead>
<tr>
<th>Contents of BSEN 1997-2</th>
<th>Equivalent BS Codes and Standards</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. General</td>
<td>BS 5930</td>
</tr>
<tr>
<td>2. Planning of ground investigations</td>
<td></td>
</tr>
<tr>
<td>3. Soil and rock sampling and groundwater measurement</td>
<td>BS 5930</td>
</tr>
<tr>
<td>4. Field tests in soils and rocks</td>
<td>BS 5930; BS 1377 Part 9</td>
</tr>
<tr>
<td>5. Laboratory tests on soils and rocks</td>
<td>BS 1377 Parts 1 — 8; BS 5930</td>
</tr>
<tr>
<td>6. Ground investigation report</td>
<td>BS 5930</td>
</tr>
</tbody>
</table>

Annex A. List of results of geotechnical test standards
Annex B. Planning strategies for geotechnical investigations
Annex C. Example of groundwater measurement derivations
Annex D. Cone Penetration Test                               | BS 1377 Part 9; BS 5930           |
Annex E. Pressuremeter Test                                 | BS 5930                          |
Annex F. Standard Penetration Test                           | BS 1377 Part 9; BS5930            |
Annex G. Dynamic Probing                                     | BS 1377 Part 9; BS 5930           |
Annex H. Weight Sounding Test                                |                                  |
Annex I. Field vane Test                                     | BS 1377 Part 9; BS5930            |
Annex J. Flat Dilatometer Test                               |                                  |
Annex K. Plate Loading Test                                  | BS 1377 Part 9; BS 5930           |
Annex L. Detailed information on preparation of soil specimens for testing. | BS 5930                          |
Annex M. Detailed information on Tests for classification, identification and description of soils. | BS 5930                          |
Annex N. Detailed information on chemical testing of soils.  |                                  |
Annex O. Detailed information on strength index testing of soils. |                                  |
Annex P. Detailed information on strength testing of soils.  |                                  |
Annex Q. Detailed information on compressibility testing of soils. |                                  |
Annex R. Detailed information on compaction testing of soils. |                                  |
Annex S. Detailed information on permeability testing of soils. |                                  |
Annex T. Preparation of specimen for testing of rock material. |                                  |
Annex U. Classification testing of rock material.            |                                  |
Annex V. Swelling testing of rock material                   |                                  |
Annex W. Strength testing of rock material                   |                                  |
Annex X. Bibliography.                                       |                                  |
### Table A.4.2 - CEN Testing Standards

<table>
<thead>
<tr>
<th>Eurocode 7 document</th>
<th>Title</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>BSEN 1997-1 : 2005</td>
<td>Part 1 — Geotechnical design, general rules.</td>
<td>Published December 2004</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TC341 Testing Standards</th>
<th>Title</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>BSEN ISO 22475-1</td>
<td>Drilling and sampling methods, and groundwater measurements: Part 1: sampling - principles</td>
<td>Part 1 ready for public enquiry in 2004; Parts 2 &amp; 3 to follow in spring 2004</td>
</tr>
<tr>
<td>EN-ISO/TS 22475-2</td>
<td>Part 2: sampling - qualification criteria</td>
<td></td>
</tr>
<tr>
<td>EN-ISO/TS 22475-3</td>
<td>Part 3: sampling - conformity assessment</td>
<td></td>
</tr>
<tr>
<td>22476-15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>EN-ISO 22476-9</td>
<td>Vane testing</td>
<td>Drafting underway; target date for enquiry: 2005</td>
</tr>
<tr>
<td>EN-ISO 22476-2 - ISSUED</td>
<td>Dynamic probing</td>
<td>Public enquiry completed, publication in 2004 of the two standards</td>
</tr>
<tr>
<td>EN-ISO 22476-3 - ISSUED</td>
<td>Standard penetration test</td>
<td></td>
</tr>
<tr>
<td>EN-ISO 22476-4</td>
<td>Borehole expansion tests: Menard Pressuremeter</td>
<td>Drafts on Menard, Flexible dilatometer, and borehole jack tests are well advanced</td>
</tr>
<tr>
<td>EN-ISO 22476-5</td>
<td>Flexible dilatometer</td>
<td></td>
</tr>
<tr>
<td>EN-ISO 22476-6</td>
<td>Self-boring pressuremeter</td>
<td></td>
</tr>
<tr>
<td>EN-ISO 22476-7</td>
<td>Borehole jack</td>
<td></td>
</tr>
<tr>
<td>EN-ISO 22476-8</td>
<td>Full displacement pressuremeter</td>
<td></td>
</tr>
<tr>
<td>EN-ISO 22476-16</td>
<td>Borehole shear test</td>
<td></td>
</tr>
<tr>
<td>EN-ISO 22476-14</td>
<td>Plate load test</td>
<td>Drafting yet to commence</td>
</tr>
<tr>
<td>EN-ISO 22476-14</td>
<td>Pumping tests</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TC341 Technical Specifications</th>
<th>Title</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEN ISO/TS 17892-1</td>
<td>Water content</td>
<td>All have been out for editorial comment. These are not replacing BS1377 at this stage as they are only Technical specifications</td>
</tr>
<tr>
<td>CEN ISO/TS 17892-2</td>
<td>Density of fine grained soils</td>
<td></td>
</tr>
<tr>
<td>CEN ISO/TS 17892-3</td>
<td>Density of solid particles</td>
<td></td>
</tr>
<tr>
<td>CEN ISO/TS 17892-4</td>
<td>Particle size distribution</td>
<td></td>
</tr>
<tr>
<td>CEN ISO/TS 17892-5</td>
<td>Oedometer test</td>
<td></td>
</tr>
<tr>
<td>CEN ISO/TS 17892-6</td>
<td>Fall cone test</td>
<td></td>
</tr>
<tr>
<td>CEN ISO/TS 17892-7</td>
<td>Compression test</td>
<td></td>
</tr>
<tr>
<td>CEN ISO/TS 17892-8</td>
<td>Unconsolidated triaxial test</td>
<td></td>
</tr>
<tr>
<td>CEN ISO/TS 17892-9</td>
<td>Consolidated triaxial test</td>
<td></td>
</tr>
<tr>
<td>CEN ISO/TS 17892-10</td>
<td>Direct shear test</td>
<td></td>
</tr>
<tr>
<td>CEN ISO/TS 17892-11</td>
<td>Permeability test</td>
<td></td>
</tr>
<tr>
<td>CEN ISO/TS 17892-12</td>
<td>Laboratory tests on rock</td>
<td></td>
</tr>
</tbody>
</table>

*Note that these numbers are unofficial since BSI has not yet published the documents.*
### TABLE A.4.3 — CEN ISO Standards for Identification and Classification of soil and rock

<table>
<thead>
<tr>
<th>Reference</th>
<th>Title</th>
<th>Current status</th>
<th>BS equivalent</th>
</tr>
</thead>
<tbody>
<tr>
<td>14688-1</td>
<td>Soil - Identification and Description;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14688-2</td>
<td>Soil - Classification principles and quantification of descriptive characteristics</td>
<td></td>
<td>BS5930:</td>
</tr>
<tr>
<td>14689-1</td>
<td>Rock - Identification and description</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Appendix 9.5 - Significance of statistical methods

BSEN 1997-1 states that, if statistical methods are used, they should differentiate between local and regional sampling and should allow the use of a priori knowledge of comparable experience with ground properties. This demands a high order of statistical expertise, available only from a very few designers. Attempts by statisticians to tackle geotechnical design have often failed 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 (1999) has proposed that, where a spatial mean is relevant, the characteristic value might be taken as half a standard deviation from the mean. Simpson and Driscoll (1998) suggest that this rule could be a useful guide.

Further guidance on the use of statistics in determining characteristic values may be found in Frank et al (2004).
Appendix 9.6 - Calculating pile ultimate compressive resistance using static load tests

The procedure uses the values of the compressive resistance, \( R_{c,m} \), measured in one or more static load tests on piles that must be of the same type and founded in the same stratum as the working piles.

The interpretation of the test results must take into account the variability of the ground over the site and the variability of the effects of pile installation. In other words, a careful examination is required of the site investigation information, of the piling records and of the pile load test results.

The procedure is as follows:

1. From the measured resistances, \( R_{c,m} \), determine the characteristic value, \( R_{c,k} \), using the expression:

\[
R_{c,k} = \min \left\{ \frac{R_{c,m}}{\xi_1}, \frac{R_{c,m}}{\xi_2} \right\}
\]

that is, the smaller of the two terms \( \frac{R_{c,m}}{\xi_1} \) and \( \frac{R_{c,m}}{\xi_2} \), where \( \frac{R_{c,m}}{\text{mean}} \) is the mean value of all the measured compressive resistances of the test piles and \( \frac{R_{c,m}}{\min} \) is the minimum value.

The values of the correlation factors \( \xi_1 \) and \( \xi_2 \) depend on the number (n) of piles tested. Recommended values for \( \xi_1 \) and \( \xi_2 \), which are given in Table A.9 of Annex A of BSEN 1997-1, are, for convenience, reproduced in Table A6.1.

<table>
<thead>
<tr>
<th>( \xi ) for n =</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>( \geq 5 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \xi_1 ) (mean)</td>
<td>1.4</td>
<td>1.3</td>
<td>1.2</td>
<td>1.1</td>
<td>1.0</td>
</tr>
<tr>
<td>( \xi_2 ) (min.)</td>
<td>1.4</td>
<td>1.2</td>
<td>1.05</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Table A.6.1 Values of correlation factors for static pile load test results
(n is number of tested piles)

Consider the stiffness and strength of the structure connecting the piles. If the structure is able to transfer loads from weaker piles to stronger piles, the values for \( \xi_1 \) and \( \xi_2 \) may be divided by 1.1 provided \( \xi_1 \) does not fall below 1.0.

If appropriate, \( R_{c,k} \) can be separated into characteristic values for the shaft frictional resistance, \( R_{s,k} \), and for the base resistance, \( R_{b,k}^2 \).

\[ \text{\footnotesize\textsuperscript{1}} \text{It can be seen in the Table that, as the } \xi_1 \text{ factor values reduce with more test results, the value of } R_{c,k} \text{ becomes less conservative. It can also be seen that the scatter in results (expressed by the differences between the mean and minimum values) is adjusted by differences in the } \xi_1 \text{ and } \xi_2 \text{ values, depending on the number of tests.} \]

\[ \text{\footnotesize\textsuperscript{2}} \text{This may be appropriate when measurements of shaft resistance have been made or when calculations have been made using the results of ground or dynamic pile tests.} \]
2. The design resistance is then obtained by applying the partial factor $\gamma_t$ to the total characteristic resistance $R_{c,k}$ or by applying the partial factors $\gamma_s$ and $\gamma_b$ respectively to the characteristic shaft frictional resistance and to the characteristic base resistance, thus:

$$R_{c:d} = R_{c,k}/\gamma_t$$

or

$$R_{c:d} = R_{b,k}/\gamma_b + R_{s,k}/\gamma_s$$

Tables A.6 to A.8 of Annex A list the values for these factors; for convenience, they have been drawn together in Table 4.1.

3. In the case of groups of piles, the group bearing capacity is determined either by summing the individual pile compressive resistances or by assuming a ‘block’ failure, whichever is the lower value.

4. If a number of load tests performed over the site have indicated the presence of different zones of similar results, the procedure from step 1 may be repeated for each zone\(^1\).

---

\(^1\) This can lead to a less conservative result than if the minimum value of $R_{c,m}$ is assumed to govern the whole site.
Appendix 9.7 - Calculating pile ultimate compressive resistance using the results of dynamic testing

The different techniques considered are:

- the dynamic impact (hammer blow) test.
- pile driving records.
- wave equation analysis results.

The procedure is the same for all 3 three tests:

The characteristic value of the compressive resistance, $R_{c,k}$, is determined from:

$$R_{c,k} = \min / \left\{ \frac{(R_{c,m})_{\text{mean}}}{\xi_5} ; \frac{(R_{c,m})_{\text{min}}}{\xi_6} \right\}$$  \hspace{1cm} (7.10)

where $R_{c,m}$ is the mean, static, compressive resistance derived from the dynamic measurements and $\xi_5$ and $\xi_6$ are correlation factors the values of which depend on the number of piles tested, $n$; $\xi_5$ and $\xi_6$ are applied respectively to the mean of the results, $(R_{c,m})_{\text{mean}}$, and to the lowest value, $(R_{c,m})_{\text{min}}$. Values for $\xi_5$ and $\xi_6$ are given in Table A.11 of Annex A of BSEN 1997-1 and are reproduced in Table A7.1 for convenience. Note that Table A.11 has a number of qualifying remarks about the $\xi$ values, depending on the nature of the testing.

<table>
<thead>
<tr>
<th>$\xi$ for $n =$</th>
<th>$\geq 2$</th>
<th>$\geq 5$</th>
<th>$\geq 10$</th>
<th>$\geq 15$</th>
<th>$\geq 20$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\xi_5$ (mean)</td>
<td>1.6</td>
<td>1.5</td>
<td>1.45</td>
<td>1.42</td>
<td>1.40</td>
</tr>
<tr>
<td>$\xi_6$ (min.)</td>
<td>1.5</td>
<td>1.35</td>
<td>1.3</td>
<td>1.25</td>
<td>1.25</td>
</tr>
</tbody>
</table>

Table A.7.1 Values of correlation factors for dynamic impact tests  
($n$ is the number of tested piles)

The design compressive resistance is then obtained from:

$$R_{c,d} = R_{c,k}/\gamma$$

For persistent and transient design situations, values of $\gamma$ are reproduced from BSEN 1997-1 Annex A in Table 4.1. For accidental situations, $\gamma = 1.0$. 
Appendix 9.8 - Tensile resistance of piles

Methods for determining the ground resistance for piles in tension, either from static pile load tests or from ground test results, are very similar to those for piles in compression. The main differences are:

- The base resistance is ignored;
- For stiff and strong structures, the correlation factors $\xi$ may not be divided by 1.1. This is because the failure of tensile piles is usually brittle;
- It is common practice to be rather more cautious when applying safety to tensile piles than to compressive piles, so that the value of the partial factor on shaft tension resistance, $\gamma_s; t$, is larger than that on shaft compression resistance, $\gamma_s$, (see Table 4.1).

Otherwise, the design of piles in tension is the same as for piles in compression, with the correlation factors $\xi_1$, $\xi_2$, $\xi_3$ and $\xi_4$ having the same values for both designs.

A number of issues specific to pile in tension are relevant:

- Where piles interact in a group, tension on one pile reduces the effective vertical stresses around adjacent piles; this reduces their resistance and consequently the resistance of the whole foundation.
- Cyclic loading, or reversal of load, can be highly detrimental to pile tensile capacity and must therefore be considered. The code does not give advice on this matter and experts must be consulted.
- Because of the different magnitudes of $\gamma_G$ and $\gamma_Q$, a permanent, characteristic, compressive load acting with a variable, characteristic, tensile load of similar magnitude can lead to a resultant tensile design load on the pile, although the resultant characteristic load may be compressive.

For groups of piles in tension, special consideration must be given to the possible failure in uplift of the block of ground containing the piles.

Two ultimate limit states must be checked:

- pull-out failure (tensile resistance) of individual piles;
- uplift failure of the block of ground containing the piles.

Block failure

Block failure of a group of piles occurs when all piles and the ground between them are lifted by the tensile loading. The loading may come from a structure above the water level or from water pressure acting upwards on the structure.

BSEN 1997-1 requires failure of the block to be treated as an uplift (UPL) limit state, with corresponding partial factors (see Tables A.15 and A.16 in Annex A, as well as Section 2 in this Guide).
Section 9 - Appendices

For both isolated piles and groups of piles, BSEN 1997-1 recommends checking the pull-out mechanism involving a cone of ground surrounding the piles, especially when an enlarged pile base or rock socket is used.