

Eurocode 7: Geotechnical design —

Part 1: General rules

ICS 91.010.30; 93.020

National foreword

This British Standard is the UK implementation of EN 1997-1:2004+A1:2013, incorporating corrigendum February 2009. It supersedes BS EN 1997-1:2004, which is withdrawn.

The start and finish of text introduced or altered by corrigendum is indicated in the text by tags. Text altered by CEN corrigendum February 2009 is indicated in the text by **AC1** **AC1**.

The start and finish of text introduced or altered by amendment is indicated in the text by tags. Tags indicating changes to CEN text carry the number of the CEN amendment. For example, text altered by CEN amendment A1 is indicated by **A1** **A1**.

EN 1997-1 was published as a British Standard in 2004, and the UK National Annex was added in 2007. Part 2 of Eurocode 7, EN 1997-2 was published in 2007 followed by its UK National Annex in 2009. Older related British Standards have been withdrawn or revised as noted below. The Department for Communities and Local Government (DCLG) published a letter, dated 29 January 2010, which explains how this relates to Building Regulations in England and Wales and discusses the use of both current and withdrawn standards (<http://www.communities.gov.uk/documents/planningandbuilding/pdf/1454859.pdf>). Only BS EN 1997 Parts 1 and 2 are listed under “Geotechnical work and foundations” in UK Building Regulations Approved Document A, as amended in 2013.

The following British Standards were withdrawn on 31 March 2010:

- BS 8002:1994, *Code of practice for earth retaining structures*;
- BS 8004:1986, *Code of practice for foundations*.

Withdrawal means that these documents are no longer current British Standards and are not maintained by BSI, though they will remain available for purchase. It is planned that extensively revised versions of BS 8002 and BS 8004 will be published in 2015.

The following British Standards have been retained after extensive revision to update them and make them more compatible with Eurocodes:

- BS 5930:1999+A2:2010, *Code of practice for site investigations*;
- BS 6031:2009, *Code of practice for earthworks*;
- BS 8006-1:2010, *Code of practice for strengthened/reinforced soils and other fills*;
- BS 8006-2:2011, *Code of practice for strengthened/reinforced soils – Part 2: Soil nail design*.

It is possible that further changes will be made to these standards, or they could be withdrawn, depending on future development of Eurocodes. (The letter of DCLG dated 29 January 2010 incorrectly states that BS 8006:1995 was to be withdrawn on 31 March 2010. BS 8006:1995 was withdrawn on 31 December 2011.)

This British Standard was published under the authority of the Standards Policy and Strategy Committee on 22 December 2004

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Amendments/corrigenda issued since publication

Date	Comments
31 January 2010	Implementation of CEN corrigendum February 2009
31 July 2014	Implementation of CEN amendment A1:2013

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The following British Standards remain current:

- BS 8081:1989, *Code of practice for ground anchorages*.

(The letter of DCLG dated 29 January 2010 incorrectly states that this standard was to be withdrawn on 31 March 2010.) It is planned that an extensively revised version of BS 8081 will be published in 2015.

- BS 8008:1996+A1:2008, *Safety precautions and procedures for the construction and descent of machine-bored shafts for piling and other purposes*.

The usage of BS 8081:1989 is limited, as noted in the National Annex. BS 8008:1996 contains non-contradictory complementary information as stated in the National Annex.

The UK participation in its preparation was entrusted to Technical Committee B/526, Geotechnics.

A list of organizations represented on this committee can be obtained on request to its secretary.

Where a normative part of this EN allows for a choice to be made at the national level, the range and possible choice will be given in the normative text, and a note will qualify it as a Nationally Determined Parameter (NDP). NDPs can be a specific value for a factor, a specific level or class, a particular method or a particular application rule if several are proposed in the EN.

To enable EN 1997-1 to be used in the UK, the NDPs are published in the National Annex.

The publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a British Standard cannot confer immunity from legal obligations.

English version

Eurocode 7: Geotechnical design - Part 1: General rules

Eurocode 7: Calcul géotechnique - Partie 1: Règles
générales

Eurocode 7: Entwurf, Berechnung und Bemessung in der
Geotechnik - Teil 1: Allgemeine Regeln

This European Standard was approved by CEN on 23 April 2004.

CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the Central Secretariat or to any CEN member.

This European Standard exists in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Central Secretariat has the same status as the official versions.

CEN members are the national standards bodies of Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.



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Foreword

This document (EN 1997-1) has been prepared by Technical Committee CEN/TC250 “Structural Eurocodes”, the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes.

This European Standard shall be given the status of a national standard, either by publication of an identical text, or by endorsement, at the latest by **May 2005** and conflicting national standards shall be withdrawn by **March 2010**.

This document supersedes ENV 1997-1:1994.

According to the CEN/CENELEC Internal Regulations, the national standards organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

Foreword to amendment A1

This document (EN 1997-1:2004/A1:2013) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI.

This Amendment to the European Standard EN 1997-1:2004 shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by November 2014, and conflicting national standards shall be withdrawn at the latest by November 2014.

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. CEN shall not be held responsible for identifying any or all such patent rights.

According to the CEN-CENELEC Internal Regulations, the national standards organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Bulgaria, Croatia, Cyprus, Czech Republic, Denmark, Estonia, Finland, Former Yugoslav Republic of Macedonia, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Romania, Slovakia, Slovenia, Spain, Sweden, Switzerland, Turkey and the United Kingdom.

Background to the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980s.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement¹ between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commissions Decisions dealing with European standards (e.g. the Council Directive 89/106/EEC on construction products - CPD - and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

EN 1990	Eurocode :	Basis of Structural Design
EN 1991	Eurocode 1:	Actions on structures
EN 1992	Eurocode 2:	Design of concrete structures
EN 1993	Eurocode 3:	Design of steel structures
EN 1994	Eurocode 4:	Design of composite steel and concrete structures
EN 1995	Eurocode 5:	Design of timber structures
EN 1996	Eurocode 6:	Design of masonry structures
EN 1997	Eurocode 7:	Geotechnical design
EN 1998	Eurocode 8:	Design of structures for earthquake resistance
EN 1999	Eurocode 9:	Design of aluminium structures

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes:

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire;
- as a basis for specifying contracts for construction works and related engineering services;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

¹ Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents² referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards³. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex.

The National annex may only contain information on those parameters, which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e. :

- values and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- country specific data (geographical, climatic), *e.g.* snow map,
- the procedure to be used where alternative procedures are given in the Eurocode.

It may also contain:

- decisions on the application of informative annexes,
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works⁴. Furthermore, all the information accompanying the CE Marking of the construction products, which refer to Eurocodes should clearly mention which Nationally Determined Parameters have been taken into account.

2 According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.

3 According to Art. 12 of the CPD the interpretative documents shall :

- a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;
 - b) indicate methods of correlating these classes or levels of requirement with the technical specifications, *e.g.* methods of calculation and of proof, technical rules for project design, etc. ;
 - c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.
- The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

4 see Art.3.3 and Art.12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.

Additional information specific to Eurocode 7

EN 1997-1 gives design guidance and actions for geotechnical design of buildings and civil engineering works.

EN 1997-1 is intended for clients, designers, contractors and public authorities.

EN 1997-1 is intended to be used with EN 1990 and EN 1991 to EN 1999.

In using EN 1997-1 in practice, particular regard should be paid to the underlying assumptions and conditions given in 1.3.

The 12 sections of EN 1997-1 are complemented by 1 normative and 8 informative annexes.

National annex for EN 1997-1

This standard gives alternative procedures and recommended values with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1997-1 should have a National annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1997-1 through the following paragraphs:

- 2.1(8)P, 2.4.6.1(4)P, 2.4.6.2(2)P, 2.4.7.1(2)P, 2.4.7.1(3), AC1 2.4.7.1(4), 2.4.7.1(5), 2.4.7.1(6), AC1 2.4.7.2(2)P, 2.4.7.3.2(3)P, 2.4.7.3.3(2)P, 2.4.7.3.4.1(1)P, 2.4.7.4(3)P, 2.4.7.5(2)P, 2.4.8(2), 2.4.9(1)P, 2.5(1), 7.6.2.2(8)P, 7.6.2.2(14)P, 7.6.2.3(4)P, 7.6.2.3(5)P, 7.6.2.3(8), 7.6.2.4(4)P, 7.6.3.2(2)P, 7.6.3.2(5)P, 7.6.3.3(3)P, 7.6.3.3(4)P, 7.6.3.3(6), A1 8.4(6)P, 8.4(7)P, 8.5.1(1)P, 8.5.1(2)P, 8.5.2(1)P, 8.5.2(2)P, 8.5.2(3)P, 8.5.2(5)P, 8.5.3(1)P, 8.5.3(2)P, 8.5.3(3)P, 8.5.3(4)P, 8.6.2(2)P, 8.6.2(3)P, A1 AC1 10.2(3), AC1 11.5.1(1)P

and the following clauses in annex A:

- A.2

- A.3.1, A.3.2, A.3.3.1, A.3.3.2, A.3.3.3, A1 *Text deleted* A1 A.3.3.5, A.3.3.6,

- A.4

- A.5

- A1 — A.6 A1

Section 1 General

1.1 Scope

1.1.1 Scope of EN 1997

- (1) EN 1997 is intended to be used in conjunction with EN 1990:2002, which establishes the principles and requirements for safety and serviceability, describes the basis of design and verification and gives guidelines for related aspects of structural reliability.
- (2) EN 1997 is intended to be applied to the geotechnical aspects of the design of buildings and civil engineering works. It is subdivided into various separate parts (see 1.1.2 and 1.1.3).
- (3) EN 1997 is concerned with the requirements for strength, stability, serviceability and durability of structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.
- (4) Numerical values of actions on buildings and civil engineering works to be taken into account in design are provided in EN 1991 for the various types of construction. Actions imposed by the ground, such as earth pressures, shall be calculated according to the rules of EN 1997.
- (5) Separate European Standards are intended to be used to treat matters of execution and workmanship. They are denoted in the relevant sections.
- (6) In EN 1997 execution is covered to the extent that is necessary to comply with the assumptions of the design rules.
- (7) EN 1997 does not cover the special requirements of seismic design. EN 1998 provides additional rules for geotechnical seismic design, which complete or adapt the rules of this Standard.

1.1.2 Scope of EN 1997-1

- (1) EN 1997-1 is intended to be used as a general basis for the geotechnical aspects of the design of buildings and civil engineering works.
- (2) The following subjects are dealt with in EN 1997-1:

Section 1: General

Section 2: Basis of geotechnical design

Section 3: Geotechnical data

Section 4: Supervision of construction, monitoring and maintenance

Section 5: Fill, dewatering, ground improvement and reinforcement

Section 6: Spread foundations

Section 7: Pile foundations

A1 Section 8: Anchors **A1**

Section 9: Retaining structures

Section 10: Hydraulic failure

Section 11: Overall stability

Section 12: Embankments

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

- in A: recommended γ_{AC1} partial factor γ_{AC1} values; different values of the partial factors may be set by the National annex;
- in B to J: supplementary informative guidance such as internationally applied calculation methods.

1.1.3 Further Parts of EN 1997

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

1.2 Normative references

(1) This European Standard incorporates by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies (including amendments).

NOTE The Eurocodes were published as European Prestandards. The following European Standards which are published or in preparation are cited in normative clauses

EN 1990:2002	Eurocode: Basis of structural design
EN 1991	Eurocode 1 Actions on structures
EN 1991-4	Eurocode 1 Actions on structures - Part 4 Actions in silos and tanks
EN 1992	Eurocode 2 Design of concrete structures
γ_{A1} EN 1992-1-1:2004	Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings γ_{A1}
EN 1993	Eurocode 3 Design of steel structures
EN 1994	Eurocode 4 Design of composite steel and concrete structures
EN 1995	Eurocode 5 Design of timber structures
EN 1996	Eurocode 6 Design of masonry structures
EN 1997-2	Eurocode 7 Geotechnical design - Part 2: Ground investigation and testing
EN 1998	Eurocode 8 Design of structures for earth quake resistance
EN 1999	Eurocode 9 Design of aluminium and aluminium alloy structures
EN 1536:1999	Execution of special geotechnical work: Bored piles
γ_{A1} EN 1537	Execution of special geotechnical work; Ground anchors γ_{A1}
EN 12063:1999	Execution of special geotechnical work; Sheet-pile walls

EN 12699:2000	Execution of special geotechnical work; Displacement piles
EN 14199	Execution of special geotechnical works – Micropiles
EN-ISO 13793: 2001	Thermal performance of buildings – Thermal design of foundations to avoid frost heave
 EN ISO 22477-5	Geotechnical investigation and testing – testing of geotechnical structures – Part 5: Testing of anchors (in preparation) 

1.3 Assumptions

(1) Reference is made to 1.3 of EN 1990:2002.

(2) The provisions of this standard are based on the assumptions given below:

- data required for design are collected, recorded and interpreted by appropriately qualified personnel;
- structures are designed by appropriately qualified and experienced personnel;
- adequate continuity and communication exist between the personnel involved in data-collection, design and construction;
- adequate supervision and quality control are provided in factories, in plants, and on site;
- execution is carried out according to the relevant standards and specifications by personnel having the appropriate skill and experience;
- construction materials and products are used as specified in this standard or in the relevant material or product specifications;
- the structure will be adequately maintained to ensure its safety and serviceability for the designed service life;
- the structure will be used for the purpose defined for the design.

(3) These assumptions need to be considered both by the designer and the client. To prevent uncertainty, compliance with them should be documented, e.g. in the geotechnical design report.

1.4 Distinction between Principles and Application Rules

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

(2) The Principles comprise:

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

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

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

(5) It is permissible to use alternatives to the Application Rules given in this standard, provided it is shown that the alternative rules accord with the relevant Principles and are at least

equivalent with regard to the structural safety, serviceability and durability, which would be expected when using the Eurocodes.

NOTE If an alternative design rule is submitted for an application rule, the resulting design cannot be claimed to be wholly in accordance with EN 1997-1, although the design will remain in accordance with the Principles of EN 1997-1. When EN 1997-1 is used in respect of a property listed in an Annex Z of a product standard or an ETAG, the use of an alternative design rule may not be acceptable for CE marking.

(6) In EN 1997-1, the Application rules are identified by a number in brackets e.g. as in this clause.

1.5 Definitions

1.5.1 Definitions common to all Eurocodes

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

1.5.2 Definitions specific for EN 1997-1

1.5.2.1

geotechnical action

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

NOTE Definition taken from EN 1990:2002

1.5.2.2

comparable experience

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

1.5.2.3

ground

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

1.5.2.4

structure

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

NOTE Definition derived from EN 1990:2002

1.5.2.5

derived value

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

1.5.2.6

stiffness

material resistance against deformation

1.5.2.7

resistance

capacity of a component, or cross-section of a component of a structure to withstand actions without mechanical failure e.g. resistance of the ground, bending resistance, buckling resistance, tensile resistance

NOTE Definition derived from EN 1990:2002

1.6 Symbols

(1) For the purpose of EN 1997-1 the following symbols apply.

Latin letters

A'	effective base area ($A' = B' \times L'$)
A_b	base area under pile
A_c	total base area under compression
$A_{s,i}$	pile shaft surface area in layer i
a_d	design value of geometrical data
a_{nom}	nominal value of geometrical data
Δa	change made to nominal geometrical data for particular design purposes
B	width of a foundation.
b'	effective width of a foundation
C_d	limiting design value of the relevant serviceability criterion
c	cohesion intercept
c'	cohesion intercept in terms of effective stress
c_u	undrained shear strength
$c_{u,d}$	design value of undrained shear strength
d	embedment depth;
E_d	design value of the effect of actions
$E_{stb;d}$	design value of the effect of stabilising actions
$E_{dst;d}$	design value of the effect of destabilising actions
$F_{c;d}$	design axial compression load on a pile or a group of piles
F_d	design value of an action
F_k	characteristic value of an action
F_{rep}	representative value of an action
$F_{t;d}$	design axial tensile load on a tensile pile or a group of tensile piles

$F_{tr,d}$	design value of the transverse load on a pile or a pile foundation
$G_{dst,d}$	design value of the destabilising permanent actions for uplift verification
$G_{stb,d}$	design value of the stabilising permanent vertical actions for uplift verification
$G'_{stb,d}$	design value of the stabilising permanent vertical actions for heave verification (submerged weight)
H	horizontal load, or component of total action acting parallel to the foundation base
H_d	design value of H
h	height of a wall
h	water level for hydraulic heave
h'	height of a soil prism for verifying hydraulic heave
$h_{w,k}$	characteristic value of the hydrostatic water head at the bottom of a soil prism
K_0	coefficient of earth pressure at rest
$K_{0,\beta}$	coefficient of earth pressure at rest for a retained earth surface inclined at angle β to the horizontal
k	ratio $\delta_d/\varphi_{cv,d}$
$\boxed{AC1} L \boxed{AC1}$	foundation length;
l	effective foundation length
n	number of e.g. piles or test profiles
$\boxed{A1} \text{ text deleted } \boxed{A1}$	
P_d	design value of P
$\boxed{A1} \text{ text deleted } \boxed{A1}$	
$Q_{dst,d}$	design value of the destabilising variable vertical actions for uplift verification
$\boxed{AC1} q_{b,k}$	characteristic value of unit base resistance $\boxed{AC1}$
$\boxed{AC1} q_{s,i,k}$	characteristic value of unit shaft resistance in stratum i $\boxed{AC1}$
$\boxed{AC1} q_u$	unconfined compressive strength $\boxed{AC1}$
$\boxed{A1} \text{ text deleted } \boxed{A1}$	
$\boxed{A1} \text{ text deleted } \boxed{A1}$	
$\boxed{A1} \text{ text deleted } \boxed{A1}$	
$R_{b,cal}$	pile base resistance, calculated from ground test results, at the ultimate limit state

$R_{b;d}$	design value of the base resistance of a pile
$R_{b;k}$	characteristic value of the base resistance of a pile
R_c	compressive resistance of the ground against a pile, at the ultimate limit state
$R_{c;cal}$	calculated value of R_c
$R_{c;d}$	design value of R_c
$R_{c;k}$	characteristic value of R_c
$R_{c;m}$	measured value of R_c in one or several pile load tests
R_d	design value of the resistance to an action
$R_{p;d}$	design value of the resisting force caused by earth pressure on the side of a foundation
$R_{s;d}$	design value of the shaft resistance of a pile
$R_{s;cal}$	ultimate shaft friction, calculated using ground parameters from test results
$R_{s;k}$	characteristic value of the shaft resistance of a pile
R_t	ultimate tensile resistance of an isolated pile
$\boxed{A_1} R_{t;d}$	design value of the tensile resistance of a pile or of a group of piles $\boxed{A_1}$
$R_{t;k}$	characteristic value of the tensile resistance of a pile or a pile group
$R_{t;m}$	measured tensile resistance of an isolated pile in one or several pile load tests
R_{tr}	resistance of a pile to transverse loads
$R_{tr;d}$	design resistance of transversally loaded pile
$S_{dst;d}$	design value of the destabilising seepage force in the ground
$S_{dst;k}$	characteristic value of the destabilising seepage force in the ground
s	settlement
s_0	immediate settlement
s_1	settlement caused by consolidation
s_2	settlement caused by creep (secondary settlement)
T_d	design value of total shearing resistance that develops around a block of ground in which a group of tension piles is placed, or on the part of the structure in contact with the ground
u	pore-water pressure

$u_{dst;d}$	design value of destabilising total pore-water pressure
V	vertical load, or component of the total action acting normal to the foundation base
V_d	design value of V
V'_d	design value of the effective vertical action or component of the total action acting normal to the foundation base
$V_{dst;d}$	design value of the destabilising vertical action on a structure
$V_{dst;k}$	characteristic value of the destabilising vertical action on a structure
X_d	design value of a material property
X_k	characteristic value of a material property
z	vertical distance

A1 Latin letters used in Section 8

$E_{ULS;d}$	ultimate limit state design force to be resisted by the anchor
$F_{Serv;d}$	design value of the maximum anchor force, including effect of lock off load, and sufficient to prevent a serviceability limit state in the supported structure
$F_{Serv;k}$	characteristic value of the maximum anchor force, including effect of lock off load, and sufficient to prevent a serviceability limit state in the supported structure
$F_{ULS;d}$	design value of the force required to prevent any ultimate limit state in the supported structure
$k_{l;SLS}$	permissible cumulative loss of load over specified time period, defined in EN ISO 22477-5, used to determine serviceability limit state resistance of an anchor
$k_{l;ULS}$	permissible cumulative loss of load over specified time period, defined in EN ISO 22477-5, used to determine ultimate limit state resistance of an anchor
n	minimum number of investigation and/or suitability tests
P_c	the critical creep load, determined as the load corresponding to the end of the pseudo linear part of the α versus load diagram as defined in EN ISO 22477-5
P_o	lock-off load, load left in the anchor head immediately on completion of the stressing operation
P_p	proof load, maximum test load to which an anchor is subjected in a particular load test
R_m	measured value of the resistance of an anchor
$R_{SLS;d}$	design value of the resistance of an anchor complying with serviceability limit state criteria
$R_{SLS;k}$	characteristic value of the resistance of an anchor complying with serviceability limit state criteria
$R_{SLS;m}$	measured value of the resistance of an anchor complying with serviceability limit state criteria
$(R_{SLS;m})_{min}$	lowest value of $R_{SLS;m}$ measured from a number of investigation or suitability tests (n), for each distinct ground condition
$R_{t;d}$	ultimate limit state design resistance of the structural elements of an anchor

$R_{ULS;d}$ design value of the resistance of an anchor complying with ultimate limit state criteria

$R_{ULS;k}$ characteristic value of the resistance of an anchor complying with ultimate limit state criteria

$R_{ULS;m}$ measured value of the resistance of an anchor complying with ultimate limit state criteria

$(R_{ULS;m})_{min}$ lowest value of $R_{ULS;m}$ measured from a number of investigation or suitability tests (n), for each distinct ground condition

(end text)

After " ϕ'_d " design value of ϕ' ", add:

(begin text)

Greek letters used in Section 8

α_{SLS} creep rate defining serviceability limit state resistance of an anchor, determined from the displacement per log cycle of time at constant anchor load as defined in EN ISO 22477-5

α_{ULS} creep rate defining ultimate limit state resistance of an anchor, determined from the displacement per log cycle of time at constant anchor load as defined in EN ISO 22477-5

$\gamma_{Serv}, \gamma_{a;ULS}, \gamma_{a;SLS}, \gamma_{a;acc;ULS}, \gamma_{a;acc;SLS}$ partial factors

ξ_{ULS} correlation factor $\sqrt{A_1}$

Greek letters

α inclination of a foundation base to the horizontal

β slope angle of the ground behind a wall (upward positive)

δ structure-ground interface friction angle

δ_d design value of δ

γ weight density

$\sqrt{A_1}$ text deleted $\sqrt{A_1}$

$\sqrt{A_1}$ text deleted $\sqrt{A_1}$

$\sqrt{A_1}$ text deleted $\sqrt{A_1}$

$\gamma_{a;t}$ partial factor for temporary anchorages

γ_b partial factor for the base resistance of a pile

γ_c partial factor for the effective cohesion

γ_{cu} partial factor for the undrained shear strength

γ_E partial factor for the effect of an action

γ	partial factor for actions, which takes account of the possibility of unfavourable deviations of the action values from the representative values
γ_F	partial factor for an action
γ_G	partial factor for a permanent action
$\gamma_{G,dst}$	partial factor for a permanent destabilising action
$\gamma_{G,stb}$	partial factor for a permanent stabilising action
γ_m	partial factor for a soil parameter (material property)
$\gamma_{m,i}$	partial factor for a soil parameter in stratum i
γ_M	partial factor for a soil parameter (material property), also accounting for model uncertainties
γ_Q	partial factor for a variable action
γ_{qu}	partial factor for unconfined strength
γ_R	partial factor for a resistance
$\gamma_{R,d}$	partial factor for uncertainty in a resistance model
$\gamma_{R,e}$	partial factor for passive earth resistance
$\gamma_{R,h}$	partial factor for sliding resistance
$\gamma_{R,v}$	partial factor for bearing resistance
γ_s	partial factor for shaft resistance of a pile
$\gamma_{S,d}$	partial factor for uncertainties in modelling the effects of actions
$\gamma_{Q,dst}$	partial factor for a variable destabilising action
$\gamma_{Q,stb}$	partial factor for a variable stabilising action
$\gamma_{s,t}$	partial factor for tensile resistance of a pile
γ	partial factor for total resistance of a pile
γ_w	weight density of water
γ_ϕ	partial factor for the angle of shearing resistance ($\tan \phi$)
γ_γ	partial factor for weight density
θ	direction angle of H
ξ	correlation factor depending on the number of piles tested or of profiles of tests

A1 text deleted A1

- ξ_1, ξ_2 correlation factors to evaluate the results of static pile load tests
- ξ_3, ξ_4 correlation factors to derive the pile resistance from ground investigation results, not being pile load tests.
- ξ_5, ξ_6 correlation factors to derive the pile resistance from dynamic impact tests
- ψ factor for converting the characteristic value to the representative value
- $\sigma_{\text{stb;d}}$ design value of stabilising total vertical stress
- $\sigma'_{h;0}$ horizontal component of effective earth pressure at rest
- $\sigma(z)$ stress normal to a wall at depth z
- $\tau(z)$ stress tangential to a wall at depth z
- ϕ angle of shearing resistance in terms of effective stress
- ϕ_{cv} critical state angle of shearing resistance
- $\phi_{\text{cv;d}}$ design value of ϕ_{cv}
- ϕ'_{d} design value of ϕ

Abbreviations

CFA Continuous flight auger piles

OCR over-consolidation ratio

NOTE 1 The symbols commonly used in all Eurocodes are defined in EN 1990:2002

NOTE 2 The notation of the symbols used is based on ISO 3898:1997.

(2) For geotechnical calculations, the following units or their multiples are recommended:

— force	kN
— mass	kg
— moment	kNm
— mass density	kg/m ³
— weight density	kN/m ³
— stress, pressure, strength and stiffness	kPa
— coefficient of permeability	m/s
— coefficient of consolidation	m ² /s

Section 2 Basis of geotechnical design

2.1 Design requirements

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

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

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

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

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

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

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

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

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

(8)P In order to establish minimum requirements for the extent and content of geotechnical investigations, calculations and construction control checks, the complexity of each geotechnical design shall be identified together with the associated risks. In particular, a distinction shall be made between:

- light and simple structures and small earthworks for which it is possible to ensure that the minimum requirements will be satisfied by experience and qualitative geotechnical investigations, with negligible risk;
- other geotechnical structures.

NOTE The manner in which these minimum requirements are satisfied may be given in the National annex.

(9) For structures and earthworks of low geotechnical complexity and risk, such as defined above, simplified design procedures may be applied.

(10) To establish geotechnical design requirements, three Geotechnical Categories, 1, 2 and 3, may be introduced.

(11) A preliminary classification of a structure according to Geotechnical Category should normally be performed prior to the geotechnical investigations. The category should be checked and changed, if necessary, at each stage of the design and construction process.

(12) The procedures of higher categories may be used to justify more economic designs, or if the designer considers them to be appropriate.

(13) The various design aspects of a project can require treatment in different Geotechnical Categories. It is not required to treat the whole of the project according to the highest of these categories.

(14) Geotechnical Category 1 should only include small and relatively simple structures:

- for which it is possible to ensure that the fundamental requirements will be satisfied on the basis of experience and qualitative geotechnical investigations;
- with negligible risk.

(15) Geotechnical Category 1 procedures should be used only where there is negligible risk in terms of overall stability or ground movements and in ground conditions, which are known from comparable local experience to be sufficiently straightforward. In these cases the procedures may consist of routine methods for foundation design and construction.

(16) Geotechnical Category 1 procedures should be used only if there is no excavation below the water table or if comparable local experience indicates that a proposed excavation below the water table will be straightforward.

(17) Geotechnical Category 2 should include conventional types of structure and foundation with no exceptional risk or difficult AC_1 ground AC_1 or loading conditions

(18) Designs for structures in Geotechnical Category 2 should normally include quantitative geotechnical data and analysis to ensure that the fundamental requirements are satisfied.

(19) Routine procedures for field and laboratory testing and for design and execution may be used for Geotechnical Category 2 designs.

NOTE The following are examples of conventional structures or parts of structures complying with Geotechnical Category 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 anchors and other tie-back systems;
- tunnels in hard, non-fractured rock and not subjected to special water tightness or other requirements.

(20) Geotechnical Category 3 should include structures or parts of structures, which fall outside the limits of Geotechnical Categories 1 and 2.

(21) Geotechnical Category 3 should normally include alternative provisions and rules to those in this standard.

NOTE Geotechnical Category 3 includes the following examples:

- very large or unusual structures;
- structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions;
- structures in highly seismic areas;
- structures in areas of probable site instability or persistent ground movements that require separate investigation or special measures.

2.2 Design situations

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

(2) In geotechnical design, the detailed specifications of design situations should include, as appropriate:

- the actions, their combinations and load cases;
- the general suitability of the ground on which the structure is located with respect to overall stability and ground movements;
- the disposition and classification of the various zones of soil, rock and elements of construction, which are involved in any calculation model;
- dipping bedding planes;
- mine workings, caves or other underground structures;
- in the case of structures resting on or near rock:
 - interbedded hard and soft strata;
 - faults, joints and fissures;
 - possible instability of rock blocks;
 - solution cavities, such as swallow holes or fissures filled with soft material, and continuing solution processes;
- the environment within which the design is set, including the following:
 - effects of scour, erosion and excavation, leading to changes in the geometry of the ground surface;
 - effects of chemical corrosion;

- effects of weathering;
- effects of freezing;
- effects of long duration droughts;
- variations in ground-water levels, including, e.g. the effects of dewatering, possible flooding, failure of drainage systems, water exploitation;
- the presence of gases emerging from the ground;
- other effects of time and environment on the strength and other properties of materials; e.g. the effect of holes created by animal activities;
- earthquakes;
- ground movements caused by subsidence due to mining or other activities;
- the sensitivity of the structure to deformations;
- the effect of the new structure on existing structures, services and the local environment.

2.3 Durability

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

(2) In designing for durability of materials used in the ground, the following should be considered:

a) for concrete:

- aggressive agents in the ground-water or in the ground or fill material, such as acids or sulfate salts;

b) for steel:

- chemical attack where foundation elements are buried in ground that is sufficiently permeable to allow the percolation of ground-water and oxygen;
- corrosion on the faces of sheet pile walls exposed to free water, particularly in the mean water level zone;
- the pitting type of corrosive attack on steel embedded in fissured or porous concrete, particularly for rolled steel where the mill scale, acting as a cathode, promotes electrolytic action with the scale-free surface acting as an anode;

c) for timber:

- fungi and aerobic bacteria in the presence of oxygen;

d) for synthetic fabrics:

- the ageing effects of UV exposure or ozone degradation or the combined effects of temperature and stress, and secondary effects due to chemical degradation.

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

2.4 Geotechnical design by calculation

2.4.1 General

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

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

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

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

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

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

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

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

(7) A calculation model may include simplifications.

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

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

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

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

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

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

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

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

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

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

2.4.2 Actions

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

(2)P The values of geotechnical actions to be used shall be selected, since they are known before a calculation is performed; they may change during that calculation.

NOTE Values of geotechnical actions may change during the course of calculation. In such cases they will be introduced as a first estimate to start the calculation with a preliminary, known value.

(3)P Any interaction between the structure and the ground shall be taken into account when determining the actions to be adopted in the design.

(4) In geotechnical design, the following should be considered for inclusion as actions:

- the weight of soil, rock and water;
- stresses in the ground;
- earth pressures AC1 *text deleted* AC1;
- free water pressures, including wave pressures;
- ground-water pressures;
- seepage forces;
- dead and imposed loads from structures;
- surcharges;
- mooring forces;
- removal of load or excavation of ground;
- traffic loads;

- movements caused by mining or other caving or tunnelling activities;
- swelling and shrinkage caused by vegetation, climate or moisture changes;
- movements due to creeping or sliding or settling ground masses;
- movements due to degradation, dispersion, decomposition, self-compaction and solution;
- movements and accelerations caused by earthquakes, explosions, vibrations and dynamic loads;
- temperature effects, including frost action;
- ice loading;
- imposed pre-stress in ground anchors or struts;
- downdrag.

(5)P Consideration shall be given to the possibility of variable actions occurring both jointly and separately.

(6)P The duration of actions shall be considered with reference to time effects in the material properties of the soil, especially the drainage properties and compressibility of fine-grained soils.

(7)P Actions, which are applied repeatedly, and actions with variable intensity shall be identified for special consideration with regard to, e.g. continuing movements, liquefaction of soils, change of ground stiffness and strength.

(8)P Actions that produce a dynamic response in the structure and the ground shall be identified for special consideration.

(9)P Actions in which ground- and free-water forces predominate shall be identified for special consideration with regard to deformations, fissuring, variable permeability and erosion.

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

2.4.3 Ground properties

(1)P Properties of soil and rock masses, as quantified for design calculations by geotechnical parameters, shall be obtained from test results, either directly or through correlation, theory or empiricism, and from other relevant data.

(2)P Values obtained from test results and other data shall be interpreted appropriately for the limit state considered.

(3)P Account shall be taken of the possible differences between the ground properties and geotechnical parameters obtained from test results and those governing the behaviour of the geotechnical structure.

(4) The differences to which 2.4.3(3)P refers can be due to the following factors:

- many geotechnical parameters are not true constants but depend on stress level and mode of deformation;

- soil and rock structure (e.g. fissures, laminations, or large particles) that may play a different role in the test and in the geotechnical structure;
- time effects;
- the softening effect of percolating water on soil or rock strength;
- the softening effect of dynamic actions;
- the brittleness or ductility of the soil and rock tested;
- the method of installation of the geotechnical structure;
- the influence of workmanship on artificially placed or improved ground;
- the effect of construction activities on the properties of the ground.

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

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

(6)P Calibration factors shall be applied where necessary to convert laboratory or field test results according to EN 1997-2 into values that represent the behaviour of the soil and rock in the ground, for the actual limit state, or to take account of correlations used to obtain derived values from the test results.

2.4.4 Geometrical data

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

2.4.5 Characteristic values

2.4.5.1 Characteristic and representative values of actions

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

2.4.5.2 Characteristic values of geotechnical parameters

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

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

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

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

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

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

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

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

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

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

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

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

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

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

2.4.5.3 Characteristic values of geometrical data

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

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

2.4.6 Design values

2.4.6.1 Design values of actions

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

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

$$F_d = \gamma_f \cdot F_{rep} \quad (2.1a)$$

with

$$F_{rep} = \psi \cdot F_k \quad (2.1b)$$

(3)P Appropriate values of ψ shall be taken from EN 1990:2002.

(4)P The partial factor γ_f for persistent and transient situations defined in Annex A shall be used in equation (2.1a).

NOTE 1 The values of the partial factors may be set by the National annex.

NOTE 2 The recommended values in Annex A indicate the appropriate level of safety for conventional designs.

(5) If design values of geotechnical actions 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.

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

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

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

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

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

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

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

2.4.6.2 Design values of geotechnical parameters

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

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

or shall be assessed directly.

(2)P The partial factor γ_M for persistent and transient situations defined in Annex A shall be used in equation (2.2).

NOTE 1 The values of the partial factors may be set by the National annex.

NOTE 2 The recommended values in Annex A indicate the minimum level of safety for conventional designs.

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

2.4.6.3 Design values of geometrical data

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

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

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

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

2.4.6.4 Design values of structural properties

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

2.4.7 Ultimate Limit States

2.4.7.1 General

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

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

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

(2)P The partial factors in persistent and transient situations defined in Annex A shall be used.

NOTE The values of the partial factors may be set by the National annex. The tables in Annex A give the recommended values.

(3) All values of partial factors for actions or the effects of actions in accidental situations should normally be taken equal to 1,0. All values of partial factors for resistances should then be selected according to the particular circumstances of the accidental situation.

NOTE The values of the partial factors may be set by the National annex.

(4) More severe values than those recommended in Annex A should be used in cases of abnormal risk or unusual or exceptionally difficult ground or loading conditions.

AC1 NOTE The values of partial factors may be set by the National Annex. **AC1**

(5) Less severe values than those recommended in Annex A may be used for temporary structures or transient design situations, where the likely consequences justify it.

AC1 NOTE The values of partial factors may be set by the National Annex. **AC1**

(6) When calculating the design value of the resistance, (R_d), or the design value of the effect of actions, (E_d), model factors, ($\gamma_{F;d}$) or ($\gamma_{S;d}$) respectively, may be introduced to ensure that the results of the design calculation model are either accurate or err on the safe side.

AC1 NOTE The values of model factors may be set by the National Annex. **AC1**

2.4.7.2 Verification of static equilibrium

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

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

with

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

and

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

(2)P The partial factors for persistent and transient situations defined in A.2(1)P and A.2(2)P shall be used in equation (2.4).

NOTE 1 Static equilibrium EQU is mainly relevant in structural design. In geotechnical design, EQU verification will be limited to rare cases, such as a rigid foundation bearing on rock, and is, in principle, distinct from overall stability or buoyancy problems. If any shearing resistance T_d is included, it should be of minor importance.

NOTE 2 The values of the partial factors may be set by the National annex. Tables A.1 and A.2 give the recommended values.

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

2.4.7.3.1 General

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

$$E_d \leq R_d \quad (2.5)$$

2.4.7.3.2 Design effects of actions

(1) Partial factors on actions may be applied either to the actions themselves (F_{rep}) or to their effects (E):

$$E_d = E\{\gamma_F F_{\text{rep}}; X_k / \gamma_M; a_d\} \quad (2.6a)$$

or

$$E_d = \gamma_E E\{F_{\text{rep}}; X_k / \gamma_M; a_d\}. \quad (2.6b)$$

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

(3)P The partial factors defined in A.3.1(1)P and A.3.2(1)P shall be used in equations (2.6a) and (2.6b).

NOTE The values of the partial factors may be set by the National annex. Tables A.3 and A.4 give the recommended values.

2.4.7.3.3 Design resistances

(1) Partial factors may be applied either to ground properties (X) or resistances (R) or to both, as follows:

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

or

$$R_d = R\{\gamma_F F_{rep}; X_k; a_d\}/\gamma_R \quad (2.7b)$$

or

$$R_d = R\{\gamma_F F_{rep}; X_k/\gamma_M; a_d\}/\gamma_R \quad (2.7c)$$

NOTE In design procedures where the effects of actions are factored, the partial factor for actions $\gamma_F = 1,0$. (see also B.3(6))

(2)P The partial factors, defined in A.3.3.1(1)P, A.3.3.2(1)P, A.3.3.4(1)P, A.3.3.5(1)P and A.3.3.6(1)P shall be used in equations (2.7a, b, and c).

NOTE The values of the partial factors may be set by the National annex. Tables A.5, A.6, A.7, A.8, A.12, A.13 and A.14 give the recommended values.

2.4.7.3.4 Design Approaches

2.4.7.3.4.1 General

(1)P The manner in which equations (2.6) and (2.7) are applied shall be determined using one of three Design Approaches.

NOTE 1 The way to use equations (2.6) and (2.7) and the particular Design Approach to be used may be given in the National annex.

NOTE 2 Further clarification of the Design Approaches is provided in Annex B.

NOTE 3 The partial factors in Annex A to be used in equations (2.6) and (2.7) are grouped in sets denoted by A (for actions or effects of actions), M (for soil parameters) and R (for resistances). They are selected according to the Design Approach used.

2.4.7.3.4.2 Design Approach 1

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

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

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

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

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

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

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

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

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

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

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

2.4.7.3.4.3 Design Approach 2

(1)P It shall be verified that a limit state of rupture or excessive deformation will not occur with the following combination of sets of partial factors:

Combination: $A1$ "+" $M1$ "+" $R2$

NOTE 1 In this approach, partial factors are applied to actions or to the effects of actions and to ground resistances.

NOTE 2 If this approach is used for slope and overall stability analyses the resulting effect of the actions on the failure surface is multiplied by γ_E and the shear resistance along the failure surface is divided by $\gamma_{R;e}$.

2.4.7.3.4.4 Design Approach 3

(1)P It shall be verified that a limit state of rupture or excessive deformation will not occur with the following combination of sets of partial factors:

Combination: ($A1^*$ or $A2^\dagger$) "+" $M2$ "+" $R3$

*on structural actions

† on geotechnical actions

NOTE 1 In this approach, partial factors are applied to actions or to the effects of actions from the structure and to ground strength parameters.

NOTE 2 For slope and overall stability analyses, actions on the soil (e.g. structural actions, traffic load) are treated as geotechnical actions by using the set of load factors $A2$.

2.4.7.4 Verification procedure and partial factors for uplift

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

$$V_{dst;d} \leq G_{stb;d} + R_d \quad (2.8)$$

where

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

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

(3)P The partial factors for $G_{\text{dst};d}$, $Q_{\text{dst};d}$, $G_{\text{stb};d}$ and R_d for persistent and transient situations defined in A.4(1)P and A.4(2)P shall be used in equation (2.8).

NOTE The values of the partial factors may be set by the National annex. Tables A.15 and A.16 give the recommended values.

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

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

$$u_{\text{dst};d} \leq \sigma_{\text{stb};d} \quad (2.9a)$$

$$S_{\text{dst};d} \leq G'_{\text{stb};d} \quad (2.9b)$$

(2)P The partial factors for $u_{\text{dst};d}$, $\sigma_{\text{stb};d}$, $S_{\text{dst};d}$ and $G'_{\text{stb};d}$ for persistent and transient situations defined in A.5(1)P shall be used in equations 2.9a and 2.9b.

NOTE The values of the partial factors may be set by the National annex. Table A.17 gives the recommended values.

2.4.8 Serviceability limit states

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

$$E_d \leq C_d, \quad (2.10)$$

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

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

NOTE The values of the partial factors may be set by the National annex.

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

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

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

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

2.4.9 Limiting values for movements of foundations

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

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

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

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

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

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

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

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

2.5 Design by prescriptive measures

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

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

(2) Design by prescriptive measures may be used where comparable experience, as defined in 1.5.2.2, makes design calculations unnecessary. It may also be used to ensure durability

against frost action and chemical or biological attack, for which direct calculations are not generally appropriate.

2.6 Load tests and tests on experimental models

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

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

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

2.7 Observational method

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

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

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

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

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

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

2.8 Geotechnical Design Report

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

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

(3) The Geotechnical Design Report should normally include the following items, with cross-reference to the Ground Investigation Report (see 3.4) and to other documents, which contain more detail:

- a description of the site and surroundings;
- a description of the ground conditions;
- a description of the proposed construction, including actions;
- design values of soil and rock properties, including justification, as appropriate;
- statements on the codes and standards applied;
- statements on the suitability of the site with respect to the proposed construction and the level of acceptable risks;
- geotechnical design calculations and drawings;
- foundation design recommendations;
- a note of items to be checked during construction or requiring maintenance or monitoring.

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

(5) In relation to supervision and monitoring the Geotechnical Design Report should state:

- the purpose of each set of observations or measurements;
- the parts of the structure, which are to be monitored and the locations at which observations are to be made;
- the frequency with which readings are to be taken;
- the ways in which the results are to be evaluated;
- the range of values within which the results are to be expected;
- the period of time for which monitoring is to continue after construction is complete;
- the parties responsible for making measurements and observations, for interpreting the results obtained and for maintaining the instruments.

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

Section 3 Geotechnical data

3.1 General

(1)P Careful collection, recording and interpretation of geotechnical information shall always be made. This information shall include geology, geomorphology, seismicity, hydrology and history of the site. Indications of the variability of the ground shall be taken into account.

(2)P Geotechnical investigations shall be planned taking into account the construction and performance requirements of the proposed structure. The scope of geotechnical investigations shall be continuously reviewed as new information is obtained during execution of the work.

(3)P Routine field investigations and laboratory testing shall be carried out and reported generally in accordance with internationally recognised standards and recommendations. Deviations from these standards and additional test requirements shall be reported.

(4) Requirements for laboratory and field-testing should be taken from EN 1997-2.

3.2 Geotechnical investigations

3.2.1 General

(1)P Geotechnical investigations shall provide sufficient data concerning the ground and the ground-water conditions at and around the construction site for a proper description of the essential ground properties and a reliable assessment of the characteristic values of the ground parameters to be used in design calculations.

(2)P The composition and amount of the geotechnical investigations shall be adjusted to the particular investigation phase and the geotechnical category (see EN 1997-2, Section 2).

(3) For very large or unusual structures, structures involving abnormal risks or unusual or exceptionally difficult ground or loading conditions, and structures in highly seismic areas, it is possible that the extent of investigations specified in EN 1997 will not be sufficient to meet the design requirements.

(4) If the character and extent of the investigations are related to the Geotechnical Category of the structure, ground conditions that may influence the choice of Geotechnical Category should be determined as early as possible in the investigation.

(5) The investigations should include visual inspections of the site to enable the design assumptions to be verified during construction.

3.2.2 Preliminary investigations

(1)P Preliminary investigations shall be carried out:

- to assess the general suitability of the site;
- to compare alternative sites, if relevant;
- to estimate the changes that may be caused by the proposed works;
- to plan the design and control investigations, including identification of the extent of ground, which may have significant influence on the behaviour of the structure;
- to identify borrow areas, if relevant.

3.2.3 Design investigations

(1)P Design investigations shall be carried out:

- to provide the information required for an adequate design of the temporary and permanent works;
- to provide the information required to plan the method of construction;
- to identify any difficulties that may arise during construction.

(2)P The design investigation shall identify in a reliable way the disposition and properties of all ground relevant to or affected by the proposed construction.

(3)P The parameters, which affect the ability of the structure to satisfy its performance criteria shall be established before the start of the final design.

(4) In order to ensure that the design investigation covers all relevant ground formations, particular attention should be paid to the following geological features:

- ground profile;
- natural or man-made cavities;
- degradation of rocks, soils, or fill materials;
- hydrogeological effects;
- faults, joints and other discontinuities;
- creeping soil and rock masses;
- expansible and collapsible soils and rocks;
- presence of waste or man-made materials.

(5)P The history of the site and its surroundings shall be taken into account.

(6)P The investigation shall be carried out at least through the formations, which are assessed as being relevant to the project.

(7)P The existing ground-water levels shall be established during the investigation. Any free water levels observed during the investigation shall be recorded (see EN 1997-2).

(8) The extreme water levels of any water source, which might influence the ground-water pressures should be established.

(9)P The location and capacities of any dewatering or water abstraction wells in the vicinity of the site shall be established.

3.3 Evaluation of geotechnical parameters

3.3.1 General

(1) In the following requirements concerning the evaluation of geotechnical parameters, only the most commonly used laboratory and field tests have been referred to. Other tests may be used provided their suitability has been demonstrated through comparable experience.

3.3.2 Characterisation of soil and rock type

(1)P The character and basic constituents of the soil or rock shall be identified before the results of other tests are interpreted.

(2)P The material shall be examined, identified and described in accordance with a recognised nomenclature. A geological evaluation shall be made.

(3) Soils should be classified and soil layers described according to an acknowledged geotechnical soil classification and description system.

(4) Rock should be classified in terms of the quality of the solid (stone) material and jointing. Stone quality should be described in terms of weathering, particle organisation, dominant grain size of minerals, and hardness and toughness of the main mineral. Jointing should be characterised in terms of joint type, width, spacing and fill quality.

(5) In addition to visual inspection, a number of tests for classification, identification and quantification of soils and rocks may be used (see EN 1997-2), such as

for soils:

- grain size distribution;
- weight density;
- porosity;
- water content;
- grain shape;
- grain surface roughness;
- density index;
- Atterberg limits;
- swelling;
- carbonate content;
- organic matter content.

for rocks:

- mineralogy;
- petrography;
- water content;
- weight density;
- porosity;
- sound velocity;
- quick water absorption;

- swelling;
- slake-durability index;
- uniaxial compressive strength.

3.3.3 Weight density

(1)P The weight density shall be determined with sufficient accuracy to establish design or characteristic values of the actions that derive from it.

(2) The weight density should be determined on specimens of soil and rock taken from undisturbed samples (see EN 1997-2). Alternatively, it may be derived from well established or documented correlations based on, for example, penetration tests.

3.3.4 Density index

(1)P The density index shall express the degree of compaction of a non-cohesive soil with respect to the loosest and densest condition as defined by standard laboratory procedures.

3.3.5 Degree of compaction

(1)P The degree of compaction of natural ground or fill shall be expressed as the ratio between dry weight density and maximum dry weight density obtained from a standard compaction test.

3.3.6 Shear strength

(1)P In assessing the shear strength of soil, the influence of the following features shall be considered:

- the stress level imposed on the soil;
- anisotropy of strength, especially in clays of low plasticity;
- fissures, especially in stiff clays;
- strain rate effects;
- very large strains where these may occur in a design situation;
- pre-formed slip surfaces;
- time effects;
- sensitivity of cohesive soil;
- degree of saturation.

(2) When the shear strength assessment is based on test results, the level of confidence in the theory used to derive shear strength values should be taken into account, as well as the possible disturbance during sampling and heterogeneity of samples.

(3) As to time effects, it should be considered that the period for which a soil will be effectively undrained depends on its permeability, the availability of free water and the geometry of the situation.

(4)P The values of effective shear strength parameters c' and $\tan \phi'$ shall be assumed to be constant only within the range of stresses for which they have been evaluated.

3.3.7 Soil stiffness

(1)P In assessing the soil stiffness, the following features shall be considered:

- drainage conditions;
- level of mean effective stress;
- natural or artificial pre-consolidation;
- level of imposed shear strain or induced shear stress, this latter often normalised with respect to the shear strength at failure.

(2) Reliable measurements of the stiffness of the ground are often very difficult to obtain from field or laboratory tests. In particular, owing to sample disturbance and other effects, measurements obtained from laboratory specimens often underestimate the in-situ stiffness of the soil. Observations of the behaviour of previous constructions should therefore be analysed wherever available.

3.3.8 Quality and properties of rocks and rock masses

3.3.8.1 General assessment

(1)P In assessing the quality and properties of rocks and rock masses, a distinction shall be drawn between the behaviour of rock material as measured on undisturbed core samples and the behaviour of much larger rock masses, which include structural discontinuities such as bedding planes, joints, shear zones and solution cavities. Consideration shall be given to the following characteristics of the joints:

- spacing;
- orientation;
- aperture;
- persistence (continuity);
- tightness;
- roughness, including the effects of previous movements on the joints;
- filling.

(2)P In addition, when assessing the properties of rocks and rock masses, the following items shall be considered, if relevant:

- in situ stresses;
- water pressure;
- pronounced variations in properties between different layers.

(3) Estimates of rock mass properties, such as:

- strength and stiffness,

- jointing, especially in fractured zones,
- water permeability of the joint system,
- deformation properties of weathered rock,

may be obtained by using the concept of rock mass classification described in EN 1997-2.

(4)P The sensitivity of rocks to e.g. climate or stress changes, shall be assessed. Consideration shall also be given to the influence of chemical degradation on the performance of rock foundations.

(5) In assessing the quality of rocks and rock masses, consideration should be given to the following features:

- some porous soft rocks degrade rapidly to soils of low strength, especially if exposed to weathering;
- some rocks exhibit high solution rates due to ground-water causing channels, caverns and sinkholes, which may develop to the ground surface;
- when unloaded and exposed to the air, certain rocks experience pronounced swelling due to the absorption of water by clay minerals.

3.3.8.2 Uniaxial compressive strength and deformability of rock materials

(1)P In assessing the uniaxial compressive strength and deformability of rock materials the influence of the following features shall be considered:

- the orientation of the axis of loading with respect, for example, to specimen anisotropy, bedding planes, foliation;
- method of sampling, storage history and environment;
- number of specimens tested;
- the geometry of the tested specimens;
- water content and degree of saturation at time of test;
- test duration and stress rate;
- method for determination of the Young's modulus and the axial stress level or levels at which it is determined.

3.3.8.3 Shear strength of joints

(1)P In assessing the shear strength of joints of rock materials, the influence of the following features shall be considered:

- orientation of the joint within the rock test in relation to the assumed direction of actions;
- orientation of the shear test;
- number of specimens tested;
- dimensions of the sheared area;

- pore-water pressure conditions;
- possibility of progressive failure governing the behaviour of the rock in the ground.

(2) Planes of weakness in rock normally coincide with joints or planes of bedding, schistosity or cleavage, or with the interface between soil and rock or concrete and rock. Measurements of the shear strength on these planes should normally be used for the limit equilibrium analysis of rock masses.

3.3.9 Permeability and consolidation parameters of soil and rock

3.3.9.1 Permeability and consolidation parameters of soil

(1)P In assessing permeability and consolidation parameters, the following factors shall be considered:

- the effects of heterogeneity;
- the effects of anisotropy;
- the effects of fissures or faults;
- the effects of stress changes under the proposed loading.

(2) Permeability measurements made on small laboratory samples may not be representative of the in-situ conditions. Whenever possible, in-situ tests, which measure average properties of a large ground volume should therefore be preferred. However, consideration should be given to possible changes in the permeability with increased effective stress above the in-situ value.

(3) Sometimes permeability should be evaluated on the basis of knowledge of the grain size distribution.

3.3.9.2 Permeability parameters of rock

(1)P Since the permeability of rock masses depends mainly on the degree of jointing and the existence of other discontinuities such as fractures and fissures, it shall be measured by appropriate in situ tests or evaluated from local experience.

(2) In situ permeability may be determined by a system of pumping tests combined with flow logging, with due consideration of the spatial, hydrogeological flow conditions around the structure and the mapping of the patterns of joints and other discontinuities.

(3) Laboratory permeability tests should only be used to study the effect of discontinuities, for example, in terms of variable aperture.

3.3.10 Geotechnical parameters from field tests

3.3.10.1 Cone penetration test

(1)P In assessing values of the cone resistance, the sleeve friction and, possibly, the pore-water pressure during penetration, the following aspects shall be taken into account:

- the detailed design of the cone and friction sleeve. This may affect the results significantly and allowance must therefore be made for the type of cone used;

- the results can only be interpreted with confidence when the succession of soil layers is established; in many situations borings will therefore be needed in conjunction with the penetration tests;
- the effects of ground-water and overburden;
- in heterogeneous soils in which widely fluctuating results are recorded, the penetration values shall be selected for the zone of soil relevant to the construction;
- established correlations with other test results, such as density measurements and other forms of penetration testing.

3.3.10.2 Standard penetration and dynamic probing test

(1)P In assessing blow counts, the following features shall be considered:

- type of test;
- detailed description of the test procedure;
- ground-water conditions;
- the influence of the overburden pressure;
- the nature of the ground, particularly if cobbles or coarse gravel are encountered.

3.3.10.3 Vane test

(1)P The following shall be considered when assessing the test results:

- details of the test procedure;
- whether standardised vane equipment has been used;
- whether measurements have been made at several depths to provide a profile of strength in the succession of soil layers;
- skin friction along the rod.

(2) Vane tests may be used for the assessment of undrained shear strength, c_u , of cohesive soil.

NOTE The vane test is a simple and cheap way of checking the trafficability of soft ground for heavy equipment and vehicles.

(3) To obtain derived values of c_u , the measured values should be corrected by a factor based on local experience and depending for example, on the liquid limit, plasticity index and effective vertical stress.

3.3.10.4 Weight sounding test

(1)P In assessing weight sounding test results, the following features shall be taken into account:

- detailed description of the test procedure;
- ground-water conditions;