

TC 250/SC7/PT1/Version “h”

prEN 1997-1
EUROCODE 7 GEOTECHNICAL DESIGN
PART 1 GENERAL RULES

Final draft

October 2001

Contents

Foreword	7
Section 1 General	
1.1 Scope	10
1.1.1 Scope of EN 1997	10
1.1.2 Scope of EN 1997-1	10
1.1.3 Further Parts of EN 1997	11
1.2 References	11
1.3 Distinction between Principles and Application Rules	11
1.4 Assumptions	12
1.5 Definitions	12
1.5.1 Terms common to all Eurocodes	12
1.5.2 Special terms used in EN 1997-1	12
1.6 S.I. units	13
1.7 Symbols common to all Eurocodes	13
1.8 Symbols and abbreviated terms used in EN 1997-1	13
Section 2 Basis of geotechnical design	
2.1 Design requirements	18
2.2 Design situations	20
2.3 Durability	21
2.4 Geotechnical design by calculation	21
2.4.1 General	21
2.4.2 Actions	23
2.4.3 Ground properties	24
2.4.4 Geometrical data	25
2.4.5 Characteristic values	25
2.4.6 Design values	27
2.4.7 Ultimate limit states	29
2.4.8 Serviceability limit states	33
2.4.9 Limiting values for movements of structures	34
2.4.10 Verification procedures	34
2.5 Design by prescriptive measures	34
2.6 Load tests and tests on experimental models	35
2.7 Observational method	35
2.8 Geotechnical Design Report	36
Section 3 Geotechnical data	
3.1 General	
3.2 Geotechnical investigations	37
3.2.1 General	37
3.2.2 Preliminary investigations	38
3.2.3 Design investigations	38
3.3 Evaluation of geotechnical parameters	39
3.3.1 General	39
3.3.2 Characterization of soil and rock type	39
3.3.3 Weight density	40
3.3.4 Density index	40
3.3.5 Degree of compaction	40
3.3.6 Shear strength	40
3.3.7 Soil stiffness	40
3.3.8 Quality and properties of rocks and rock masses	41
3.3.9 Permeability and consolidation parameters of soil and rock	42
3.3.10 Geotechnical parameters from field tests	43
3.4 Ground investigation report	45
3.4.1 Requirements	45
3.4.2 Presentation of geotechnical information	45
3.4.3 Evaluation of geotechnical information	46

Section 4 Supervision of construction, monitoring and maintenance	
4.1 General	48
4.2 Supervision	48
4.2.1 Plan of supervision	48
4.2.2 Inspection and control	49
4.2.3 Assessment of the design	49
4.3 Checking ground conditions	50
4.3.1 Soil and rock	50
4.3.2 Groundwater	50
4.4 Checking construction	51
4.5 Monitoring	52
4.6 Maintenance	53
Section 5 Fill, dewatering, ground improvement and reinforcement	
5.1 General	54
5.2 Fundamental requirements	54
5.3 Fill construction	54
5.3.1 Principles	54
5.3.2 Selection of fill material	54
5.3.3 Selection of procedures for fill placement and compaction	56
5.3.4 Checking the fill	56
5.4 Dewatering	57
5.5 Ground improvement and reinforcement	58
Section 6 Spread foundations	
6.1 General	60
6.2 Limit states	60
6.3 Actions and design situations	60
6.4 Design and construction considerations	60
6.5 Ultimate limit state design	61
6.5.1 Overall stability	61
6.5.2 Bearing resistance	62
6.5.3 Sliding resistance	63
6.5.4 Loads with large eccentricities	64
6.5.5 Structural failure due to foundation movement	64
6.6 Serviceability limit state design	65
6.6.1 General	65
6.6.2 Settlement	65
6.6.3 Heave	67
6.6.4 Vibration analysis	67
6.7 Foundations on rock: additional design considerations	67
6.8 Structural design of spread foundations	67
6.9 Preparation of the subsoil	68
Section 7 Pile foundations	
7.1 General	69
7.2 Limit states	69
7.3 Actions and design situations	69
7.3.1 General	69
7.3.2 Actions due to ground displacement	70
7.4 Design methods and design considerations	71
7.4.1 Design methods	71
7.4.2 Design considerations	72
7.5 Pile load tests	73
7.5.1 General	73
7.5.2 Static load tests	73
7.5.3 Dynamic load tests	75
7.5.4 Load test report	75
7.6 Axially loaded piles	75
7.6.1 General	75

7.6.2 Compressive ground resistance	76
7.6.3 Ground tensile resistance	82
7.6.4 Vertical displacements of pile foundations (Serviceability of supported structure)	86
7.7 Transversely loaded piles	87
7.7.1 General	87
7.7.2 Transverse load resistance from pile load tests	87
7.7.3 Transverse load resistance from ground test results and pile strength parameters	87
7.7.4 Transverse displacement	88
7.8 Structural design of piles	88
7.9 Supervision of construction	88
Section 8 Anchorages	
8.1 General	91
8.1.1 Scope	91
8.1.2 Definitions	91
8.2 Limit states	92
8.3 Actions and design situations	92
8.4 Design and construction considerations	92
8.5 Ultimate limit state design	94
8.5.1 Design of the anchor	94
8.5.2 Design values of pull-out resistance determined by tests	94
8.5.3 Design values of pull-out resistance determined by calculations	94
8.5.4 Design value of the structural resistance of the anchorage	95
8.5.5 Design value of the anchor load	95
8.6 Serviceability limit state design	95
8.7 Suitability tests	95
8.8 Acceptance tests	96
8.9 Supervision and monitoring	96
Section 9 Retaining structures	
9.1 General	97
9.1.1 Scope	97
9.1.2 Definitions	97
9.2 Limit states	97
9.3 Actions, geometrical data and design situations	98
9.3.1 Actions	98
9.3.2 Geometrical data	100
9.3.3 Design situations	101
9.4 Design and construction considerations	101
9.4.1 General	101
9.4.2 Drainage systems	102
9.5 Determination of earth pressure	102
9.5.1 General	102
9.5.2 At rest values of earth pressure	104
9.5.3 Limit values of earth pressure	104
9.5.4 Intermediate values of earth pressure	105
9.5.5 Compaction effects	105
9.6 Water pressures	105
9.7 Ultimate limit state design	107
9.7.1 General	107
9.7.2 Overall stability	107
9.7.3 Foundation failure of gravity walls	107
9.7.4 Rotational failure of embedded walls	108
9.7.5 Vertical failure of embedded walls	109
9.7.6 Structural design of retaining structures	110
9.7.7 Failure by pull-out of anchors	110
9.8 Serviceability limit state design	111
9.8.1 General	111
9.8.2 Displacements	111

Section 10 Hydraulic failure	
10.1 General	113
10.2 Failure by uplift	114
10.3 Failure by heave	116
10.4 Internal erosion	116
10.5 Failure by piping	117
Section 11 Overall stability	
11.1 General	118
11.2 Limit states	118
11.3 Actions and design situations	118
11.4 Design and construction considerations	119
11.5 Ultimate limit state design	120
11.5.1 Stability analysis for slopes	120
11.5.2 Slopes and cuts in rock masses	121
11.5.3 Stability of excavations	122
 11.5.4 Stability of structures on reinforced or improved ground	
11.6 Serviceability limit state design	123
11.7 Monitoring	123
Section 12 Embankments	
12.1 General	124
12.2 Limit states	124
12.3 Actions and design situations	124
12.4 Design and construction considerations	125
12.5 Ultimate limit state design	126
12.6 Serviceability limit state design	127
12.7 Supervision and monitoring	127
Annexes	
Annex A Partial factors for ultimate limit states	
A.0 Partial factors and their recommended values	129
A.1 Partial factors for equilibrium limit state (EQU) verification	129
A.2 Partial factors for <i>structural</i> (STR) and <i>geotechnical</i> (GEO) limit states verification	130
A.2.1 Partial factors on actions (γ_F) of the effects of actions (γ_E)	130
A.2.2 Partial factors for soil parameters (γ_M)	130
A.2.3 Partial resistance factors (γ_R)	130
A.3 Partial factors for uplift limit state (UPL) verifications	133
A.3.1 Partial factors on actions (γ_F)	133
A.3.2 Partial factors for soil parameters (γ_M)	133
A.4 Partial factors for hydraulic heave limit state (HYD) verifications	134
Annex B Background information on partial factors for Design Approaches 1, 2 and 3	
B.1 General	135
B.2 Factors on actions and the effects of actions	135
B.3 Factors on material strengths and resistances	137
Annex C Sample procedures to determine limit values of earth pressures on vertical walls	
C.1 Limit values of earth pressure	139
C.2 Movements to mobilize limit earth pressures	142
Annex D A sample analytical method for bearing resistance calculation	
D.1 Symbols used in Annex D	144
D.2 General	144
D.3 Undrained conditions	145
D.4 Drained conditions	145
Annex E A sample semi-empirical method for bearing resistance estimation	147

Annex F Sample methods for settlement evaluation	
F.1 Stress-strain method	148
F.2 Adjusted elasticity method	148
F.3 Settlements without drainage	149
F.4 Settlements caused by consolidation	149
F.5 Time-settlement behaviour	149
Annex G A sample method for deriving presumed bearing resistance for spread foundations on rock	150
Annex H Limiting foundation movements and structural deformation	152
Annex J Checklist for construction supervision and performance monitoring	
J.1 General	
J.2 Construction supervision	154
J.2.1 General items to be checked	154
J.2.2 Water flow and pore pressure	154
J.3 Performance monitoring	155

Foreword

This European Standard has been prepared by Technical Committee CEN/TC250 « Structural Eurocodes », the secretariat of which is held by BSI.

CEN/TC250 is responsible for all Structural Eurocodes.

This document is currently submitted to the formal vote (only in formal vote stage)

This European Standard supersedes ENV 1997-1:1994.

The Annexes B to J are informative

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).

Eurocode programme

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

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).

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)

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, etc.), e.g. snow map,
- the procedure to be used where alternative procedures are given in the Eurocode,
- decisions on the application of informative annexes,

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.

- references to non-contradictory complementary information to assist the user to apply the Eurocode.

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

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.

Additional information specific to Eurocode 7

The scope of Eurocode 7 is defined in 1.1.1 of this standard and the scope of Eurocode 7 Part 1 General rules (EN 1997-1) in 1.1.2. Eurocode 7 Part 1 is supplemented by additional Parts which provide the requirements and rules for the performance and evaluation of field and laboratory testing.

In using EN 1997-1 in practice, particular regard should be paid to the underlying assumptions and conditions given in 1.4.

The 12 sections of EN 1997-1 are complemented by 9 annexes.

National annex for EN 1997-1

This standard gives alternative procedures *and recommended values and recommendations for classes* 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), 2.4.7.1(3), 2.4.7.3.4(1)P, 2.4.8(2), 2.4.9(3)P, 7.6.2.3(10), 7.6.3.3(8), 8.5.2(3), 8.5.5(2) and the following annexes
- A.1.1, A.1.2
- A.2.1, A.2.2, A.2.3.1, A.2.3.2.1, A.2.3.2.2, A.2.3.2.3, A.2.3.3.1, A.2.3.3.2, A.2.3.3.3, A.2.3.4, A.2.3.5, A.2.3.6
- A.3.1, A.3.2
- A.4

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.

Section 1 General

1.1 Scope

1.1.1 Scope of EN 1997

(1) EN 1997-1 shall be used in conjunction with EN 1990 - Basis of structural design that 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 1997-1 shall 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 1997-1 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 - Actions on Structures applicable to 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 shall be used to treat matters of execution and workmanship. They are denoted in the relevant sections.

(6) In EN 1997-1 execution is covered to the extent that is necessary to comply with the assumptions of the design rules.

(7) Eurocode 7 does not cover the special requirements of seismic design. EN 1998 - Design of structures for earth quake resistance, 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 shall 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 - Geotechnical design:

Section 1: General

Section 2: Basis of Geotechnical Design

Section 3: Geotechnical Data

Section 4: Supervision of Construction, Monitoring and Maintenance

Section 5: Fill, Dewatering, Ground Improvement and Reinforcement

Section 6: Spread Foundations

Section 7: Pile Foundations

Section 8: Anchorages

Section 9: Retaining Structures

Section 10: Hydraulic failure

Section 11: Overall stability

Section 12: Embankments

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

- in A: recommended partial safety factors; the values of the partial factors may be set in the National annex;
- in B to J: supplementary information such as e.g. internationally applied calculation methods.

1.1.3 Further Parts of EN 1997

(1) EN 1997-1 is supplemented by *ENV 1997-2 and ENV 1997-3* that treat the requirements and rules for the performance and evaluation of field and laboratory testing.

1.2 References

(1) This European Standard incorporates, by dated or undated reference, provisions from other standards. 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).

ISO 3898:1997	Basis of design of structures – Notations. General symbols
EN 1990:2002	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
ENV 1997-2	Eurocode 7 Geotechnical design Part 2 Design assisted by laboratory testing
ENV 1997-3	Eurocode 7 Geotechnical design Part 3 Design assisted by field testing
EN 1998	Eurocode 8 Design of structures for earth quake resistance
EN 1999	Eurocode 9 Design of aluminum and aluminum alloy structures
EN 1536:1999	Execution of special geotechnical work Bored piles
EN 1537:1999	Execution of special geotechnical work Ground anchors
EN 12063:1999	Execution of special geotechnical work Sheet pile walls
EN 12699:2000	Execution of special geotechnical work Displacement piles
	To be completed and checked.

1.3 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 recognized 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.

1.4 Assumptions

(1) The provisions of this standard are based on the assumptions given below. Users of the standard shall strive to ensure that:

- 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.

(2) 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.

1.5 Definitions

1.5.1 Definitions common to all Eurocodes

(1) The definitions common to all Eurocodes are given in EN 1990 - Basis of structural design. The definitions that are specific for EN 1997-1 are given in 1.5.2

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 groundwater.
(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. [**shearing**] resistance of the ground, bending resistance, buckling resistance, tension resistance

NOTE: Definition derived from EN 1990:2002

1.6 S.I. units

(1) S.I. units shall be used in accordance with ISO 1000.

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

– forces	kN
– moments	kNm
– mass density	kg/m ³
– weight density	kN/m ³
– stresses, pressures, strengths and stiffness	kPa
– coefficient of permeability:	m/s
– coefficient of consolidation:	m ² /s

1.7 Symbols common to all Eurocodes

(1) The symbols used in common for all Eurocodes are defined in EN 1990:2002

1.8 Symbols and abbreviated terms used in EN 1997-1

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

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

Latin letters

A'	effective base area
A_c	area of the total base area being under compression
$A'_{s,i}$	pile shaft surface area in layer i (7)
a	adhesion (see Annex C)
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. (D) (F)
b'	effective width of a foundation (fig. D.1); (D)
C_d	limiting design value of the effect of an action (2)
c	<i>ground cohesion</i>
c'	cohesion intercept in terms of effective stress (1.8)
c_u	undrained shear strength (1.8)
$c_{u,d}$	design value of the undrained shear strength (6)
d	embedment depth; (D)
E_d	design value of the effect of actions (2)
$E_{stb,d}$	design value of the effect of stabilising actions (2)
$E_{dst,d}$	design value of the effect of destabilising actions (2)
$F_{c,d}$	design axial compression load on a pile or a group of piles (7)
F_d	design value of an action (2)
F_k	characteristic value of an action (2)
F_{rep}	representative value of an action (2)
$F_{t,d}$	design axial tensile load on a tensile pile or a group of tensile piles (7)
$F_{tr,d}$	design value of the transverse load on a pile or a pile foundation (7)
$G_{dst,d}$	<i>design value of the destabilising permanent actions for uplift verification (2)</i>
$G_{stb,d}$	design value of the stabilizing permanent vertical actions for the uplift verification (2)
H	horizontal load, <i>or component of the total action acting parallel to the foundation base</i> ; (D)
H_d	design value of the component of total action acting parallel to the foundation base (6)
h	<i>height of a wall</i>
h	water level for hydraulic heave (see fig.10.2)
h'	height of the soil prism for verifying hydraulic heave
$h_{w,k}$	characteristic value of the hydrostatic water head at the bottom of a soil prism (10)
J_e, J_{qT}	
J_γ	inclination factors of the bearing capacity analysis (see Annex D)
$K_{a,h}$	coefficient of horizontal active earth pressure (C)
$K_{p,h}$	coefficient of horizontal passive earth pressure (C)
K_0	coefficient of earth pressure at rest (9)
$K_{0,\beta}$	coefficient of earth pressure at rest for a retained earth surface inclined at β to the horizontal

k	ratio δ/φ'
l	foundation length;
l'	effective foundation length
N_{e1}, N_{q1}	
N	bearing capacity factors (see Annex D)
n	number of e.g. piles or test profiles
P	load on an anchorage (8)
P_d	design value of P (8)
P_p	proof load in a suitability test of a grouted anchorage (8)
$Q_{dst,d}$	design value of the stabilizing variable vertical actions for the uplift verification
q	overburden or surcharge pressure at the level of the foundation base (1.8)
q'	effective overburden pressure at the level of the foundation base; (see Annex D)
$q_{b,k}$	characteristic value of the base resistance pressure
$q_{s,i;k}$	characteristic value of the shaft friction stress
R_a	anchorage pull-out resistance (8)
$R_{a,d}$	design value of R_a (8)
$R_{a,k}$	characteristic value of R_a (8)
$R_{b,cal}$	base resistance, calculated from ground test results at the ultimate limit state (7)
$R_{b,d}$	design value of the base resistance of a pile (7)
$R_{b,k}$	characteristic value of the base resistance of a pile (7)
R_c	compressive resistance of the ground against a pile at the ultimate limit state (7);
$R_{c,d}$	design value of the compressive resistance of the ground against a pile at the ultimate limit state
$R_{c,k}$	characteristic value of the compressive resistance of the ground against a pile at the ultimate limit state
$R_{c,m}$	measured value of R_c in one or several pile load tests (7)
R_d	design value of the resistance to an action (2)
$R_{p,d}$	design value of the resisting force caused by earth pressure on the side of the footing (6)
$R_{s,d}$	design value of the shaft resistance of a pile (7)
$R_{s,cal}$	ultimate shaft friction, calculated using test results from ground parameters
$R_{s,k}$	characteristic value of the shaft resistance of a pile (7)
R_t	ultimate tensile resistance of an isolated pile
$R_{t,k}$	characteristic value of the tensile resistance of a pile or a pile group (7)
$R_{t,d}$	design value of the tensile resistance of a pile or of a group of piles (7); or of the structural tensile resistance of an anchorage (8)
$R_{t,m}$	measured tensile resistance of an isolated pile in one or several pile load tests (7)
R_{tr}	resistance of a pile to transverse loads)
$S_{dst,d}$	design value of the destabilizing seepage force in the ground (2)
$S_{dst,k}$	characteristic value of the destabilizing seepage force in the ground (10)
S_{e1}, S_{q1}	
S_g	the shape factors of the bearing capacity analysis (see Annex D)

T_d	<i>design value of total frictions that develops alongside a block of ground in which a group of tensionpiles is placed, or and the subterrain part of the structure in contact with the ground</i>
s	<i>settlement</i>
s_0	immediate settlement
s_1	settlement caused by consolidation
s_2	settlement caused by creep (secondary settlement)
U_d	design value of the water pressure force acting upward under a pile foundation (7)
u	pore water pressure (1.8)
$u_{dst,d}$	destabilising total pore pressure (10)
$\Delta u_{dst,d}$	excess pore pressure (10)
V	Vertical load, or component of the total action acting normal to the foundation base; (D)
V_d	design value of V (6)
V_d'	design value of the effective vertical load, or component of the total action acting normal to the foundation base (6)
$V_{dst,d}$	design value of the destabilizing vertical action by water pressure below a structure or below an impermeable layer (2)
$V_{dst,k}$	characteristic value of the destabilizing vertical action by water pressure
v_a	wall motion to mobilize active earth pressure (C) (see Annex C)
v_p	wall motion to mobilize passive earth pressure (C) (see Annex C)
W_d	<i>design value of the weight of a structure or ground layer</i>
X_d	design value of a material property
X_k	characteristic value of a material property
z	vertical distance

Greek letters

α	inclination of a foundation base to the horizontal; (D)
β	slope angle of the ground behind a wall (upward positive) (C)
δ	structure-ground interface friction angle
δ_d	design value of δ
φ'	angle of shearing resistance in terms of effective stress
φ'_{cv}	critical state angle of shearing resistance in terms of effective stress
γ	weight density
γ'	weight density, taking account of uplift and/or seepage where appropriate
γ_a	partial factor for anchorages
$\gamma_{a,p}$	partial factor for permanent anchorages (8)
$\gamma_{a,t}$	partial factor for temporary anchorages (8)
γ_b	partial factor on the base resistance of a pile
γ_c	partial factor for the effective cohesion
γ_{cu}	partial factor for the undrained strength
γ_E	partial factor for the effect of an action
γ_F	partial factor for an action
γ_G	partial factor for a permanent action

$\gamma_{G,dst}$	partial factor for a permanent destabilizing action (10)
$\gamma_{G,stab}$	partial factor for a permanent stabilizing action (10)
γ_m	partial factor for a material property
γ_M	partial factor for a material property, also accounting for model uncertainties
γ_Q	partial factor for a variable action
γ_{qu}	partial material factor for unconfined strength
γ_R	partial factor for a resistance
$\gamma_{R,d}$	partial factor covering uncertainty in the resistance model
$\gamma_{R,e}$	partial factor for the earth resistance
$\gamma_{R,h}$	partial factor for the sliding resistance
$\gamma_{R,v}$	partial factor for the bearing resistance
$\gamma_{S,d}$	partial factor taking account of uncertainties in modelling the effects of actions
γ_{dst}	partial factor for a destabilizing action causing hydraulic failure
γ_{stab}	partial factor for a stabilizing action against hydraulic failure
$\gamma_{s,t}$	partial factor on the tensile resistance of a pile (7)(A.2.3.2)
γ_t	partial factor on the total resistance of a pile (7)(A.2.3.2)
γ_w	weight density of groundwater
γ_ϕ	partial material factor for the $\tan \phi$
γ_σ	partial material factor for weight density
θ	direction angle of H . (D)
ξ	correlation factor depending on the number of piles tested or of profiles of tests
ξ_a	correlation factor for anchorages
$\xi_1; \xi_2$	correlation factors to evaluate the results of static pile load tests
$\xi_3; \xi_4$	correlation factors to derive the pile resistance from ground investigation results, not being pile load tests.
$\xi_5; \xi_6$	correlation factors to derive the pile resistance from dynamic impact tests
ψ	factor for converting characteristic value to representative value
$\sigma_{stab,d}$	design value of the stabilising total vertical stress
$\sigma'_{stab,d}$	effective vertical stress
$\sigma'_{h,0}$	horizontal component of effective earth pressure at rest
$\sigma(z)$	stress normal to the wall at depth z
$\tau(z)$	stress tangential to the wall at depth z

Abbreviations

OCR *over-consolidation ratio*

Section 2 Basis of geotechnical design

2.1 Design requirements

(1)P REQ The following factors shall be considered when determining the geotechnical design requirements:

- 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 (neighbouring structures, traffic, utilities, vegetation, hazardous chemicals, etc.);
- ground conditions;
- groundwater conditions;
- regional seismicity;
- influence of the environment (hydrology, surface water, subsidence, seasonal changes of temperature and moisture).

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

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

(4) REC 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) PER 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) REC Buildings should normally be protected against the penetration of groundwater or the transmission of vapour or gases to their interiors.

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

(8)P REQ 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. ~~[The manner in which these minimum requirements are satisfied shall be subject to national determination]~~
- other geotechnical structures.

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

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

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

(11) REC A preliminary classification of a structure according to geotechnical category should normally be performed prior to the geotechnical investigations. ~~[This category may later be changed]~~. The category should be checked and changed, if necessary, at each stage of the design and construction process.

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

(13) POS 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) REC Geotechnical Category 1 ~~[This category]~~ 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) REC Geotechnical Category 1 procedures should be *used* ~~[sufficient]~~ 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 ~~[may be used]~~ for foundation design and construction.

(16) REC Geotechnical Category 1 procedures *should be used* ~~[sufficient]~~ 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) REC Geotechnical Category 2 ~~[This category]~~ should include conventional types of structure and foundation with no exceptional risk or difficult soil or loading conditions [, ~~and for which the estimated risk for site instability or persistent ground movement is negligible.~~]

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

(19) PER 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) REC Geotechnical Category 3 [~~This category~~] should include structures or parts of structures which fall outside the limits of Geotechnical Categories 1 and 2.

(21) REC Geotechnical Category 3 should normally include *alternative* provisions and rules [~~alternative~~] 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 REQ Both short-term and long-term design situations shall be considered.

(2)P REQ In geotechnical design, the detailed specifications of design situations shall 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 [~~nature of 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 groundwater levels, including the effects of dewatering, possible flooding, failure of drainage systems, water exploitation, etc.
- 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 by subsidence due to mining or other causes;
- the sensitivity of the structure to deformations;
- the effect of the new structure on existing structures [ø], services *and the local environment*.

2.3 Durability

(1)P REQ 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) REC In designing for durability of materials used in the ground, the following should be considered:

- a) for concrete:
 - aggressive agents in the groundwater *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 groundwater 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) REC Reference should be made to [any] durability provisions in construction materials standards.

2.4 Geotechnical design by calculation

2.4.1 General

(1)P REQ Design by calculation shall be in accordance with the fundamental requirements of EN 1990 - Basis of Design and with the particular rules of *this standard*. [~~These~~] *Design by calculation* involves:

- actions, which may be either imposed loads or imposed displacements, for example from ground movements;
- properties of soils, rocks and other materials;

- geometrical data;
- limiting values of deformations, crack widths, vibrations etc.
- calculation models.

(2) REC 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 REQ The calculation model shall describe the assumed behaviour of the ground for the limit state under consideration.

(4)P REQ *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 [this] the specific limit state considered is sufficiently improbable.*

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

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

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

(7) PER A calculation model may include simplifications.

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

(9) REC 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 REQ *If an [The] empirical relationship is used in the analysis, it shall be clearly established that it is relevant for the [relevant] prevailing ground conditions.*

(11) REC 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) POS 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) REC Compatibility of strains at a limit state should be considered [~~especially for materials which are brittle or which have strain softening properties~~]. 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 which are brittle or which have strain-softening properties.*

(14) POS 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 [are]* to a great extent [~~dependent~~] on the stiffness of the structure, the stiffness and strength of the ground and the state of stress in the ground.

(15) REC 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 REQ The definition of actions shall be taken from EN 1990. The values of actions shall be taken from EN 1991, where relevant.

(2)P REQ [~~The designer shall select~~] The values of geotechnical actions to be used *shall be selected, since* they are known before a calculation is performed; [~~and are not derived from~~] *they may change during* that calculation.

NOTE: *Values of geotechnical actions may change [be altered]* 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 REQ Any interaction between the structure and the ground shall be taken into account when determining the actions to be adopted in the design.

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

- the weight of soil, rock and water;
- stresses in the ground;
- *earth pressures* and groundwater pressure;
- free water pressures, including wave pressures;
- groundwater 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 prestress in ground anchors or struts;
- ~~[earth pressures]~~
- down-drag.

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

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

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

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

~~[Old clauses (10), (11) and (12) to 2.4.6.1]~~

2.4.3 Ground properties

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

(2)P REQ *Values obtained from* test results ~~[, derived values]~~ and other data shall be interpreted appropriately for the limit state considered.

(3)P REQ 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) POS ~~[These]~~ Differences *as meant in 2.4.3(3)P* 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 (fissures, laminations, large particles, etc) 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)P REQ When establishing values of geotechnical parameters, the following shall 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 REQ Calibration factors shall be applied where necessary to convert laboratory or field test results *according to EN 1997-2 and EN 1997-3* 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 deduce derived values from the test results.

2.4.4 Geometrical data

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

2.4.5.2 Characteristic values of geotechnical parameters

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

(2)P REQ 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 REQ The greater variance of c' compared to that of $\tan \phi'$ shall be considered when the characteristic value is determined.

(4)P REQ 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) POS Characteristic values *can* be lower values, which are less than the most probable values, or upper values, which are greater.

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

(7) REC 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 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) REC 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) REC *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) REC 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) REC 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 REQ *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 REQ Characteristic values of the levels of ground and groundwater or free water shall be measured, nominal or estimated upper or lower levels.

(2) REC 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 REQ The design value of an action shall be determined in accordance with EN 1990.

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

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

with

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

where the symbols are defined in [2.0(4)P] 1.8.

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

(4)P-REQ REC Values of the partial factor γ_F should be *selected* [~~determined in accordance with 2.4.7.1(2)P~~] *from Annex A*. These values indicate the appropriate level of safety for conventional designs.

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

(5)P REQ If design values of geotechnical actions are assessed directly, the partial factors selected [~~determined in accordance with 2.4.7.1(2)P~~] *from Annex A* shall be used as a guide to the required level of safety.

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

(6)P REQ When dealing with groundwater pressures [~~and seepage forces~~] for limit states with severe consequences (generally ultimate limit states), design values shall represent the most unfavourable values which could occur [~~in extreme circumstances~~] 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) PER *In some cases extreme water pressures complying with 1.5.3.5 of EN 1990:2002, may be treated as accidental actions.*

(8) PER Design values of groundwater pressures may [~~also~~] be derived either by applying partial factors to characteristic water pressures or by applying a safety margin to the characteristic water level [~~by raising or lowering the level~~] *in accordance with 2.4.4(1)P and 2.4.5.3(1)P.*

(9)P REQ *The following features which may affect the water pressures shall be considered:*
- the level of the free water surface or the groundwater 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) REC 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) REC Unless the adequacy of the drainage system can be demonstrated and its maintenance ensured, the design groundwater 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 REQ Design values of geotechnical parameters (X_d) shall either be derived from characteristic values using the following equation:

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

where the symbols are as defined in 1.8,

or assessed directly

(2)P-REQ REC Values of the partial factor γ_M should be selected [determined in accordance with 2.4.7.1(2)P] from Annex A. These values indicate the minimum level of safety for conventional designs.

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

(3)P REQ If design values of geotechnical parameters are assessed directly, the partial factors [determined [in accordance with 2.4.7.1(2)P] selected from Annex A shall be used as a guide to the required level of safety.

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

2.4.6.3 Design values of geometrical data

(1) REC 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 REQ 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 and where the symbols are defined in [2.0(1)P] 1.8.

2.4.6.4 Design values of structural properties

(1)P REQ 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 REQ 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 footings, piles, basement walls, etc., in which the strength of structural materials is significant in providing resistance (STR);
 - failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance (GEO);
 - 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-REQ REC [Unless alternative values have been nationally determined,]~~ The partial factors to be used *in persistent and transient situations* should be selected from Annex A.

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

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

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

(4)P REQ More severe values than those identified in ~~[2.4.7.1(2)P]~~ Annex A shall be used in cases of abnormal risk or unusual or exceptionally difficult ground or loading conditions.

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

(5) PER Less severe values than those identified in ~~[2.4.7.1(2)P]~~ Annex A may be used for temporary structures or transient design situations, where the likely consequences justify it.

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

(6) PER When calculating the design value of the resistance, (R_d), or the design value of the effect of actions, (E_d), model factors, (γ_{Rd}) or (γ_{Sd}) respectively, may be introduced to ensure that the results of the design calculation model are either accurate or err on the safe side.

2.4.7.2 Verification of *static equilibrium*

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

$$E_{dst,d} \leq E_{stb,d} \quad (2.4)$$

with

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

and

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

where the symbols are defined in [2.0(1)P] 1.8.

(2)P REQ REC The partial factors *for persistent and transient situations* to be used in equation (2.4) should be *selected* from Annex A (Tables A.1.1 and A.1.2).

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 founded on rock and is, in principle, distinct from overall stability or buoyancy problems.

NOTE 2: The values of the partial factors may be set by the National annex.

2.4.7.3 Verification of *resistance for persistent and transient situations*

2.4.7.3.1 General

(1)P REQ 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:

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

Where:

E_d is the design value of the effects of all the actions,

R_d is the design value of the corresponding resistance.

2.4.7.3.2 Design effects of actions

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

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

or

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

where the symbols are defined in 1.8.

(2) *PER* 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) *REC* The partial factors to be used in equations (2.6a) and (2.6b) should be selected from Annex A (Tables A.2.1 and A.2.2).

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

2.4.7.3.3 Design resistances

(1) *PER* 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

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

where the symbols are defined in 1.8.

(2) *REC* The partial factors to be used in equations (2.7a, b, and c) should be selected from Annex A (Table A.2.2 and Tables A.2.3).

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

2.4.7.3.4 Design Approaches

(1) *P REQ* 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 will be given in the National annex.

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

2.4.7.3.4.1 Design Approach 1

(1) *P REQ* 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".

(2) *P REQ* 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

*when calculating resistances of piles or anchors

†when calculating unfavourable actions on piles owing e.g. to negative skin friction or transverse loading.

(3) PER 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.

2.4.7.3.4.2 Design Approach 2

(1)P REQ 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 $\gamma_{R,e}$.

2.4.7.3.4.3 Design Approach 3

(1)P REQ 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†) “+” M2 “+” R3

*on structural actions

†on geotechnical actions

NOTE 1: In this approach, partial factors are applied to actions or 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, etc.) are treated as geotechnical actions by using the set of load factors A.2.

2.4.7.4 Verification procedure and partial factors for uplift

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

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

where the symbols are defined in 1.8

(2)P REC The values of partial factors for $G_{dst;d}$, $Q_{dst;d}$ and $G_{stb;d}$ for persistent and transient situations should be selected from Annex A (Tables A.3.1 and A.3.2).

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

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

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

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

$$\Delta u_{dst;d} \leq \sigma'_{stb;d} \quad (2.9b)$$

where the symbols are defined in 1.8

(2)P REC The values of partial factors for $\sigma_{stb;d}$, $\sigma'_{stb;d}$, $u_{dst;d}$ and $\Delta u_{dst;d}$ for persistent and transient situations should be selected from Annex A (Table A.4) .

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

2.4.8 Serviceability limit states

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

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

Where:

E_d is the design value of the effects of all the actions,

C_d is the limiting design value of the effect of an action

(2) REC 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) REC Characteristic values should be changed appropriately if changes of ground properties by groundwater lowering, desiccation, etc. may occur during the life of the structure.

(4) PER As an alternative to 2.4.8(1)P, 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 [in 2.4.8(4) shall be]* 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.

~~[old (5)P combined with (4)]~~

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

(2)P REQ 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 REQ moved to 2.4.8]~~

(3)P REQ The amount of permitted foundation movement shall be selected *during the design*.
~~[by the designer or by national determination].~~

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

(4)P REQ 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 there are no problems with the services entering the structure.

~~[(5)P REQ (moved to (3))]~~

(5)P REQ *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.*

(6) PER In the absence of *specified* limiting values of structural deformations ~~[agreed with the designer]~~ of the supported structure, the values of structural deformation and foundation movement given in Annex H may be used.

~~[2.4.10 Verification procedures; moved to 2.4.9(5)P]~~

2.5 Design by prescriptive measures

(1) PER In design situations where calculation models are not available or not necessary, the exceedence of limit states may be avoided by the use of prescriptive measures. These involve conventional and generally conservative ~~[nationally determined,]~~ 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) PER Design by prescriptive measures may be used where comparable experience, as defined in 1.5.2(2)P, 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 REQ 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 effect of stress levels shall be considered, together with the effects of particle size.

(2) *PER* 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) POS 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 REQ The following requirements shall be met before construction is started:

- the limits of behaviour which are acceptable 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 REQ During construction, the monitoring shall be carried out as planned.

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

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

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

(3) REC The 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 REQ 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 in the report. When the required checks have been carried out during construction, they shall be recorded in an addendum to the report.

(5) REC 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 considered;
- 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 monitoring and maintaining the instruments.

(6)P REQ An extract of 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 REQ 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 REQ 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 REQ Routine field investigations and laboratory testing shall be carried out and reported generally in accordance with internationally recognized standards and recommendations. Deviations from these standards and additional test requirements shall be reported.

(4) REC Requirements for equipment and test procedures for laboratory and field testing should be taken from ENV 1997-2 and ENV 1997-3 respectively.

3.2 Geotechnical investigations

3.2.1 General

(1)P REQ Geotechnical investigations shall provide sufficient data concerning the ground and the groundwater 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 REQ The composition and amount of the geotechnical investigations shall be adjusted to the particular investigation phase and the geotechnical category, see ENV 1997-3, Section 2.

(3) POS 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 Eurocode 7 is not sufficient to meet the design requirements.

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

(5) REC The investigations should include visual inspections of the [construction] site to enable *the* design assumptions to be verified [as late as] during *construction* [the supervision of the works].

3.2.2 Preliminary investigations

(1)P REQ Preliminary investigations shall be carried out:

- to assess the general suitability of the site;
- to compare alternative sites, if relevant;
- to estimate the changes which 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 REQ 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 REQ 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 REQ 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)P REQ In order to ensure that the design investigation covers all relevant ground formations, particular attention shall 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 REQ The history of the site and its surroundings shall be taken into account.

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

(7)P REQ The existing groundwater levels shall be established during the investigation. Any free water levels observed during the investigation shall be recorded, see ENV 1997-3. The extreme levels of any free water which might influence the groundwater pressures shall be established.

(8)P REQ 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) *PER* 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 Characterization of soil and rock type

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

(2) *P REQ* The material shall be examined and described in accordance with a recognized nomenclature. A geological evaluation shall be made.

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

(4) *REC* 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 organization, dominant grain size of minerals, and hardness and toughness of the main mineral. Jointing should be characterized in terms of joint type, width, spacing and fill quality.

(5) *PER* In addition to visual inspection, a number of tests for classification, identification and quantification of soils and rocks may be used, see ENV 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 REQ The weight density shall be determined with sufficient accuracy to establish design or characteristic values of the actions which derive from it.

(2) REC The weight density should be determined on specimens of soil and rock when undisturbed samples are taken. In other cases it may be derived from well established or documented correlations based on, for example, penetration tests.

3.3.4 Density index

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

NOTE: The same applies if the relative density index is used instead.

3.3.5 Degree of compaction

(1)P REQ 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 REQ 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;
- preformed slip surfaces;
- time effects;
- sensitivity of cohesive soil;
- degree of saturation.

(2) REC 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) REC 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 REQ 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 REQ In assessing the soil stiffness, the following features shall be considered:

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

(2) REC 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 REQ 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 REQ 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) PER Estimates of rock mass properties, such as:

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

may be obtained by using the concept of rock mass classification described in *section 13, A.13 and B.13 of ENV 1997-2, 13, A.13 and B.13*.

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

(5) REC 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 groundwater 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 REQ 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 to specimen anisotropy, bedding planes, foliation, etc.;
- 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 REQ 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) REC 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 REQ 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) REC 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) REC 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 REQ 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) *PER* 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) REC 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

~~(1)P REQ Reference shall be made to ENV 1997-3 for information on the use of field tests for geotechnical parameters.(moved to 3.4.1)~~

3.3.10.1 Cone penetration test

(1)P REQ In assessing values of the cone resistance, the sleeve friction and, possibly, the pore 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 groundwater and overburden;
- in heterogeneous soils for 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 REQ In assessing blow counts, the following features shall be considered:

- type of test;
- detailed description of the test procedure;
- groundwater 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) PER Vane tests may be used for the assessment of undrained shear strength, c_u , of soft and very soft cohesive soil. (PT NOTE: made consistent with sequence for other tests)~~

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

- details of the test procedure;
- whether standardized 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)P REQ To obtain derived values of c_u , the measured values shall be corrected by a factor depending on the liquid limit, plasticity index and effective vertical stress, and based on local experience.

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

~~(3) PER Vane tests may be used for the assessment of undrained shear strength, c_u , of soft and very soft cohesive soil.~~

3.3.10.4 Weight sounding test

~~(1) REC Weight sounding tests may be used for the assessment of soil layer boundaries and the density of cohesionless soils.~~

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

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

~~(2) PER Weight sounding tests may be used for the assessment of soil layer boundaries and the density of cohesionless soils.~~

3.3.10.5 Pressuremeter test

(1)P REQ In assessing the values of the limit pressure and the pressuremeter modulus, the following features shall be taken into account:

- the type of equipment;
- the procedure used to install the pressuremeter in the ground.

(2) REC Test curves which exhibit more than a moderate degree of disturbance should not be used. Where the limit pressure is not reached during the test, a moderate and conservative extrapolation of the curve may be used to estimate it. For tests in which only the initial part of the pressuremeter curve is determined, general correlations or, preferably, local correlations from the same site may be used conservatively to estimate the limit pressure from the pressuremeter modulus.

3.3.10.6 Dilatometer test

(1)P REQ In assessing dilatometer values the installation procedure shall be taken into account.

(2)P REQ The succession *of soil layers* and especially some basic parameters such as granulometry and degree of saturation shall be determined prior to the test.

(3) REC If strength parameters are to be evaluated, the penetration resistance should be taken into account.

(4) REC The dilatometer values should be used as an index for determining derived values of the stiffness moduli of the soil succession.

3.3.10.7 Compactibility tests

(1)P REQ In assessing the compactibility of a fill material, the following features shall be taken into account:

- type of soil or rock;
- grain size distribution;
- grain shape;
- the heterogeneity of the material;
- the degree of saturation or water content;
- type of plant to be used.

(2) REC When using field measurements (e.g. soundings, dynamic compaction tests, plate load tests, settlement records) to control site compaction, the results of field compaction trials (see 5.3.3(4)) should be related to standard laboratory compaction test values in order to obtain a direct measure of the compactibility of a soil or a rock fill.

3.4 Ground Investigation Report

3.4.1 Requirements

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

(2)P REQ Reference shall be made to ENV 1997-2 and ENV 1997-3 for information on the use of laboratory and field tests for geotechnical parameters.

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

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

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

3.4.2 Presentation of geotechnical information

(1)P REQ The presentation of geotechnical information shall include:

- a factual account of all field and laboratory work;
- documentation of the methods used to carry out the field investigations and the laboratory testing.

The documentation shall be based on the test reports described in ENV 1997-2 and ENV 1997-3.

(2) REC In addition, the factual account should include the following information, as relevant:

- names of all consultants and subcontractors;
- purpose and scope of the geotechnical investigation;
- dates between which field and laboratory work was performed;
- field reconnaissance of the general area of the project noting particularly:
 - evidence of groundwater;
 - behaviour of neighbouring structures;
 - exposures in quarries and borrow areas;
 - areas of instability;
 - difficulties during excavation;
- history of the site;
- geology of the site, including faulting;
- survey data
- information from available aerial photographs;
- local experience in the area;
- information about the seismicity of the area;
- procedures used for sampling and transportation and storage of samples;
- types of field equipment used;
- tabulation of quantities of field and laboratory work, presentation of field observations which were made by the supervising field personnel during the subsurface explorations;
- data on fluctuations of groundwater table with time in the boreholes during the performance of the field work and in piezometers after the completion of the field work;
- compilation of boring logs, including photographs of the cores, with descriptions of subsurface formations based on field descriptions and on the results of the laboratory tests;
- the occurrence, or the possibility of occurrence, of radon;
- description of frost susceptibility of soils;
- grouping and presentation of field and laboratory test results in appendices.

3.4.3 Evaluation of geotechnical information

(1)P REQ The evaluation of the geotechnical information shall include as appropriate:

- a review of the field and laboratory work. Any limitations in the data (e.g. defective, irrelevant, insufficient or inaccurate) shall be pointed out and commented upon. The sampling and sample transportation and storage procedures shall be considered when interpreting the test results. Any particularly adverse test results shall be considered carefully in order to determine if they are misleading or represent a real phenomenon that must be accounted for in the design;
- a review of the derived values of geotechnical parameters;
- any proposals for necessary further field and laboratory work, with comments justifying the need for this extra work. Such proposals shall be accompanied by a detailed programme for the extra investigations to be carried out with specific reference to the questions that have to be answered.

(2) REC In addition, the evaluation of the geotechnical data should include the following, if relevant:

- tabulation and graphical presentation of the results of the field and laboratory work in relation to the requirements of the project and, if deemed necessary,
- histograms illustrating the range of values of the most relevant data and their distribution;
- depth of the groundwater table and its seasonal fluctuations;
- subsurface profile(s) showing the differentiation of the various formations;
- detailed description of all formations including their physical properties and their deformation and strength characteristics;
- comments on irregularities such as pockets and cavities;
- the range and any grouping of derived values of the geotechnical data for each stratum.

Section 4 Supervision of construction, monitoring and maintenance

4.1 General

(1)P REQ To ensure the safety and quality of a structure, the following shall be [~~checked, supervised or monitored~~] *undertaken*, as appropriate:

- [the construction processes [~~including~~] *and* workmanship *shall be supervised*;
- the performance of the structure *shall be monitored* during and after construction;
- [~~maintenance of~~] the structure *shall be adequately maintained*.

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

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

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

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

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

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

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

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

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

(9) REC Annex J gives a checklist for construction supervision and performance monitoring.

4.2 Supervision

4.2.1 Plan of supervision

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

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

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

4.2.2 Inspection and control

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

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

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

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

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

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

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

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

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

4.2.3 Assessment of the design

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

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

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

4.3 Checking ground conditions

4.3.1 Soil and rock

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

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

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

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

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

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

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

NOTE: Normally this is reported to the designer.

(7)P REQ It shall be checked that the principles used in design are appropriate for the geotechnical features of the ground which are encountered.

4.3.2 Groundwater

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

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

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

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

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

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

(7) REC For structures below groundwater level which may ~~float~~ *be subject to uplift*, porewater pressures should be monitored until the weight of the structure is sufficient to rule out the possibility of ~~floating~~ *uplift*.

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

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

(10)P REQ Deviations from the groundwater features assumed in the design shall be reported without delay ~~to the designer~~.

(11)P REQ It shall be checked that the principles used in design are appropriate for the groundwater features which are encountered.

4.4 Checking construction

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

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

(3)P REQ It shall be checked that the principles ~~used~~ *followed* in design are appropriate for the sequence of construction operations which are used.

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

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

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

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

4.5 Monitoring

(1)P REQ ~~The aims of monitoring~~ *Monitoring shall be applied:*

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

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

~~(3) POS Records of the actual performance of structures are important in order to collect data bases for comparable experience.~~

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

(4) REC Monitoring *should* include measurement of the following:

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

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

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

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

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

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

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

(11)P REQ For structures which may have an adverse effect on ground or groundwater conditions, the possibility of leakage or of alterations to the pattern of groundwater flow,

especially when fine grained soils are involved, shall be taken into account when planning the monitoring programme.

(12) POS Examples of this type of structure are:

- water retaining structures;
- structures intended to control seepage;
- tunnels;
- large underground structures;
- deep basements;
- slopes and earth retaining structures;
- ground improvements.

4.6 Maintenance

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

NOTE: normally this is specified to the owner/client.

(2) REC The maintenance specifications should provide information on:

- critical parts of the structure which require regular inspection;
- frequency of the inspection.

Section 5 Fill, dewatering, ground improvement and reinforcement

5.1 General

(1)P REQ The provisions in this section shall apply where adequate ground conditions are achieved by:

- placing natural soil, crushed rock, blasted stone or certain waste products;
- dewatering;
- treating ground;
- reinforcing ground.

NOTE 1: Situations where soil or granular material is placed for engineering purposes include:

- fills beneath foundations and ground slabs;
- backfills to excavations and retaining structures;
- general landfill including hydraulic fill, landscape mounds and spoil heaps;
- embankments ~~for dikes~~, small dams and infrastructure ~~[networks]~~.

NOTE 2: Dewatering of ground may be temporary or permanent.

NOTE 3: Ground which is treated to improve its properties may be either natural ground or fill. The ground improvement may be either temporary or permanent.

(2)P REQ Design procedures for geotechnical works that include the use of fill, dewatering, improvement and reinforcement shall be those presented in Sections 6 to 12.

5.2 Fundamental requirements

(1)P REQ Fill and dewatered, improved or reinforced ground shall be capable of sustaining the actions caused by loads, seeping water, vibrations, temperature, rain, etc.

(2)P REQ These fundamental requirements shall also be satisfied for the ground on which the fill is placed.

5.3 Fill construction

5.3.1 Principles

(1)P REQ *When designing fill constructions it shall be considered that the adequacy of the fill shall depend on the following:*

- good material handling properties,
- adequate engineering properties after compaction.

(2) REC Transport and placement *of the fill* should be considered in the design.

5.3.2 Selection of fill material

(1)P REQ The criteria for specifying material as suitable for use as fill shall be based on achieving adequate strength, stiffness and permeability after compaction. These criteria shall take account of the purpose of the fill and the requirements of any structure to be placed on it.

(2) PER Suitable fill materials *may* include most graded natural granular materials and certain waste products such as selected colliery waste and pulverised fuel ash. Some manufactured materials, such as light aggregate, may also be used in some circumstances. Some cohesive materials may be suitable but require particular care.

(3)P REQ The following aspects shall be taken into account when specifying a fill material:

- grading;
- resistance to crushing;
- compactibility;
- permeability;
- plasticity;
- strength of underlying ground;
- organic content;
- chemical aggression;
- pollution effects;
- solubility;
- susceptibility to volume changes (swelling clays and collapsible materials);
- low temperature and frost susceptibility;
- resistance to weathering;
- effect of excavation, transportation and placement;
- possibility of cementation occurring after placement (e.g. blast furnace slags).

(4) POS If local materials in their natural state are not suitable for use as fill, it *can* be necessary to adopt one of the following procedures:

- adjust the water content;
- mix with cement, lime or other materials;
- crush, sieve or wash;
- protect with appropriate material;
- use drainage layers.

(5) REC Frozen, expansive or soluble soils should not normally be used as a fill material.

(6)P REQ When the selected material contains potentially aggressive or polluting chemicals, adequate provisions shall be adopted to prevent it from attacking structures or services or polluting the groundwater. Such materials shall only be used in large amounts in permanently monitored locations.

(7)P REQ In case of doubt, the fill material shall be tested at source to ensure that it is suitable for its intended purpose. The type, number and frequency of the tests shall be selected according to the type and heterogeneity of the material and the nature of the project.

(8) POS In Geotechnical Category 1, inspection of the material ~~will~~ *may* often be sufficient.

(9)P REQ Material used for fill with specified severe requirements regarding bearing resistance, settlement and ~~support~~ *stability* shall not contain matter such as snow, ice or peat in any significant amount.

(10) PER In fills with no specified requirements for bearing resistance, settlement or ~~retainment limitations~~ *stability*, the fill material may contain small amounts of snow, ice or peat.

5.3.3 Selection of procedures for fill placement and compaction

(1)P REQ Compaction criteria shall be established for each zone or layer of fill, related to its purpose and performance requirements.

(2)P REQ The procedures for fill placement and compaction shall be specified in such a way that stability of the fill is ensured during the entire construction period and the natural subsoil is not adversely affected.

(3)P REQ The compaction procedure for fill shall be specified depending on the compaction criteria and on the following:

- the origin and nature of the material;
- the placement method;
- the placement water content and its possible variations;
- the initial and final thicknesses of the lift;
- the air temperature and rainfall;
- the uniformity of compaction;
- the nature of underlying ground.

(4) REC In order to develop an appropriate procedure for compaction, a test should be performed at the site and with the intended material and compaction equipment. This allows the determination of the compaction procedure (layer thicknesses, number of passes, adequate techniques for transportation, amount of water that shall be added and deposition of the material) to be followed. It may also be used to establish the control criteria.

(5) REC Where there is a possibility of rainfall during the placement of cohesive fill material, the fill surface should at all stages be profiled so as to permit adequate run-off.

(6) REC At temperatures below the freezing point, fill may require heating before placement and frost protection of the fill surface. The need of these measures should be evaluated case by case in the design taking into account the quality of the fill material and the requirement concerning the degree of compaction.

(7)P REQ Backfill placed around foundations and beneath floor slabs shall be compacted such that damaging subsidence does not occur.

(8) REC Fill should be placed on an undisturbed and drained ground surface. Any mixing of the fill with the ground should be prevented by using a filter textile or filter layer.

(9) REC Before placing fill underwater, ~~any~~ *all* soft material *encountered* should be removed by dredging or other means.

5.3.4 Checking the fill

(1)P REQ Fill shall be inspected or tested to ensure that the material, its placement water content and the compaction procedures comply with the specification.

(2) PER Testing *need not be performed* for some combinations of materials and compaction procedures if the compaction procedure has been proved by a field trial or by comparable experience.

(3) REC Compaction should be tested by one of the following methods:

- measurement of dry density and, if required by the design, measurement of the water content;
- measurement of properties such as penetration resistance, stiffness, etc. Such measurement may not determine if satisfactory compaction has been achieved in cohesive soils.

(4) REC Minimum fill densities, determined, for example by Proctor percentages, should be specified and checked on site.

(5) REC *For rock fill or fill containing a large amount of coarse particles, compaction should be checked by field methods.* The Proctor test is not applicable to ~~rock fill or fill containing a large amount of coarse particles.~~ For these materials. ~~compaction should be checked by field methods~~

(6) [REC]PER Site checking (see ENV 1997-2) may be made by *one of the following*:

- ensuring that compaction has been performed according to the procedure deduced from a field trial or from comparable experience;
- checking that the settlement induced by an additional pass of the compaction equipment is equal to or less than a specified value;
- plate loading tests;
- seismic or dynamic methods.

(7)P REQ In cases where over-compaction is not acceptable, an upper bound limit for the compaction shall be specified.

(8) POS Over-compaction may cause the following undesirable effects:

- the development of slickensides and high soil stiffnesses in ~~dikes and~~ slopes;
- high earth pressures on buried and earth retaining structures;
- crushing of materials such as soft rocks, slags, volcanic sands used as light weight fills.

5.4 Dewatering

(1)P REQ Any scheme for removing water from the ground or for lowering the water pressure shall be based on the results of a geotechnical or hydrogeological investigation.

(2) [REC]PER Water may be removed from the ground by gravity drainage, by pumping from sumps, well points or bored wells, or by electro-osmosis. The scheme adopted will depend on:

- the existing ground and groundwater conditions;
- the characteristics of the project: e.g. excavation depth and extent of dewatering.

(3) PER Part of the dewatering system may be a system of recharge wells at some distance from the excavation.

(4)P REQ ~~[The]~~ ***In the*** dewatering scheme ~~[shall satisfy]~~ the following conditions ***shall be considered*** as appropriate:

- *in the case of excavations, the sides of the excavation shall remain stable at all times under the effect of groundwater lowering and excessive heaving or rupture of the base, for example due to excessive water pressure beneath a less permeable layer, shall not occur;*

- the scheme shall not lead to excessive settlements or damage to nearby structures ~~for water deficiency to agriculture~~];
- the scheme shall avoid excessive loss of ground by seepage from the side or base of the excavation;
- except in the case of fairly uniformly graded material which can establish itself as a filter material, adequate filters shall be provided around the sumps to ensure that there is no significant transportation of soil with the pumped water;
- water removed from an excavation shall normally be discharged well clear of the excavated area;
- the dewatering scheme shall be so designed, arranged and installed as to maintain the water levels and pore pressures anticipated in the design without significant fluctuations;
- there shall be adequate margin of pumping capacity and ~~standby plant~~ *back-up capacity* shall be available in the case of breakdown;
- when allowing the groundwater to return to its original level, care shall be taken to prevent problems such as collapse of soils having a sensitive structure, e.g. loose sand;
- the scheme shall not lead to excessive transport of contaminated water to the excavation;
- the scheme shall not lead to excessive extraction of water in a drinking water catchment area.
- ~~failure of the pumping system shall be prevented by the provision of a power back-up supply, or other means. (PT NOTE: repetition, see above)~~

(5)P REQ The effectiveness of dewatering shall be checked by monitoring the groundwater level, the pore pressures and the ground movements as necessary. Data shall be reviewed and interpreted frequently to determine the effects of dewatering on the ground conditions and on the behaviour of nearby structures.

(6)P REQ If a pumping operation is to continue over a long period of time, the groundwater shall be checked for the presence of dissolved salts and gases which could either result in corrosion of the well screens or cause clogging of the screens by the precipitation of salts.

(7)P REQ Systems for long term dewatering shall be designed to prevent clogging by bacterial action or other causes.

5.5 Ground improvement and reinforcement

(1)P REQ A geotechnical investigation of the initial ground conditions shall be carried out before any ground improvement or reinforcement method is chosen or used.

(2)P REQ The ground improvement method for a particular situation shall be designed taking into account the following factors where appropriate:

- thickness and properties of the ground or fill material;
- magnitude of water pressure in the various strata;
- nature, size and position of the structure to be supported by the ground;
- prevention of damage to adjacent structures or services;
- if the ground improvement is temporary or permanent;
- in terms of anticipated deformations, the relationship between the ground improvement method and the construction sequence;
- the effects on the environment including pollution by toxic substances or changes in groundwater level;
- the long-term deterioration of materials.

(3)P REQ The effectiveness of the ground improvement shall be checked against the acceptance criteria by determining the induced changes in the appropriate ground properties.

Section 6 Spread foundations

6.1 General

(1)P REQ The provisions of this section apply to spread foundations including pads, strips and ~~slabs~~ *rafts*.

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

6.2 Limit states

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

- loss of overall stability;
- bearing resistance failure, punching failure, squeezing;
- failure by sliding;
- combined failure in *the* ground and in *the* structure;
- structural failure due to foundation movement;
- excessive settlements;
- excessive heave due to swelling, frost and other causes;
- unacceptable vibrations.

6.3 Actions and design situations

(1)P REQ Design situations shall be selected in accordance with the principles stated in 2.2.

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

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

6.4 Design and construction considerations

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

- reaching an adequate bearing stratum;
- the depth above which shrinkage and swelling of clay soils, due to seasonal weather changes, or to trees and shrubs, may cause appreciable movements;
- the depth above which frost damage may occur;
- the level of the water table in the ground and the problems which may occur if excavation for the foundation is required below this level;
- possible ground movements and reductions in the strength of the bearing stratum by seepage or climatic effects or by construction procedures;
- the effects of excavations on nearby foundations and structures;
- future excavations for services close to the foundation;
- high or low temperatures transmitted from the building;
- the possibility of scour;
- the effects of variation of water content due to long periods of droughts and subsequent periods of rain on the properties of volume unstable soils in arid climatic areas;

- the presence of soluble materials such as e.g. limestone, claystone, gypsum, salt rocks etc.

(2) ~~REC-POS~~ Frost damage ~~may~~ can be avoided if:

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

~~(3) REC Reference may be made to EN ISO 13793 – Frost protection of building.~~

(3) *PER prEN-ISO 13793:1995 may be applied for frost protecting measures for building foundations.*

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

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

- a direct method, in which separate analyses are carried out for each limit state. When checking against an ultimate limit state, the calculation shall model the failure mechanism which is envisaged as closely as possible. When checking against a serviceability limit state, a settlement calculation shall be used;
- a semi-empirical method using comparable experience and the results of field or laboratory measurements or observations, and chosen in relation to serviceability limit state loads so as to satisfy the requirements of all relevant limit states;
- a prescriptive method in which a presumed bearing resistance is used (see 2.5).

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

6.5 Ultimate limit state design

6.5.1 Overall stability

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

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

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

6.5.2 Bearing resistance

6.5.2.1 General

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

$$V_d \leq R_d \quad (6.1)$$

where the symbols are defined in [6.0(4)P] 1.8.

(2)P REQ R_d shall be calculated according to the principles outlined in 2.4.

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

6.5.2.2 Direct method

(1) ~~REC~~ PER The sample analytical calculation for bearing resistance given in Annex D may be used.

(2)P REQ An analytical evaluation of the short-term and long-term values of R_d shall be considered, particularly in fine-grained soils where changes of pore water pressure may lead to changes in shear strength.

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

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

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

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

(7) PER The overall stability calculations described in section 11 may be *applied*.

6.5.2.3 Semi-empirical method

(1) REC *The sample semi-empirical method for bearing resistance estimation using pressuremeter test results given in Annex E is recommended.* ~~sample semi-empirical method for bearing resistance estimation using pressuremeter test results.~~

6.5.2.4 Prescriptive method using presumed bearing resistance

(1) REC *The sample method for deriving the presumed bearing resistance for spread foundations on rock given in Annex G is recommended.* ~~sample method for deriving the presumed bearing resistance for spread foundations on rock.~~ When this method is applied, the design result should be evaluated on the basis of comparable experience.

6.5.3 Sliding resistance

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

(2)P REQ The following inequality shall be satisfied:

$$H_d \leq R_d + R_{p;d} \quad (6.2)$$

where the symbols are defined in [6.0(1)P] 1.8.

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

(4)P REQ R_d shall be calculated according to the principles outlined in 2.4.

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

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

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

(8)P REQ For drained conditions, the design shear resistance, R_d , shall be calculated using the following equation:

$$R_d = V_d' \tan \delta_d \quad (6.3)$$

where the symbols are defined in 1.8.

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

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

(11)P REQ For undrained conditions, the design shearing resistance, R_d , shall in general be limited by:

$$R_d = A_c \times c_{u,d} \quad (6.4)$$

where the symbols are defined in [6.0(1)P] 1.8.

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

$$R_d \leq 0.4 \cdot V_d \quad (6.5)$$

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

6.5.4 Loads with large eccentricities

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

Such precautions include:

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

[Note: For rectangular footings with eccentricities of loading in two orthogonal directions, special consideration is recommended, see fig. 6.1]

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

6.5.5 Structural failure due to foundation movement

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

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

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

6.6 Serviceability limit state design

6.6.1 General

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

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

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

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

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

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

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

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

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

~~(9)P REQ Account shall be taken of displacements caused by actions on the foundation, such as those listed in 2.4.2(4). (moved to (1))~~

[Fig 6.1 deleted]

6.6.2 Settlement

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

(2) REC The following three components of settlement should be considered for partially or fully saturated soils:

- s_0 : immediate settlement; for fully-saturated soil due to shear deformation at constant volume, and for partially-saturated soil due to both shear deformation and volume reduction;
- s_1 : settlement caused by consolidation;
- s_2 : settlement caused by creep.

(3) ~~REC PER~~ *The sample methods for evaluating settlements s_0 and s_1 given in Annex F gives may be applied.* ~~sample methods for evaluating settlements s_0 and s_1 .~~

(4) REC Special consideration should be given to soils such as organic soils and sensitive clays, in which settlement may be prolonged almost indefinitely due to creep.

(5) REC For compressible soil layers, the depth to be considered should depend on the size and shape of the foundation, the variation in soil stiffness with depth and the spacing of foundation elements.

(6) PER This depth may normally be taken as the depth at which the effective vertical stress due to the foundation load is 20% of the effective overburden stress.

(7) PER For many cases this depth may also be roughly estimated as 1 to 2 times the foundation width, but may be reduced for lightly-loaded, wider foundation rafts. This approach is not valid for very soft soils.

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

(9) REC The following should be considered:

- the possible effects of self-weight, flooding and vibration in fill and collapsible soils;
- the effects of stress changes in crushable sands.

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

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

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

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

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

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

(16) REC For conventional structures founded on clays, the ratio of the bearing capacity of the ground, at its initial undrained strength, to the applied serviceability loading should be calculated (see 2.4.8(4)). If this ratio is less than 3 calculations of settlements should always be

undertaken. If the ratio is less than 2, the calculations should take account of non-linear stiffness effects in the ground.

6.6.3 Heave

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

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

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

6.6.4 Vibration analysis

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

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

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

6.7 Foundations on rock: additional design considerations

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

- the deformability and strength of the rock mass and the permissible settlement of the supported structure;
- the presence of any weak layers, solution features, fault zones, etc. beneath the foundation;
- the presence of bedding *joints* and other discontinuities and their characteristics (e.g. filling, continuity, width, spacing);
- the state of weathering, decomposition and fracturing of the rock;
- disturbance of the natural state of the rock caused by construction activities, such as underground works, ~~excavated slopes~~ *slope excavation*, etc. being near to the foundation.

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

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

6.8 Structural design of spread foundations

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

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

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

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

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

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

6.9 Preparation of the subsoil

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

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

Section 7 Pile foundations

7.1 General

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

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

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

- EN 1536:1999, for bored piles
- EN 12063:2000, for sheet pile walls,
- EN 12699:2000, for displacement piles.

NOTE : EN 14199 Execution of special geotechnical works - Micro piles is in preparation.

7.2 Limit states

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

- loss of overall stability;
- bearing resistance failure of the pile foundation;
- uplift or insufficient tensile resistance of the pile foundation;
- failure in the ground due to transverse loading of the pile foundation;
- structural failure of the pile in compression, tension, bending, buckling or shear;
- combined failure in ground and in the pile foundation;
- combined failure in ground and in the structure;
- excessive settlement;
- excessive heave;
- excessive lateral movement of the ground;
- unacceptable vibrations.

7.3 Actions and design situations

7.3.1 General

(1)P REQ The actions listed in 2.4.2.(4)P shall be considered when selecting the design situations.

(2) POS Piles can be loaded axially and/or transversally

(3)P REQ Design situations shall be derived in accordance with the principles stated in 2.2.

(4) POS An analysis of the interaction between structure, pile foundation and ground *can* be necessary to prove that the limit state requirements are met.

7.3.2 Actions due to ground displacement

7.3.2.1 General

(1)P REQ Ground in which piles are located may be subject to displacement caused by consolidation, swelling, adjacent loads, creeping soil, landslides or earthquakes. *Consideration shall be given to these phenomena ~~that~~ as they can affect the piles by causing downdrag (negative skin friction), heave, stretching, transverse loading and displacement~~[- shall be considered]~~.*

(2) REC For these situations, the design values of the strength and stiffness of the moving ground should usually be upper values.

(3)P REQ One of the two following approaches shall be adopted for design:

- the ground displacement is treated as an action. An interaction analysis is then carried out to determine the forces, displacements and strains in the pile;
- an upper bound to the force which the ground could transmit to the pile shall be introduced as the design action. Evaluation of this force shall take account of the strength of the soil and the source of the load, represented by the weight or compression of the moving soil or the magnitude of disturbing actions.

7.3.2.2 Downdrag (negative skin friction)

(1)P REQ If ultimate limit state design calculations are carried out with the downdrag load as an action, its value shall be the maximum which could be generated by the downward movement of the ground relative to the pile.

(2) REC Calculation of maximum downdrag loads should take account of the shear resistance at the interface between the soil and the pile shaft and downward movement of the ground due to self-weight compression and any surface load around piles.

(3) PER An upper bound to the downdrag load on a group of piles may be calculated from the weight of the surcharge causing the movement and taking into account any changes in groundwater pressure due to groundwater lowering, consolidation or pile driving.

(4) PER Where settlement of the ground after pile installation is expected to be small, an economic design may be obtained by treating the settlement of the ground as the action and carrying out an interaction analysis.

(5)P REQ The design value of the settlement of the ground shall be derived taking account of material weight densities and compressibility in accordance with 2.4.3.

(6) REC Interaction calculations should take account of the displacement of the pile relative to the moving surrounding ground, the shear resistance of the soil along the shaft of the pile and the weight of the soil and the expected surface loads around each pile which are the cause of downdrag.

~~(7) REC Normally it should not be necessary to consider downdrag and transient loading simultaneously in load combinations.~~

(7) *PER Normally down-drag and transient loading need not be considered simultaneously in load combinations*

7.3.2.3 Heave

(1)P REQ In considering the effect of heave, or upward loads which may be generated along the pile shaft, the movement of the ground shall generally be treated as an action.

NOTE 1: Expansion or heave of the ground can result from unloading, excavation, frost action or driving of adjacent piles. It can also be due to an increase of the groundwater content resulting from the removal of trees, cessation of abstraction from aquifers, prevention (by new construction) of evaporation and from accidents.

NOTE 2: Heave may take place during construction, before piles are loaded by the structure, and may cause unacceptable uplift or structural failure of the piles.

7.3.2.4 Transverse loading

(1)P REQ Consideration shall be given to transverse actions originated by ground movements around the pile.

(2) REC Consideration should be given to the following list of design situations which may result in transverse actions on a pile:

- different amounts of surcharge on either side of a pile foundation;
- different levels of excavation on either side of a pile foundation;
- a pile foundation located at the edge of an embankment;
- a pile foundation constructed in a creeping slope;
- inclined piles in settling ground;
- piles in a seismic region.

(3) REC Transverse loading should normally be evaluated by considering the interaction between the piles as stiff or flexible beams and the moving soil mass. When the horizontal deformation of weak soil layers is large and the piles are widely spaced, the resulting transverse loading of the piles depends mainly on the shear strength of the weak soil layers.

7.4 Design methods and design considerations

7.4.1 Design methods

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

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

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

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

7.4.2 Design considerations

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

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

(3)P REQ Planned future placing or removal of overburden or potential changes in the groundwater regime shall be considered, both in calculations and in the interpretation of load test results.

(4)P REQ The choice of type of pile, including the quality of the pile material and the method of installation, shall take into account:

- the ground and groundwater conditions on the site, including the presence or possibility of obstructions in the ground;
- the stresses generated in the piles during installation;
- the possibility of preserving and checking the integrity of the pile being installed;
- the effect of the method and sequence of pile installation on piles which have already been installed and on adjacent structures or services;
- the tolerances within which the piles can be installed reliably;
- the deleterious effects of chemicals in the ground;
- the possibility of connecting different groundwater regimes;
- the handling and transportation of piles;
- the effects of pile construction on neighbouring buildings.

(5) REC In considering the aspects listed above, the following items should receive attention:

- the spacing of the piles in pile groups;
- displacement or vibration of adjacent structures due to pile installation;
- the type of hammer or vibrator used;
- the dynamic stresses in the pile during driving;
- for those types of bored piles where a fluid is used inside the borehole, the need to keep the pressure of the fluid at a level to ensure that the borehole will not collapse and hydraulic failure of the base will not occur;
- cleaning of the base and sometimes the shaft of the borehole, especially under bentonite, to remove remolded materials;
- local instability of a shaft during concreting which may cause a soil inclusion within the pile;
- ingress of soil or water into the section of cast-in-situ piles and possible disturbance of wet concrete by the flow of water through it;
- the effect of unsaturated sand layers around a pile extracting water from the concrete;
- the retarding influence of chemicals in the soil;
- soil compaction due to the driving of displacement piles;
- soil disturbance due to the boring of shafts for piles.

7.5 Pile load tests

7.5.1 General

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

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

(2) PER Load tests on piles may be used to:

- assess the suitability of the construction method;
- determine the response of a representative pile and the surrounding ground to load, both in terms of settlement and limit load;
- to allow judgement of the overall pile foundation.

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

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

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

(6)P REQ Between the installation of the test pile and the beginning of the load test, adequate time shall be allowed to ensure that the required strength of the pile material is achieved and the pore water pressures have regained their initial values.

(7) POS In some cases it *can* be necessary to record the pore water pressures caused by pile installation and their subsequent dissipation in order to take a proper decision regarding the start of the load test.

7.5.2 Static load tests

7.5.2.1 Loading procedure

(1)P REQ The pile load test procedure⁵, particularly with respect to the number of loading steps, the duration of these steps and the application of load cycles, shall be such that conclusions can be drawn about the deformation behaviour, creep and rebound of a piled foundation from

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

the measurements on the pile. For trial piles, the loading shall be such that conclusions can also be drawn about the ultimate failure load.

(2) REC Devices for the determination of loads, stresses or strains and displacements should be calibrated prior to the test.

(3) REC The direction of the test load applied to compression or tensile piles should coincide with the longitudinal axis of the pile.

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

7.5.2.2 Trial piles

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

- the ground conditions and their variability across the site;
- the geotechnical category of the structure, *if appropriate*;
- previous documented evidence of the performance of the same type of pile in similar ground conditions;
- the total number and types of piles in the foundation design.

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

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

7.5.2.3 Working piles

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

~~(2) REC The selection of working piles for testing should relate to the results of pile installation records.~~

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

7.5.3 Dynamic load tests

(1) PER Dynamic load tests⁶ may be used to estimate the compressive resistance provided an adequate site investigation has been carried out and the method has been calibrated against static load tests on the same type of pile, of similar length and cross-section, and in comparable soil conditions, see 7.6.2.4 to 7.6.2.6.

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

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

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

7.5.4 Load test report

(1)P REQ It shall be specified that a factual report shall be written for all load tests. Where appropriate, this report shall include:

- a description of the site;
- the ground conditions with reference to ground investigations;
- the pile type;
- description of the pile installation and of any problems encountered during the works;
- a description of the loading and measuring apparatus and the reaction system;
- calibration documents for the load cells, the jacks and the gauges;
- the installation records of the test piles;
- photographic records of the pile and the test site;
- test results in numerical form;
- time-displacement plots for each applied load when a step loading procedure is used;
- the measured load-displacement behaviour;
- reasons for any departures from the above requirements .

7.6 Axially loaded piles

7.6.1 General

7.6.1.1 Limit state design

(1)P REQ The design shall demonstrate that *the exceedence of* the following [~~classes of~~] limit states ~~are-is~~ sufficiently improbable:

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

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

(3) REC For piles in compression it is often difficult to define an ULS from [~~the~~] a load settlement [~~curve~~] *plot* showing a continuous curvature. In these cases, [~~it is widely accepted practice to use a~~] settlement *of the pile top* equal to 10% of the pile base diameter *should be adopted* as the "failure" criterion.

(4)P REQ For piles which undergo significant settlements, ultimate limit states may occur in supported structures before the resistance of the piles is fully mobilized. In these cases a cautious estimate of the possible range of the [~~whole of the load-settlement curve~~] *settlements* shall be *adopted in design*. Settlement of piles is considered in 7.6.4

7.6.1.2 Overall stability

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

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

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

7.6.2 Compressive ground resistance

7.6.2.1 General

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

$$F_{c;d} \leq R_{c;d} \quad (7.1)$$

with the symbols defined in 1.8.

(2) REC In principle $F_{c;d}$ should include the weight of the pile itself and $R_{c;d}$ should include the overburden pressure of the soil at the foundation base. However these two items may be disregarded if they cancel approximately. They *need* not cancel ~~when~~ if:

- downdrag is significant;
- the soil is very light,
- the pile extends above the surface of the ground.

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

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

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

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

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

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

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

(8) REC Special attention should be given to possible failure of edge piles caused by inclined or eccentric loads from the supported structure.

(9)P REQ If the ~~stratum~~ *layer* in which the piles bear overlies a layer of weak soil, the effect of the weak layer on the compressive resistance of the foundation shall be considered.

(10)P REQ The strength of a zone of ground above and below the pile base shall be taken into account when calculating the pile base resistance.

NOTE: This zone may extend several diameters above and below the pile base. Any weak ground in this zone has a relatively large influence on the base resistance.

(11) REC Punching failure should be considered if weak ground is present at a depth of less than 4 times the base diameter below the base of the pile.

(12)P REQ Where the pile base diameter exceeds the shaft diameter, the possible adverse effect shall be considered.

(13) REC For open-ended driven tube or box-section piles with openings of more than 500 mm in any direction and without special devices inside the pile to induce plugging, the base resistance should be limited to the smaller of:

- the shearing resistance between the soil plug and the inside face of *the* pile;
- the base resistance derived using the cross sectional area of the base.

7.6.2.2 Ultimate compressive resistance from static load tests

(1)P REQ The manner in which load tests are carried out shall be in accordance with 7.5 and shall be specified in the geotechnical design report.

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

(3) REC If the diameter of the trial pile differs from that of the working piles, the possible difference in performance of piles of different diameters should be considered in assessing the compressive resistance to be adopted.

(4) *PER* In the case of a very large diameter pile, it is often impractical to carry out a load test on a full size trial pile. Load tests on smaller diameter trial piles may be considered provided that:

- the ratio of the trial pile diameter/working pile diameter is not less than 0,5;
- the smaller diameter trial piles are fabricated and installed in the same way as the piles used for the foundation;
- the trial pile is instrumented in such a manner that the base and shaft resistances can be derived separately from the measurements.

This approach should be used with caution for open-ended driven piles because of the influence of the diameter on the mobilization of the compressive resistance of a soil plug in the pile.

(5)P REQ *In the case of a pile foundation subjected to downdrag, the pile resistance at failure, or at a displacement which equals the criterion for the verification of the ultimate limit state determined from the load test results, shall be corrected. The correction shall be achieved by subtracting the measured, or the most unfavourable, positive shaft resistance in the*

compressible stratum where negative skin friction develops, from the loads measured at the pile head.

(6) REC During the load test of a pile subject to downdrag, positive shaft friction will develop along the total length of the pile and should be considered in accordance with 7.3.2.2(6). The maximum test load applied to the working pile should be in excess of the sum of the design external load plus twice the downdrag force.

(7)P REQ When deriving the ultimate characteristic compressive resistance $R_{c;k}$ from values $R_{c;m}$ measured in one or several pile load tests, an allowance shall be made for the variability of the ground and the variability of the effect of pile installation.

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

$$R_{c;k} = \text{Min} \{ (R_{c;m})_{\text{mean}} / \xi_1 ; (R_{c;m})_{\text{min}} / \xi_2 \} \quad (7.2)$$

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

(9)P REQ REC Values of ξ_1 and ξ_2 to be used in equation (7.2) should be selected in accordance with 2.4.7.1(2)P from Annex A.

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

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

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

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

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

$$R_{c;k} = R_{b;k} + R_{s;k} \quad (7.3)$$

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

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

$$R_{c;d} = R_{c;k} / \gamma_t \quad (7.4)$$

or

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

where the symbols are defined in 1.8.

(16) ~~P-REQ REC~~ Values of γ_b and γ_s or γ_t , to be used in equations (7.4) and (7.5) should be ~~taken in accordance with 2.4.7.1(2)P~~ selected from Annex A .

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

7.6.2.3 Ultimate compressive resistance from ~~[semi-empirical methods]~~ **ground test results**

(1) ~~P-REQ [The semi-empirical m]~~Methods ~~[used to assess]~~ for assessing the compressive resistance of a pile foundation *from ground test results* shall have been established from pile load tests and from comparable experience as defined in 1.5.2(2).

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

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

$$R_{c;d} = R_{b;d} + R_{s;d} \quad (7.6)$$

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

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

where the symbols are defined in 1.8.

(5) ~~P-REQ REC~~ Values of γ_b and γ_s should be ~~taken]~~ selected from Annex A. ~~[in accordance with 2.4.7.1(2)P.]~~

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

(6) ~~P-REQ~~ The characteristic values $R_{b;k}$ and $R_{s;k}$ shall be determined by

$$R_{c;k} = (R_{b;k} + R_{s;k}) = \frac{R_{b;cal} + R_{s;cal}}{\xi} = \frac{R_{c;cal}}{\xi} = \text{Min} \left\{ \frac{(R_{c;cal})_{mean}}{\xi_3}, \frac{(R_{c;cal})_{min}}{\xi_4} \right\} \quad (7.8)$$

where ξ_3 and ξ_4 are correlation factors that depend on the number of *profiles of tests*, n , and are applied respectively:

- to the mean values $(R_{c;cal})_{mean} = (R_{b;cal} + R_{s;cal})_{mean} = (R_{b;cal})_{mean} + (R_{s;cal})_{mean}$
- and to the lowest values $(R_{c;cal})_{min} = (R_{b;cal} + R_{s;cal})_{min}$.

(7) ~~P-REQ REC~~ Values of ξ_3 and ξ_4 to be used in equation 7.8 should be ~~taken]~~ selected ~~[in accordance with 2.4.7.1(2)P]~~ from Annex A.

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

(8) ~~P-REQ~~ The systematic and random components of the variation in the ground shall be recognized in the interpretation of the ground tests and calculated resistances.

(9) PER For structures with sufficient stiffness and strength to transfer loads from weak to strong piles, the factors ξ_3 and ξ_4 may be divided by 1,1, provided that ξ_3 is never less than 1,0.

(10) PER As an alternative to (6), the characteristic values may be obtained by calculating

$$R_{b;k} = A_b \cdot q_{b;k} \text{ and } R_{s;k} = \sum_i A_{s;i} \cdot q_{s;k;i} \quad (7.9)$$

where $q_{b;k}$ and $q_{s;k;i}$ (see 1.8) are characteristic values of base resistance and shaft friction *in the various strata*, [determined]-obtained from values of ground parameters.

NOTE : If this alternative procedure is applied, the values of the partial factors γ_b and γ_s selected according to 2.4.7.1(2) from Annex A, may need to be corrected by a model factor larger than 1,0. The values of the model factor may be set by the National annex.

(11)P REQ If Design Approach 3 is used, the characteristic values of ground parameters shall be determined according to 2.4.5. Partial factors shall then be applied to these characteristic values to obtain design values of the ground parameters for calculating the design values of the pile resistance.

(12) REC In assessing the validity of a model *based on ground test results*, the following items should be considered:

- soil type, including grading, mineralogy, angularity, density, preconsolidation, compressibility and permeability;
- method of installation of the pile, including method of boring or driving;
- length, diameter, material and shape of the shaft and of the base of the pile (e.g. enlarged base);
- method of ground testing.

7.6.2.4 Ultimate compressive resistance from dynamic impact tests

(1)P REQ Where a dynamic impact (hammer blow) pile test (measurement of strain and acceleration versus time during the impact event, see 7.5.3(1)) is used to assess the resistance of individual compression piles, the validity of the result shall have been demonstrated by previous evidence of acceptable performance in static load tests on the same pile type of similar length and cross section and in similar ground conditions.

(2) REC When using a dynamic impact load test, the driving resistance of the pile *should be* measured directly on the site in question. A load test of this type *can* also include a process of signal matching to measured stress wave figures. Signal matching enables an approximate evaluation of shaft and base resistance of the pile as well as a simulation of its load-settlement behaviour.

(3)P REQ The impact energy shall be high enough to allow for an appropriate interpretation of the pile capacity at a correspondingly high enough strain level.

(4)P REQ The design value of the compressive resistance of the pile, $R_{c;d}$ shall be derived from

$$R_{c;d} = R_{c;k} / \gamma_t \quad (7.10)$$

where the symbols are defined in 1.8 and

$$R_{c;k} = \text{Min} \left\{ \frac{(R_{c;m})_{\text{mean}}}{\xi_5}, \frac{(R_{c;m})_{\text{min}}}{\xi_6} \right\} \quad (7.11)$$

where ξ_5 and ξ_6 are correlation factors related to the number of piles tested, n , and are applied to the mean $(R_{c;m})_{\text{mean}}$ and the lowest $(R_{c;m})_{\text{min}}$ [result] value of $R_{c;m}$ respectively.

(5) ~~P REQ~~ REC Values of γ_i and of ξ_5 and ξ_6 to be used in equations 7.10 and 7.11 should be [taken]-selected [in accordance with 2.4.7.1(2)P] from Annex A.

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

7.6.2.5 Ultimate compressive resistance by applying pile driving formulae

(1)P REQ Pile driving formulae shall only be used if the stratification of the ground has been determined.

(2)P REQ If pile driving formulae are used to assess the ultimate compressive resistance of individual piles in a foundation, the validity of the formulae shall have been demonstrated by previous experimental evidence of acceptable performance *in* static load tests on the same type of pile, of similar length and cross section, and in similar ground conditions.

(3)P REQ For end bearing piles driven into cohesionless soil, the design value of the compressive resistance, $R_{c;d}$, shall be assessed by the same procedure as in 7.6.2.4.

(4) REC When a pile driving formula is applied to verify the compression resistance of a pile, the pile driving test should have been carried out on at least 5 piles distributed at sufficient spacing in the piling area in order to check a suitable blow count for the final series of blows. The penetration of the pile point for the final series of blows should be recorded for each pile.

7.6.2.6 Ultimate compressive resistance from wave equation analysis

(1)P REQ Wave equation analysis shall normally only be used where stratification of the ground has been determined by borings and field tests.

(2)P REQ Where wave equation analysis is used to assess the resistance of individual compression piles, the validity of the analysis shall have been demonstrated by previous evidence of acceptable performance *in* static load tests on the same pile type of similar length and cross section and in similar ground conditions.

(3)P REQ The design value of the compressive resistance, $R_{c;d}$, derived from the results of wave equation analysis of a number of representative piles shall be assessed by the same procedure as in 7.6.2.4.

~~[(4) REC]~~ *NOTE:* Wave equation analysis is based on a mathematical model of soil, pile and driving equipment without stress wave measurements on site. The method is usually applied to study hammer performance, dynamic soil parameters and stresses in the pile during driving. It

is also, on the basis of the models, possible to determine the required driving resistance (blow count) that is usually related to the expected compressive resistance of the pile.

7.6.2.7 Redriving

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

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

~~(3) REC~~ NOTE: Redriving of friction piles in clayey soils normally results in reduced compressive resistance.

7.6.3 Ground tensile resistance

7.6.3.1 General

(1)P REQ The design of piles in tension shall be consistent with the design rules given in 7.6.2, where applicable. Design rules that are specific for foundations involving piles in tension are presented *below*.

(2)P REQ To verify that the foundation will support the design load with adequate safety against a failure in tension, the following inequality shall be satisfied for all ultimate limit state load cases and load combinations:

$$F_{t,d} \leq R_{t,d} \quad (7.12)$$

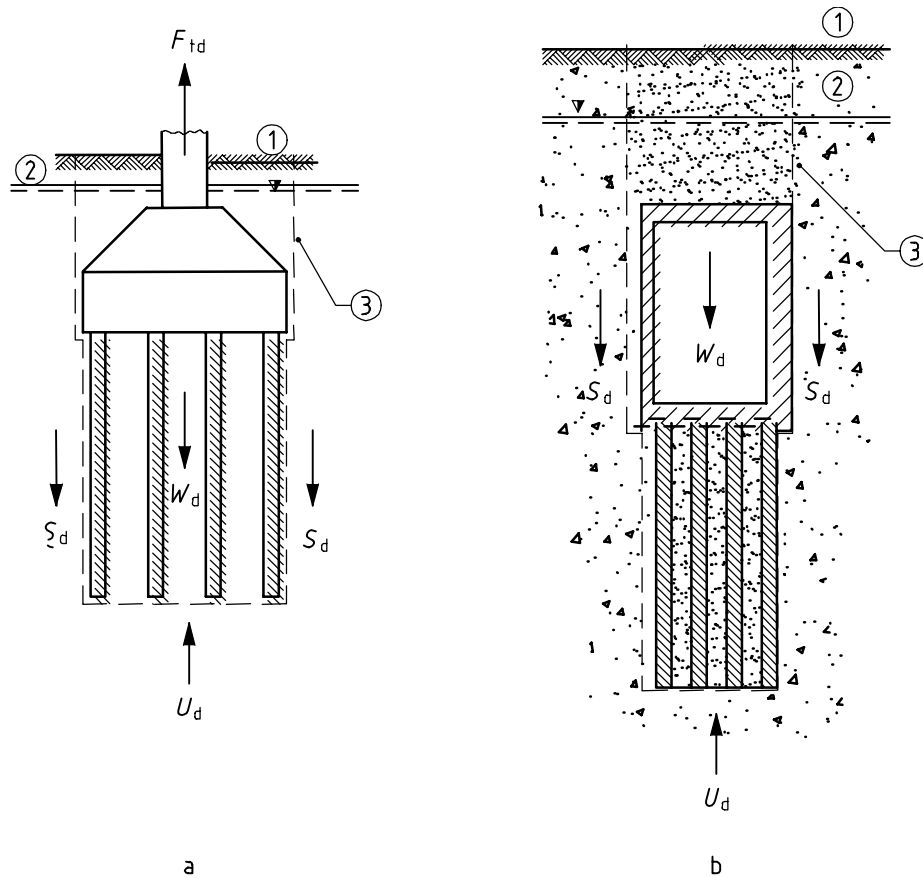
where the symbols are defined in 1.8.

(3)P REQ For tension piles, two failure mechanisms shall be considered:

- pull-out of the piles from the ground mass;
- uplift of the block of ground containing the piles.

(4)P REQ Verification against uplift failure of the block of ground containing the piles, (see fig.7.1), shall be carried out in accordance with 2.4.7.4.

(5) REC For isolated tensile piles or a group of tensile piles, the failure mechanism may be governed by the pull-out resistance of a cone of ground, especially for piles with an enlarged base or rock socket.



- 1 ground surface
 2 groundwater level
 3 side of the 'block', where resistance $[R]T$ develops
 PT NOTE: correction to figure: W_d must be G_d ; S_d must be T_d .

Fig. 7.1 Examples of uplift (UPL) of a group of piles

- (6) PER When considering the uplift of the block of ground containing the piles the shear resistance $[R]T_d$ along the sides of the block may be added to the resisting forces shown in fig. 7.1
- (7) REC Normally the block effect will govern the design tensile resistance if the distance between the piles is equal to or less than the square root of the product of the pile diameter and the pile penetration into the main resisting stratum.
- (8)P REQ The group effect, which may reduce the effective vertical stresses in the soil and hence the shaft resistances of individual piles in the group, shall be considered when assessing the tensile resistance of a group of piles.
- (9)P REQ The severe adverse effect of cyclic loading and reversals of load on the tensile resistance shall be considered.
- (10) REC Comparable experience based on pile load tests should be applied to appraise this effect.

7.6.3.2 Ultimate tensile resistance from pile load tests

(1)P REQ Pile load tests to determine the ultimate tensile resistance of an isolated pile, R_t , shall be carried out in accordance with 7.5.1 and 7.5.2 and with regard to 7.6.2.2.

(2)P REQ The design tensile resistance, $R_{t,d}$, shall be derived from

$$R_{t,d} = R_{t,k} / \gamma_{s,t} \quad (7.13)$$

where the symbols are defined in 1.8..

(3)P-REQ REC Values of $\gamma_{s,t}$ to be used in equation (7.13) should be ~~[taken]~~ *selected* ~~[in accordance with 2.4.7.1(2)P]~~ from Annex A .

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

(4) REC Normally when piles are to be loaded in tension, it should be specified that more than one pile should be tested. In the case of a large number of tension piles, at least 2 % should be tested.

(5) REC The records of the installation of the test pile(s) should be checked and any deviation from the normal construction conditions should be accounted for in the interpretation of the pile load test results.

(6)P REQ The characteristic value of the pile tensile resistance shall be determined by

$$R_{t,k} = \text{Min} \left\{ \frac{(R_{t,m})_{\text{mean}}}{\xi_1}, \frac{(R_{t,m})_{\text{min}}}{\xi_2} \right\} \quad (7.14)$$

where ξ_1 and ξ_2 are correlation factors related to the number of piles tested, n , and are applied respectively to the mean $(R_{t,m})_{\text{mean}}$ and the lowest $(R_{t,m})_{\text{min}}$ value of the measured tensile resistances.

(7)P-REQ REC Values of ξ_1 and ξ_2 to be used in equation (7.14) should be ~~[taken]~~ *selected* ~~[in accordance with 2.4.7.1(2)P]~~ from Annex A .

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

7.6.3.3 Ultimate tensile resistance from ~~[semi-empirical methods]~~ **ground test results**

(1)P REQ ~~[The semi-empirical m]Methods [used to assess]~~ *for assessing* the tensile resistance of a pile foundation *from ground test results* shall have been established from pile load tests and from comparable experience as defined in 1.5.2(2).

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

(3)P REQ The design value of tensile resistance of a pile, $R_{t,d}$, shall be derived from

$$R_{t,d} = R_{t,k} / \gamma_{s,t} \quad (7.15)$$

where:

$$R_{t;k} = R_{s;k} \quad (7.16)$$

and the symbols are defined in 1.8.

(4) ~~P-REQ~~ REC Values of $\gamma_{s,t}$ to be used in equation (7.15) should be ~~[taken]~~selected [in accordance with 2.4.7.1(2)] ~~P~~ from Annex A .

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

(5) ~~P~~ REQ The characteristic value $R_{t;k}$ shall be determined by

$$R_{t;k} = \text{Min} \left\{ \frac{(R_{s;cal})_{\text{mean}}}{\xi_3}, \frac{(R_{s;cal})_{\text{min}}}{\xi_4} \right\} \quad (7.17)$$

where ξ_3 and ξ_4 are correlation factors that depend on the number of profiles of tests, n , and are applied respectively to the mean $(R_{s;cal})_{\text{mean}}$ and to the lowest value $(R_{s;cal})_{\text{min}}$ of $R_{s;cal}$.

(6) ~~P-REQ~~ REC Values of ξ_3 and ξ_4 to be used in equation (7.17) should be ~~[taken]~~selected [in accordance with 2.4.7.1(2)] ~~P~~ from Annex A .

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

(7) ~~P~~ REQ The systematic and random components of the variation in the ground shall be recognized in the interpretation of the calculated tensile resistance.

(8) ~~PER~~ As an alternative to (5), the characteristic value of tensile resistance may be obtained by calculating

$$R_{t;k} = \sum_i A_{s;i} \cdot q_{s;k;i} \quad (7.18)$$

where $q_{s;k;i}$ are characteristic values of shaft friction *in the various strata obtained* ~~[determined]~~ from values of ground properties.

NOTE : If this alternative procedure is applied, the value of the partial factor $\gamma_{s,t}$ selected from Annex A, according to 2.4.7.1(2) may need to be corrected by a model factor larger than 1,0. The value of the partial factor may be set by the National annex.

(9) ~~P~~ REQ If Design Approach 3 is used, the characteristic values of ground parameters shall be determined according to 2.4.5; partial factors shall then be applied to these characteristic values to obtain design values of the ground parameters to calculate the design values of the pile resistance.

(10) ~~REC~~ The assessment of the validity of a model *based on ground test results* should be in accordance with 7.6.2.3(12).

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

7.6.4.1 General

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

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

(3) REC For piles bearing in medium to dense soils and for tension piles, the safety requirements for the ultimate limit state design ~~are~~ *should* normally *be assessed* ~~sufficient~~ to prevent *exceedence of* an ultimate limit state in the supported structure.

7.6.4.2 Pile foundations in compression

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

~~(2) REC~~ *NOTE:* When the pile toe is placed in a medium dense or firm layer overlying rock or very hard soil, the partial safety factors for ultimate limit state conditions are normally sufficient to satisfy serviceability limit state conditions.

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

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

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

7.6.4.3 Pile foundations in tension

(1)P REQ The assessment of upward displacements shall be in accordance with the principles of 7.6.4.2.

NOTE: Particular attention should be paid to the elongation of the pile material.

(2)P REQ When very severe criteria are set for the serviceability limit, a separate check of the upward displacements shall be carried out.

7.7 Transversely loaded piles

7.7.1 General

(1)P REQ The design of piles subjected to transverse loading shall be consistent with the design rules given in 7.4 and 7.5, where applicable. Design rules specifically for foundations involving piles subjected to transverse loading are presented ~~in this section~~ *below*.

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

$$F_{tr,d} \leq R_{tr,d} \quad (7.19)$$

where the symbols are defined in 1.8.

(3) REC One of the following failure mechanisms should be considered:

- for short piles, rotation or translation as a rigid body;
- for long slender piles, bending failure of the pile, accompanied by local yielding and displacement of the soil near the top of the pile.

(4)P REQ The group effect shall be considered when assessing the resistance of transversely loaded piles.

(5) REC *It should be considered that a transverse load applied to a group of piles may result in a combination of compression, tension and transverse forces in the individual piles.*

7.7.2 Transverse load resistance from pile load tests

(1)P REQ Transverse pile load tests shall be carried out in accordance with 7.5.2.

(2) *PER* Contrary to the load test procedure described in 7.5, *tests on transversely loaded piles need not [it is] normally [unnecessary] be to continued tests on transversely loaded piles to a state of failure.* The magnitude and line of action of the test load should simulate the design loading of the pile.

(3)P REQ An allowance shall be made for the variability of the ground, particularly over the top few metres of the pile, when choosing the number of piles for testing and when deriving the design transverse resistance from load test results.

(4) REC Records of the installation of the test pile(s) should be checked, and any deviation from the normal construction conditions should be accounted for in the interpretation of the pile load test results. For pile groups, the effects of interaction and head fixity should be accounted for when deriving the transverse resistance from the results of load tests on individual test piles.

7.7.3 Transverse load resistance from ground test results and pile strength parameters

(1)P REQ The transverse resistance of a pile or pile group shall be calculated using a compatible set of structural effects of actions, ground reactions and displacements.

(2)P REQ The analysis of a transversely loaded pile shall include the possibility of structural failure of the pile in the ground, in accordance with 7.8.

(3) PER The calculation of the transverse resistance of a long slender pile may be carried out using the theory of a beam loaded at the top and supported by a deformable medium characterized by a horizontal modulus of subgrade reaction.

(4)P REQ The degree of freedom of rotation of the piles at the connection with the structure shall be taken into account when assessing [its] *the foundation's* transverse resistance.

7.7.4 Transverse displacement

(1)P REQ The assessment of the transverse displacement of a pile foundation shall take into account:

- the stiffness of the ground and its variation with strain level;
- the flexural stiffness of the individual piles;
- the moment fixity of the piles at the connection with the structure;
- the group effect;
- the effect of load reversals or of cyclic loading.

(2) REC A general analysis of the displacement of a pile foundation should be based on expected degrees of kinematic freedom of movement.

7.8 Structural design of piles

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

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

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

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

(4)P REQ Slender piles passing through water or thick deposits of very weak soil shall be checked against buckling.

(5) PER Normally [~~it is not necessary to~~] a check for buckling *is not required* when the piles are contained by soils with a representative, undrained shear strength, c_u , that exceeds 10 kPa.

7.9 Supervision of construction

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

(2) REC The plan should give the following design information:

- the pile type;
- the location and inclination of each pile, including tolerances on position;
- pile cross section;
- for cast in situ piles, data about the reinforcement;
- pile length;
- pile number;
- required pile load carrying capacity;
- pile toe level (with respect to a fixed datum within or near the site), or the required penetration resistance;
- installation sequence;
- known obstructions;
- any other constraints on piling activities.

(3)P REQ It shall be specified that the the installation of all piles ~~[shall be]~~ is monitored and records ~~to be signed by the supervisor of the work and the piling contractor [shall be]~~ are made ~~at the site and~~ as the piles are installed.

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

- pile number;
- installation equipment;
- pile cross section and length;
- date and time of installation (including interruptions to the installation process);
- concrete mix, volume of concrete used and method of placing for cast-in-situ piles;
- weight density, pH, Marsh viscosity and fines content of bentonite slurry (when used);
- for continuous flight auger piles or other injection piles, *volumes and* pumping pressures of the grout or concrete, internal and external diameters, pitch of screw and penetration per revolution;
- for displacement piles, the values of driving resistance measurements such as weight and drop or power rating of hammer, blow frequency and number of blows for at least the last 0.25 m penetration;
- the power take-off of vibrators (where used);
- the torque applied to the drilling motor (where used);
- for bored piles, the strata encountered in the borings and the condition of the base if the performance of the ~~base~~ *pile toe* is critical;
- obstructions encountered during piling;
- deviation of position and directions and as-built elevations.

NOTE: EN 16199 on the execution of micropiles is in preparation

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

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

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

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

Section 8 Anchorages

8.1 General

8.1.1 Scope

(1)P REQ This section ~~is concerned~~ *shall apply to* with the design of temporary and permanent anchorages used:

- to support a retaining structure;
- to provide the stability of slopes, cuts, or tunnels;
- to resist uplift forces on structures

by transmitting a tensile force to a load bearing formation of soil or rock.

(2)P REQ This section is applicable to;

- pre-stressed anchorages consisting of an anchor head, a tendon free length and a fixed tendon length bonded to the ground by grout;
- non pre-stressed anchorages consisting of an anchor head, a free tendon length and a restraint such as a fixed anchor length bonded to the ground by grout, a deadman anchorage, a screw anchor or a rock bolt.

(3) REC This section ~~is should not be applied to~~ *applicable to* soil nails.

(4)P REQ Section 7 shall apply to the design of anchorages comprising tension piles.

8.1.2 Definitions

8.1.2.1 permanent anchorage

anchorage with a design life of more than two years

NOTE: definition taken from EN 1537

8.1.2.2 temporary anchorage

anchorage with a design life of less than two years

NOTE: definition taken from EN 1537

8.1.2.3 acceptance test

load test on site to confirm that each anchorage meets the design requirements,

8.1.2.4 suitability test

load test on site to confirm that ~~[the anchorage]~~ *a particular anchor design* will be adequate in ~~[the]~~ particular ground conditions

NOTE: definition taken from EN 1537

8.1.2.5 investigation test

load test to establish the ultimate resistance of an ~~grouted anchorage~~ at the grout/ground interface and to determine the characteristics of the anchorage in the working load range.

NOTE: definition taken from EN 1537

8.1.2.6 anchor bond length: length of the anchor that is bonded directly to the ground through a grout body. **(7) tendon free length:** the length of the tendon between the anchor head and the proximal end of the ~~grout body~~-tendon bond length.

NOTE: definition taken from EN 1537

8.1.2.8 tendon bond length: length of the tendon that is bonded directly to the grout and capable of transmitting the applied tensile load.

NOTE: definition taken from EN 1537

8.2 Limit states

(1)P REQ The following limit states shall be considered for anchorages, both individually and in combination:

- structural failure of the tendon or anchor head, caused by the applied stresses, distortion of the anchor head or corrosion;
- for grouted anchors, failure at the interface between the body of grout and the ground;
- failure of the bond between the steel tendon and the grout;
- for deadman anchorages, failure by insufficient resistance of the deadman;
- loss of anchorage force by excessive displacements of the anchor head or by creep and relaxation;
- failure or excessive deformation of parts of the structure due to the applied anchorage force;
- loss of overall stability of the retained ground and the retaining structure;
- interaction of groups of anchorages with the ground and adjoining structures.

8.3 Actions and design situations

(1)P REQ When selecting the design situations, consideration shall be given to:

- all circumstances during the construction and the design life of the structure;
- all pertinent limit states listed in 8.2, and their combinations;
- the level of the groundwater and the water pressures in confined aquifers;
- the consequences of the failure of any anchorage;
- the possibility that the forces applied to the anchorages during prestressing (anchorage load) may exceed the forces required for the design of the structure.

(2)P REQ The anchorage load, P , shall be treated as an unfavourable action for the anchorage design.

8.4 Design and construction considerations

(1)P REQ The design of the anchorage and the specification for its execution shall take into account any adverse effects of tensile stresses transmitted to ground beyond the vicinity of the anchorage.

(2)P REQ The zone of ground into which tensile forces are to be transferred shall be included in site investigations.

(3)P REQ For pre-stressed anchorages, the anchor head shall allow the tendon or rod to be stressed, proof-loaded and locked-off and, if required by the design, released, de-stressed and re-stressed.

(4)P REQ For all types of anchorages, the anchor head shall be designed to tolerate angular deviations of the anchor force, taking into account 6.3 of EN 1537:1999, and to be able to accommodate deformations which may occur during the design life of the structure.

(5)P REQ Where different materials are combined in an anchorage, their design strengths shall be assessed with due account of the compatibility of their deformation performance.

(6)P REQ Since the effect of anchorage systems depends on their free lengths, the following requirements shall be fulfilled:

- the anchor force shall act in ground that is sufficiently distant from the retained volume of ground that the stability of this volume is not adversely affected;
- the anchor force shall act in ground that is sufficiently distant from existing foundations to avoid any adverse effects on them;
- measures shall be taken to avoid adverse interactions between the fixed lengths of anchorages that pass close to each other;

(7) REC Adverse interactions between the bond lengths of anchorages should be avoided, if possible, by keeping a space not less than 1,5 m between their bond lengths.

(8)P REQ Only anchorage systems shall be used which have been tested by investigation tests (see EN 1537) or for which successful comparable experience is documented in terms of both performance and durability.

(9)P REQ The direction of the tendon shall normally be such as to provide self-stressing with deformations due to potential failure mechanisms. In case this is not feasible, adverse effects shall be taken into account in the design.

(10)P REQ For grouted anchorages and screw anchorages, the characteristic value of the pull-out resistance, $R_{a,k}$, shall be determined on the basis of suitability tests *according to 8.7* or comparable experience. The design resistance shall be checked by acceptance tests after execution.

(11)P REQ The performance of the *tendon* free length of pre-stressed ground anchorages shall be checked in accordance with EN 1537.

(12)P REQ A sufficient lock-off force shall be used to ensure that the anchorage resistance under serviceability limit state conditions will be mobilized with tolerable head displacements.

(13)P REQ Corrosion protection of pre-stressed anchorages shall comply with 6.9 of EN 1537:1999.

(14)P REQ Corrosion protection of anchorages having a steel tendon shall be designed taking into account the aggressiveness of the ground environment.

(15) REC Suitable means, such as the use of a protective sheath or the provision of sacrificial steel, should be specified if necessary to protect steel tendons against corrosion.

8.5 Ultimate limit state design

8.5.1 Design of the anchorage

(1)P REQ The design value $R_{a;d}$ of the pull-out resistance, R_a , of an anchorage shall fulfill the limit condition

$$R_{a;d} \geq P_d \quad (8.1)$$

where the symbols are defined in [8.0(1)P] 1.8.

(2) PER Design values of pull-out resistance may be determined by the results of tests on anchorages, or by calculations.

8.5.2 Design values of pull-out resistance determined by tests

(1)P REQ The design value of the pull-out resistance shall be derived from the characteristic value using the equation:

$$R_{a;d} = R_{a;k} / \gamma_a \quad (8.2)$$

where the symbols are defined in [8.0(1)P] 1.8.

~~(2) REC~~ NOTE: The partial factor, γ_a , takes into account unfavourable deviations of the anchorage force transmitted into the ground, either in terms of fixed length or of a dead-man structure.

~~(2)P-REQ REC~~ Values of γ_a to be used in equation (8.2) should be selected [in accordance with 2.4.7.1(2)P] from Annex A.

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

~~[(4)P-REQ For grouted anchorages, the characteristic value $R_{a;k}$ determined on the basis of suitability tests and comparable experience shall be used when ???????].~~

[PT note: deleted because the same message is given in 8.4(10)]

(3) REC The characteristic values ~~may~~ should be related to the suitability test results by applying a correlation factor ξ_a .

NOTE: 8.5.2(3) refers to those types of anchorage that are not individually checked by acceptance tests. For the value of the correlation factor ξ_a see the National annex

8.5.3 Design values of pull-out resistance determined by calculations

(1)P REQ The design value of pull-out resistance of anchorages shall be assessed according to the principles in 2.4.7 and 2.4.8 where appropriate.

8.5.4 Design value of the structural resistance of the anchorage

(1)P REQ The structural design of the anchorage shall satisfy the following inequality:

$$R_{t,d} \geq R_{a,d} \quad (8.3)$$

(2)P REQ The material resistance of the anchorages, $R_{t,d}$, shall be calculated according to EN 1992, EN 1993 and EN 1537:1999, as relevant.

(3)P REQ If anchors are submitted to suitability tests, $R_{t,d}$ shall take account of the proof load, see 9.5 of EN 1537:1999.

8.5.5 Design value of the anchorage load

(1)P REQ The design value required by the anchorage load, P_d , shall be derived from the design of the retained structure as the maximum value of

- the ultimate limit state force of the retained structure, and if relevant
- the serviceability limit state force of the retained structure.

(2) REC A model factor ~~may~~ *should* be applied on the SLS force to ensure that the resistance of the anchorage is sufficiently safe.

NOTE: The value of the model factor may be set in the National annex.

8.6 Serviceability limit state design

(1)P REQ For the verification of a serviceability limit state of the supported structure, an anchorage shall be regarded as a spring.

(2)P REQ For pre-stressed anchorages (e.g. grouted anchorages), the spring shall be regarded as an elastic, pre-stressed spring.

(3) REC The most adverse combination of the minimum or maximum anchorage stiffness and minimum or maximum pre-stress should be selected when analysing the design situation indicated in 8.6(2)P.

(4) REC *When* considering a non-pre-stressed anchorage as a (non-pre-stressed) spring, its stiffness should be selected to achieve compatibility between calculated displacements of the retained structure and the displacement and elongation of the anchorage.

(5) REC Account should be taken of the effects of any deformations imposed on adjacent foundations by the anchorage pre-stress force.

8.7 Suitability tests

(1)P REQ Suitability tests shall be specified for grouted anchorages, screw anchorages and rock bolts. The performance of the test shall comply with EN 1537:1999.

(2) REC At least three suitability tests should be performed for each distinct condition of ground and structure to determine the characteristic resistance of the anchor.

(3)P REQ The proof load, P_p , of a suitability test of grouted anchorages shall comply with EN 1537:1999.

(4) REC Until a specific test is available, a suitability test for screw anchorages and rock bolts should follow the same procedure as indicated in EN 1537:1999 for grouted anchorages.

8.8 Acceptance tests

(1)P REQ It shall be specified in the design that all grouted anchorages shall be subjected to acceptance tests prior to lock-off and before they become operational.

(2)P REQ The procedure for acceptance tests shall follow the rules given in EN 1537:1999 for grouted anchorages.

(3) REC Where groups of anchorages are crossing with fixed lengths at spacings of less than 1,5 m, random control tests should be made after completion of the lock-off action.

8.9 Supervision and monitoring

(1)P REQ Supervision and monitoring shall follow the rules given in section 4 of this standard and 9.10 and 9.11 of EN 1537:1999, where appropriate.

Section 9 Retaining structures

9.1 General

9.1.1 Scope

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

(2)P REQ REC Pressure from granular material stored in silos ~~should~~ *shall* be calculated using EN 1991-4 - Actions in silos and tanks.

9.1.2 Definitions

(1) REC In considering the design of retaining structures ~~it may be appropriate to distinguish between~~ the following three main types *should be distinguished, if appropriate*.

9.1.2.1

gravity walls:

walls of stone or plain or reinforced concrete having a base footing with or without a heel, ledge or buttress. The ~~[gravity]~~ *weight* of the wall itself, sometimes including stabilizing masses of soil, rock or backfill, plays a significant role in the support of the retained material. Examples of such walls include concrete gravity walls having constant or variable thickness, spread footing reinforced concrete walls, buttress walls, etc.;

9.1.2.2

embedded walls:

relatively thin walls of steel, reinforced concrete or timber, supported by anchorages, struts and/or passive earth pressure. The bending capacity of such walls plays a significant role in the support of the retained material while the role of the weight of the wall is insignificant. Examples of such walls include: cantilever steel sheet pile walls, anchored or strutted steel or concrete sheet pile walls, diaphragm walls, etc;

9.1.2.3

composite retaining structures:

walls composed of elements from the above two types of wall. A large variety of such walls exists and examples include double sheet pile wall cofferdams, earth structures reinforced by tendons, geotextiles or grouting, structures with multiple rows of ground anchorages or soil nails, etc.

9.2 Limit states

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

- loss of overall stability;

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

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

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

and for embedded walls:

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

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

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

9.3 Actions, geometrical data and design situations

9.3.1 Actions

9.3.1.1 Basic actions

(1)P REQ The actions listed in 2.4.2.(4)P shall be considered.

9.3.1.2 Weight of backfill material

(1)P REQ Design values for the weight density of backfill material shall be estimated from knowledge of available material. The Geotechnical Design Report shall specify the checks which shall be made during the construction process to verify that the actual field values are no worse than those ~~[assumed]~~ *used* in the design.

9.3.1.3 Surcharges

(1)P REQ Determination of design values for surcharges shall take account of the presence, on or near the surface of the retained ground, of nearby buildings, parked or moving vehicles or cranes, stored material, goods, containers, etc.

(2) REC Care ~~is needed~~ *should be taken* in the case of repeated surcharge loading such as imposed by crane rails on a quay wall. The pressures induced by such surcharges ~~may~~ *can* significantly exceed those due to the first loading or those resulting from static application of a load of equal magnitude.

9.3.1.4 Weight of water

(1)P REQ Design values for the weight density of water shall reflect whether the water is fresh, saline or charged with chemicals or contaminants to an extent that the normal value needs amendment.

9.3.1.5 Wave and ice forces

(1)P REQ Design values for forces imposed by reflected waves or by ice forces shall be selected on the basis of locally available data for the climatic and hydraulic conditions at the site.

(2)P REQ ~~The~~ *When selecting* design values for static forces imposed by a sheet of ice ~~the following shall be~~ taken into account:

- the initial temperature of the ice before warming begins;
- the rate at which the temperature increases;
- the thickness of the ice sheet.

9.3.1.6 Seepage forces

(1)P REQ Seepage forces due to different groundwater levels behind and in front of a retaining structure shall be considered as they may change the earth pressure behind the wall and reduce the earth resistance in front of the wall.

~~9.3.1.6 Strut and anchorage forces~~

~~(1)P REQ Forces from struts and anchorages shall be regarded as actions. Design values shall be selected taking into account the effects of overstressing the supporting elements. For anchorages see Section 8]~~

9.3.1.7 Collision forces

(1) [REQ]PER The determination of design values for collision impact forces, caused by waves, ice floe, traffic etc., ~~shall~~ *may* take account of the energy absorbed *by the colliding mass* and by the retaining system, *e.g. by fendering and/or guide structures*.

(2) REC For lateral impacts on retaining walls, ~~it is normally necessary to consider~~ the increased stiffness exhibited by the retained ground *should be considered*.

(3) REC The risk of the occurrence of liquefaction due to lateral impact on embedded walls should be investigated.

(4)P REQ The impact load of an ice floe colliding with a retaining structure shall be calculated on the basis of the compressive strength of the ice and the thickness of the ice floe. The salinity and homogeneity of the ice shall be considered in calculating the compressive strength.

9.3.1.8 Temperature effects

- (1)P REQ The design of retaining structures shall take into account the temporal and spatial effects of abnormal temperature changes.
- (2) REC These effects should be considered particularly when determining the loads in struts and props.
- (3) REC The Structural Fire Design Parts of the material related Eurocodes should be consulted when dealing with the effects of fire.
- (4)P REQ Special precautions, such as selection of suitable backfill material, drainage or insulation, shall be taken to prevent ice lenses forming in the ground behind retaining structures.

9.3.2 Geometrical data

9.3.2.1 ~~General~~Basic data

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

9.3.2.2 Ground surfaces

- (1)P REQ Design values for the geometry of the retained material shall take account of the variation in the actual field values. The design values shall also take account of anticipated excavation or possible scour in front of the retaining structure.
- (2) REC In ultimate limit state calculations in which the stability of a retaining wall depends on the ground resistance in front of the structure, the level of the resisting soil should be lowered below the *nominally* expected level by an amount Δa . The value of Δa should be selected taking into account the degree of site control over the level of the surface. With a normal degree of control, the following should be applied:
- for a cantilever wall, Δa should equal 10% of the wall height above excavation level, limited to a maximum of 0,5 m;
 - for a supported wall, Δa should equal 10% of the distance between the lowest support and the excavation level, limited to a maximum of 0,5 m.
- (3) PER Smaller values of Δa , including 0, may be used when the surface level is specified to be controlled reliably throughout the appropriate execution period.
- (4) REC Larger values of Δa should be used where the surface level is particularly uncertain.

9.3.2.3 Water levels

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

(2)P REQ Account shall be taken of the effects of variation in permeability on the groundwater regime.

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

9.3.3 Design situations

(1)P REQ The following items shall be considered:

- variation in soil properties in time and space;
- variation in water levels and pore water pressures in time and space;
- variation in actions and in the ways they are combined;
- excavation, scour or erosion in front of the retaining structure;
- compaction effects of backfilling behind the retaining structure;
- the effects of any future structures and surcharge loadings or unloadings on or close to the retained material;
- ground movements due to subsidence, frost actions, etc.

(2) ~~REC~~ PER For waterfront structures ice forces and wave forces need not be applied simultaneously at the same point.

9.4 Design and construction considerations

9.4.1 General

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

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

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

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

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

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

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

(8)P REQ The design of retaining structures shall take account of the following items where appropriate:

- the effects of constructing the wall, including:
 - the provision of temporary support to the sides of excavations;
the changes of in situ stresses and resulting ground movements caused both by the wall installation excavation and its construction;
 - disturbance of the ground due to driving or boring operations;
 - provision of access for construction;
- the required degree of watertightness of the finished wall;
- the practicability of constructing the wall to reach a stratum of low permeability, so forming a water cut-off. The resulting equilibrium groundwater flow problem shall be assessed;
- the practicability of forming ground anchorages in adjacent ground;
- the practicability of excavating between any propping of retaining walls;
- the ability of the wall to carry vertical load;
- the ductility of structural components;
- access for maintenance of the wall and any associated drainage measures;
- the appearance and durability of the wall and any anchorages;
- for sheet piling, the need for a section stiff enough to be driven to the design penetration without loss of interlock;
- the stability of borings or slurry trench panels while they are open;
- for fill, the nature of materials available and the means used to compact them adjacent to the wall, in accordance with 5.3.

9.4.2 Drainage systems

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

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

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

9.5 Determination of earth pressures

9.5.1 General

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

(2) REC In the following context the word "earth pressure" ~~will~~ *should* also be used for denote the total earth pressure from soft and weathered rocks and *should* include the pressure of groundwater.

(3)P REQ Calculations of the magnitudes of earth pressures and directions of forces resulting from them shall take account of:

- the surcharge on and slope of the ground surface;
- the inclination of the wall to the vertical;
- the water tables and the seepage forces in the ground;
- the amount and direction of the movement of the wall relative to the ground;
- the horizontal as well as vertical equilibrium for the entire retaining structure;
- the shear strength and weight density of the ground;
- the rigidity of the wall and the supporting system;
- the wall roughness.

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

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

(5) REC The amount of shear stress which can be mobilised at the wall-ground interface *should be determined by the wall-ground interface parameter δ [and a]. [For a completely smooth wall $\delta = 0$ and for a completely rough wall values of $\delta = \varphi'$ and $a = c'$ are appropriate].*

(6) ~~REC~~PER A concrete wall or steel sheet pile wall supporting sand or gravel may be assumed to have a wall ground interface parameter ~~[$\delta = k \cdot \varphi'$ and $a = 0$]~~ $\delta_d = k \cdot \varphi_{cv;d}$. k should not exceed 2/3 for precast concrete or steel sheet piling.

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

(8) REC For a steel sheet pile in clay under undrained conditions immediately after driving, values of $\delta = 0$ ~~[and $a = 0$]~~ should be assumed. Increases in these values may take place over a period of time.

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

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

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

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

9.5.2 At rest values of earth pressure

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

(2) REC For normally consolidated soil, at rest conditions ~~will-should~~ normally be assumed exist in the ground behind a retaining structure ~~when-if~~ the movement of the structure is less than $5 \times 10^{-4} \times h$.

(3) ~~REC~~ REC For a horizontal ground surface, the at rest earth pressure coefficient, K_0 , [~~which expresses the ratio between the horizontal and vertical (i.e. overburden) effective stresses~~] ~~may~~ should be determined by:

$$K_0 = (1 - \sin\phi') \times \sqrt{OCR} \quad (9.1)$$

~~with the symbols defined in 1.8 [where R_{oc} denotes the over-consolidation ratio being the ratio of the maximum effective vertical stress experienced in the past to the present effective vertical stress in the soil].~~ The formula should not be used for very high values of OCR.

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

$$K_{0,\beta} = K_0 \cdot (1 + \sin\beta) \quad (9.2)$$

with the symbols defined in 1.8

The direction of the earth pressure force ~~may-should~~ then be assumed to be parallel to the ground surface.

9.5.3 Limiting values of earth pressure

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

(2) ~~REC~~-POS Limiting values of earth pressure assuming straight failure surfaces ~~may-can~~ significantly deviate from the values assuming curved failure surfaces for high angles of internal friction and wall-ground interface parameters δ and so lead to unsafe results.

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

(3) REC ~~When~~-In cases where struts, anchorages or similar elements impose restraints on movement of the retaining structure, *it should be considered that* the limiting active and passive

values of earth pressure, and their distributions, ~~are may not necessarily be~~ the most adverse ones.

9.5.4 Intermediate values of earth pressure

(1)P REQ Intermediate values of earth pressure occur ~~when-if~~ the wall movements are insufficient to mobilize the limiting values. The determination of the intermediate values of earth pressure shall take account of the amount of wall movement and its direction relative to the ground [~~(see Annex C, fig. C.3)~~].

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

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

9.5.5 Compaction effects

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

~~[(2) REC]~~NOTE: Measurements indicate that the additional pressures depend on the applied compactive energy, the thickness of the compacted layers and the travel pattern of the compaction plant. ~~[Compaction]~~ Horizontal pressure normal to the wall in a layer may reduce when the next layer is placed and compacted. When backfilling is complete, the additional pressure normally acts only on the upper part of the wall.

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

9.6 Water pressures

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

(2)P REQ When checking the ultimate and serviceability limit states, water pressures shall be accounted for in the combinations of actions in accordance with 2.4.5.3 and 2.4.6.1, considering the possible risks indicated in ~~[9.4(5)]~~9.4.1(5).

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

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

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

9.7 Ultimate limit state design

9.7.1 General

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

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

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

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

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

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

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

9.7.2 Overall stability

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

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

9.7.3 Foundation failure of gravity walls

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

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

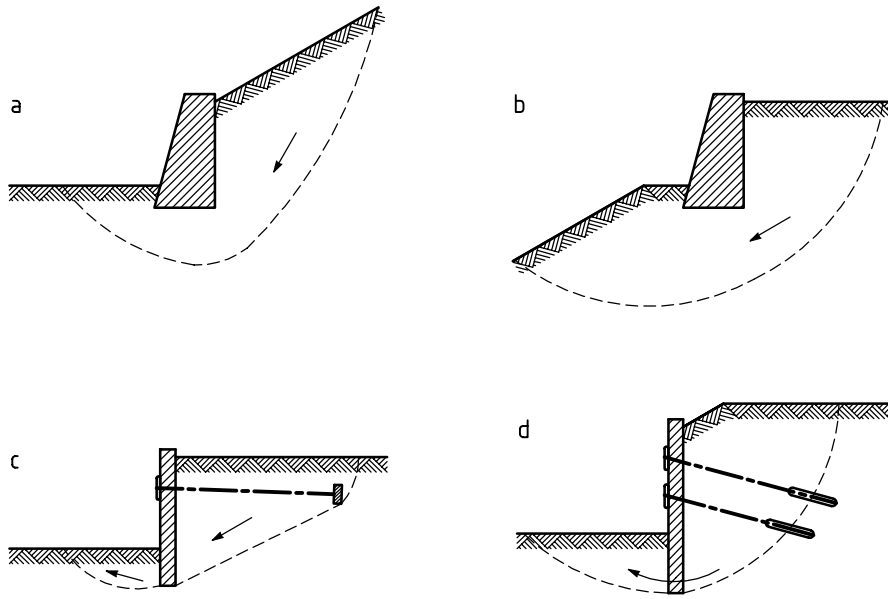


Figure 9.1 Examples of limit modes for overall stability of retaining structures.

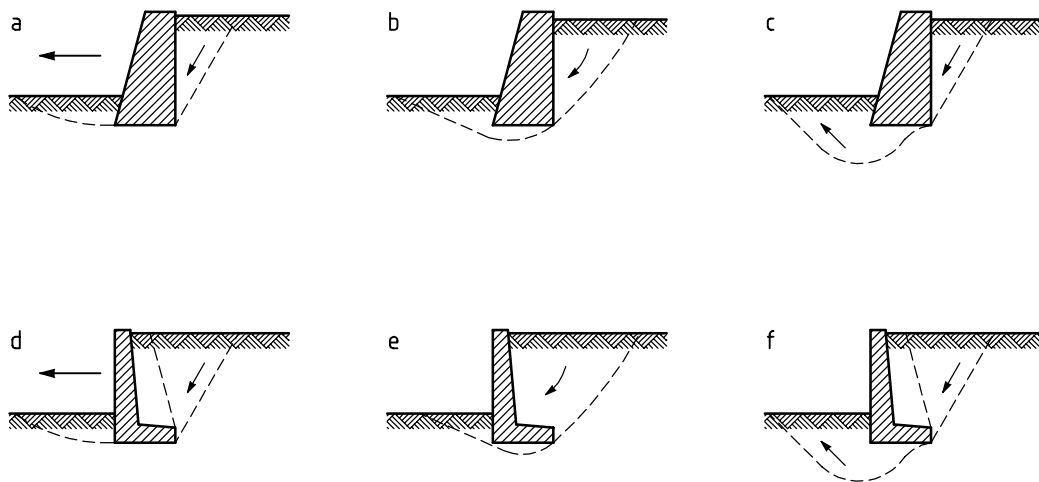


Figure 9.2 Examples of limit modes for foundation failures of gravity walls

9.7.4 Rotational failure of embedded walls

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

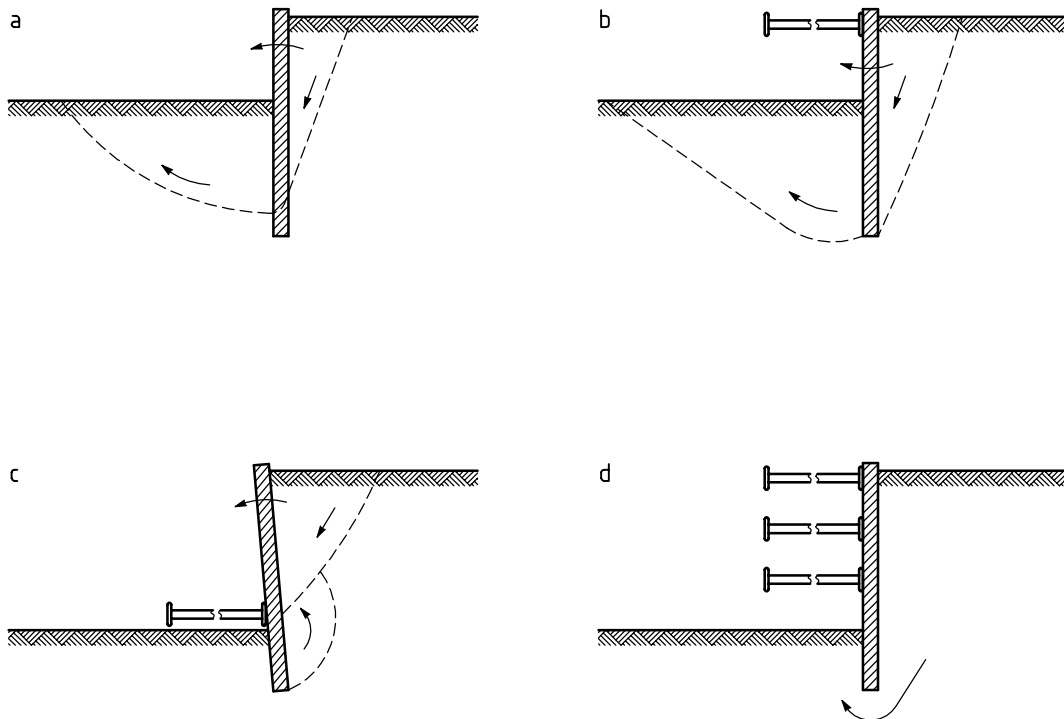


Figure 9.3 Examples of limit modes for rotational failures of embedded walls.

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

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

9.7.5 Vertical failure of embedded walls

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

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

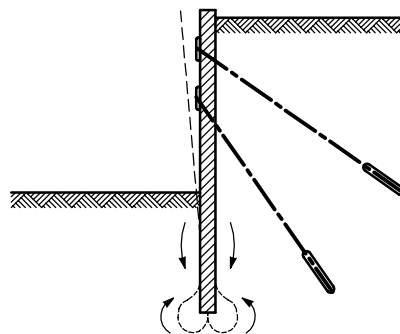


Fig. 9.4 Example of a limit mode for vertical failure of embedded walls

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

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

~~[(5) REC Vertical and rotational equilibrium should be checked with the same design values for shear stress on the wall.]~~

(5)P REQ If the wall acts as the foundation for a structure, vertical equilibrium shall be checked using the principles of section 6.

9.7.6 Structural design of retaining structures

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

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

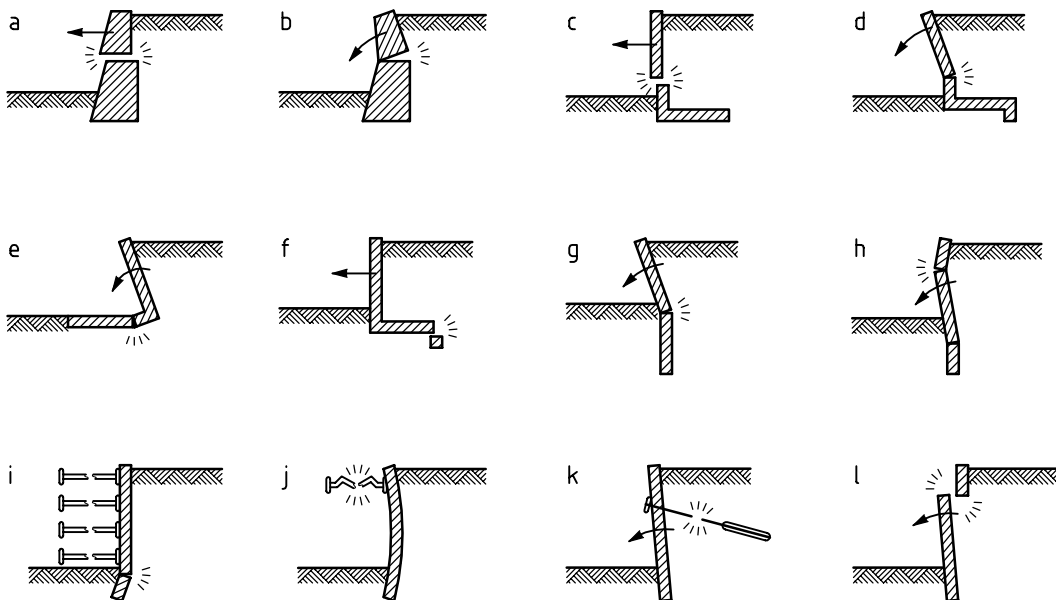


Fig. 9.5 Examples of limit modes for structural failure of retaining structures

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

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

9.7.7 Failure by pull-out of anchorages

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

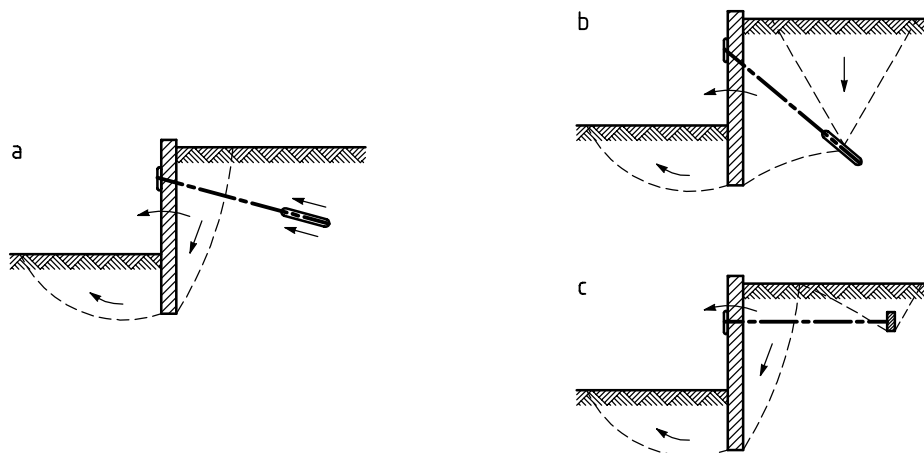


Figure 9.6 Examples of limit modes for failure by pull-out of anchors.

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

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

(4) REC For dead-man anchors, the failure mode illustrated in Figure 9.6 (c) should *also* be considered.

9.8 Serviceability limit state design

9.8.1 General

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

(2)P REQ Design values of earth pressures for the serviceability limit state shall be derived using characteristic values of all soil parameters.

(3)P REQ Permanent surcharge loads behind the retaining wall shall be taken at their characteristic values.

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

(5) REC The design values of earth pressures should be derived taking account of the allowable deformation of the structure at its serviceability limit state. These pressures ~~will~~ *may* not necessarily be limiting values.

9.8.2 Displacements

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

(2)P REQ A cautious estimate of the distortion and displacement of retaining walls, and the effects on supported structures and services, shall always be made on the basis of comparable

experience. This estimate shall include the effects of construction of the wall. The design may be justified by checking that the estimated displacements do not exceed the limiting values.

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

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

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

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

(6) REC Displacement calculations should *also* be considered ~~also~~ *in one of* the following cases:

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

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

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

(9)P REQ For the effect of vibrations on displacements see 6.6.4.

Section 10 Hydraulic failure

~~10.0~~ (moved to 1.8)

10.1 General

(1)P REQ The provisions of this section apply to four modes of ground failure induced by pore water pressure or pore water seepage, which shall be checked as relevant:

- failure by uplift (buoyancy).

NOTE: Buoyancy occurs when pore water pressure under a structure or a low permeability ground layer becomes larger than the mean overburden pressure (due to the structure and/or the overlying ground layer);

- failure by heave.

NOTE: Failure by heave occurs when upwards seepage forces act against the weight of the soil, reducing the vertical effective stress to zero. Soil particles are then lifted away by the vertical water flow and failure occurs (boiling);

- failure by internal erosion.

NOTE: Failure by internal erosion is produced by the transport of soil particles within a soil stratum, at the interface of soil strata, or at the interface between the soil and a structure. This may finally result in regressive erosion, leading to collapse of the soil structure;

- failure by piping.

NOTE: failure by piping is a particular form of failure, for example of a reservoir, by internal erosion, where erosion begins at the surface, then regresses until a pipe-shaped discharge tunnel is formed in the soil mass or between the soil and a foundation or at the interface between cohesive and non-cohesive soil strata. Failure occurs as soon as the upstream end of the eroded tunnel reaches the bottom of the reservoir.

NOTE: The conditions for hydraulic failure of the ground can be expressed in terms of total stress and pore water pressure or in terms of effective stresses and hydraulic gradient. Total stress analysis is applied to failure by uplift. For failure by heave, both total and effective stresses are applied. Conditions are put on hydraulic gradients in order to control internal erosion and piping.

(2) PER In situations where the pore water pressure is hydrostatic (negligible hydraulic gradient) it is not required to check other than failure by uplift.

(3)P REQ The determination of hydraulic gradients, pore pressures or seepage forces shall take account of:

- the variation of soil permeability in time and space;
- variations in water levels and pore water pressure in time;
- any modification of the boundary conditions (e.g. downstream excavation).

(4) REC It should be considered that the relevant soil stratification may be different for different failure mechanisms.

(5)P REQ When hydraulic heave, piping or internal erosion are a significant danger to the integrity of a geotechnical structure, measures shall be taken to decrease the hydraulic gradient.

(6) REC The measures most commonly ~~designed~~ adopted to reduce erosion or to avoid hydraulic failure should be:

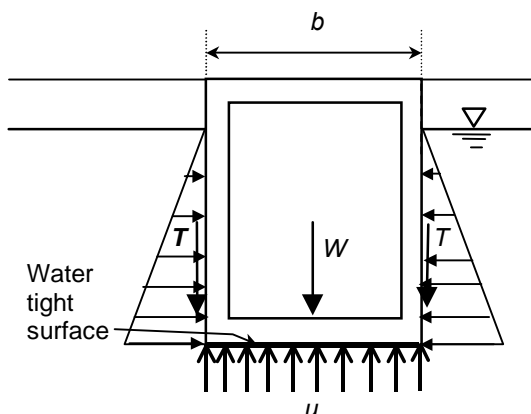
- lengthening the seepage path by screens or shoulders;
- modifications of the project in order to resist the pressures or gradients;
- seepage control;
- protective filters;
- avoidance of dispersive clays without adequate filters;
- slope revetments;
- inverted filters;
- relief wells;
- reduction of hydraulic gradient.

10.2 Failure by uplift

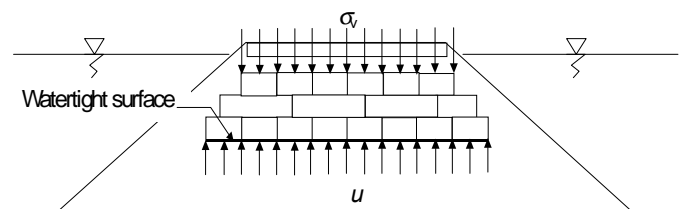
(1)P REQ The stability of a structure or of a low permeability ground layer against uplift shall be checked by comparing the permanent stabilising actions (weight, side friction, etc.) to the permanent and variable destabilising actions from water and, possibly, other sources. Examples of situations where uplift stability shall be checked are given in Figure 7.1 and Figure 10.1.

(2)P REQ The design shall be checked against failure by uplift using equation (2.8) of 2.4.7.4. In this equation, the design value of the vertical component of the stabilising permanent actions $G_{stb;d}$ is the sum of the weight W of the structure (or ground layer), any friction forces T , and any anchor forces P , etc. The design value of the vertical component of the destabilising permanent and variable actions $V_{dst;d}$ is the sum of the water pressures applied under the structure (permanent and variable parts) and any other upwards forces.

(3) PER In simple cases, the check of equation (2.8) in terms of forces can be replaced by a check in terms of total stresses and pore pressures.



a) Uplift of a buried hollow structure



b) Uplift of a lightweight embankment during flood

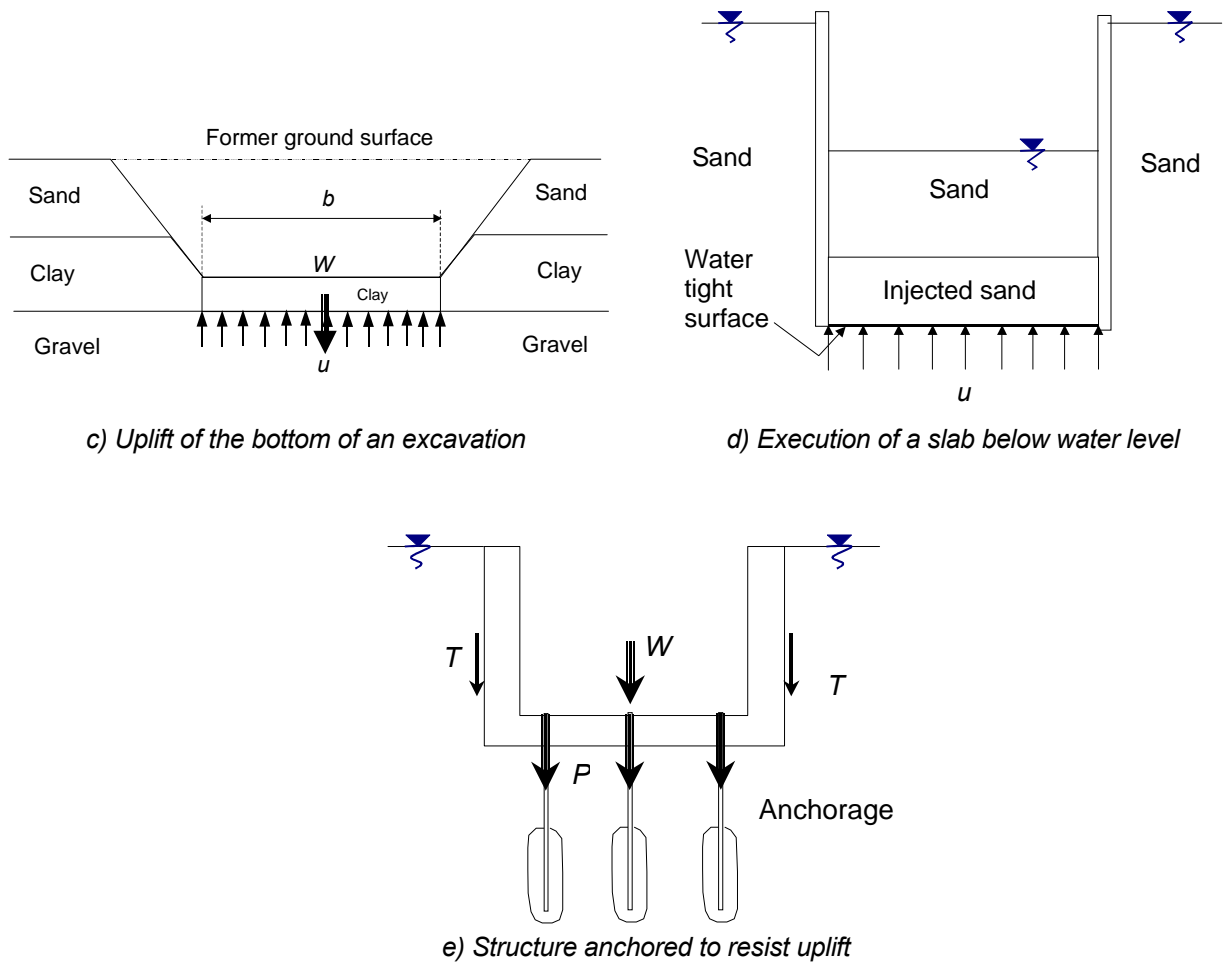


Figure 10.1 Examples of situations where uplift might be critical

(4) REC The measures most commonly ~~designed~~ adopted to resist failure by uplift should be:

- increasing the weight of the structure;
- decreasing the water pressure below the structure by drainage;
- anchoring the structure in the underlying strata.

(5)P REQ Where piles or anchors are used to provide resistance against failure by uplift, the design shall be checked according to 7.6.3 or 8.5, respectively, using the partial factors given in 2.4.7.4.

10.3 Failure by heave

(1)P REQ The stability of soil against heave shall be checked by verifying either equation (2.9a) or equation (2.9b) for every relevant soil column. Equation (2.9a) expresses the condition for stability in terms of total stresses and pore water pressures. Equation (2.9b) expresses the same condition in terms of effective stresses and excess pore water pressures. An example of situations where heave shall be checked is given in Figure 10.2.

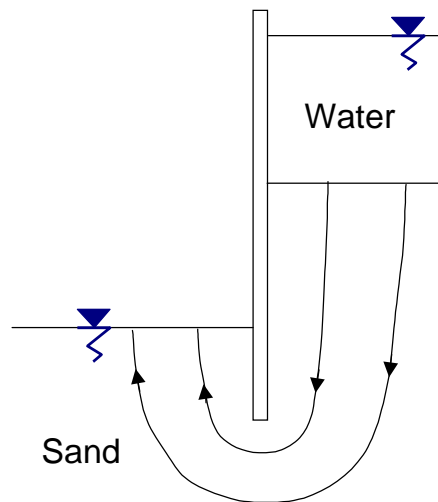


Figure 10.2 Example of situation where heave might be critical

(2)P REQ The determination of the characteristic value of the pore water pressure shall take into account all possible unfavourable conditions, such as:

- thin layers of soil of low permeability;
- spatial effects such as narrow, circular or rectangular excavations below water level.

NOTE 1: Where the soil has a significant cohesive shear resistance, the mode of failure changes from failure by heave to failure by uplift. The stability is then checked by using the provisions of paragraph 10.2 where additional resisting forces may be added to the weight.

NOTE 2: Stability against heave will not necessarily prevent internal erosion, which should be checked independently, when relevant.

(3) REC The measures most commonly adopted to resist failure by heave are:

- decreasing the water pressure below the soil mass subjected to failure;
- increasing the resisting weight .

10.4 Internal erosion

(1)P REQ Filter criteria shall be used to limit the danger of material transport by internal erosion.

(2)P REQ Where an ultimate limit state due to internal erosion can occur, measures such as filter protection shall be applied at the free surface of the ground.

(3) REC Filter protection should generally be provided by use of natural non-cohesive soil that fulfils adequate design criteria for filter materials. In some cases more than one filter layer may be necessary to ensure that the particle size distribution changes in a stepwise fashion to obtain sufficient protection both for the soil and the filter layers.

(4) PER Alternatively, artificial filter sheets such as geotextiles may be used provided it can be established that they sufficiently prevent transport of fines.

(5)P REQ If the filter criteria are not satisfied, it shall be verified that the critical hydraulic gradient is well below the design value of the gradient at which soil particles begin to move.

(6)P REQ The critical hydraulic gradient for internal erosion shall be established taking into consideration at least the following aspects:

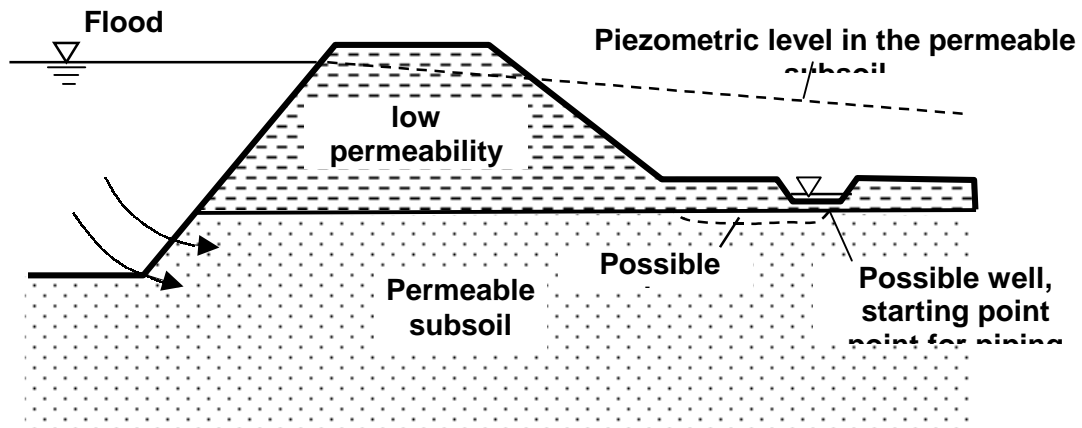
- direction of flow;
- grain size distribution and shape of grains;
- stratification of the soil.

10.5 Failure by piping

(1)P REQ Where prevailing hydraulic and soil conditions can lead to the occurrence of piping, see fig. 10.3, and where piping endangers the stability or serviceability of the hydraulic structure, prescriptive measures shall be taken to prevent the onset of the piping process either by the application of filters or by taking structural measures to control or to block the groundwater flow.

NOTE Suitable structural measures are:

- application of verges on the land side of a *retaining embankment dike*, thus displacing the possible starting point of piping farther away from the structure and decreasing the hydraulic gradient at this point;
- application of impermeable screens below the base of a hydraulic structure by which the groundwater flow is either blocked or the seepage path is increased, thereby decreasing the hydraulic gradient to a safe value.



(2)P REQ During periods of extremely unfavourable hydraulic conditions such as floods, areas susceptible to piping shall be inspected regularly so that necessary mitigating measures can be taken without delay. Materials for such measures shall be stored in the vicinity.

(3)P REQ Failure by piping shall be prevented by providing sufficient resistance against internal soil erosion in the areas where water outflow may occur.

(4) POS Such failure *can* be prevented by providing:

- sufficient safety against failure by heave where the ground surface is horizontal;
- sufficient stability of the surface layers in sloping ground (local slope stability).

(5)P REQ When determining the outflow hydraulic conditions for the verification of failure by heave or of local slope stability, account shall be taken of the fact that joints or interfaces between structure and ground can become preferred seepage paths.

Section 11 Overall stability

11.1 General

(1)P REQ The provisions in this section shall apply to the overall stability of and movements in the ground, whether natural or fill, around foundations, retaining structures, natural slopes, embankments or excavations.

(2) REC Account should be taken of overall stability clauses, related to specific structures, in sections 6 to 10 and 12.

11.2 Limit states

(1)P REQ All possible limit states for the particular ground shall be considered in order to fulfil the fundamental requirements of stability, limited deformations, durability and limitations in movements to nearby structures or services.

(2) ~~REC~~-POS Some possible limit states are listed below:

- loss of overall stability of the ground and associated structures;
- excessive movements in the ground due to shear deformations, settlement, vibration or heave;
- damage or loss of serviceability in neighbouring structures, roads or services due to movements in the ground.

11.3 Actions and design situations

(1)P REQ The list in 2.4.2(4)P shall be taken into account when selecting the actions for calculation of limit states.

(2)P REQ The effects of the following circumstances shall be taken into account as appropriate:

- construction processes;
- new slopes or structures on or near the particular site;
- previous or continuing ground movements from different sources;
- vibrations;
- climatic variations; freezing and thawing; droughts and heavy rains;
- vegetation or its removal;
- human or animal activities;
- variations in water content or porewater pressure and waves on free water.

(3)P REQ In ultimate limit states, design free water and groundwater levels, or their combination, *shall* be selected from available hydrological data and in situ observations to give the most [~~realistic~~] unfavourable conditions that could occur in the design situation being considered. The possibility of failure of drains, filters or seals *shall* be considered.

(4) REC The possibility of emptying a canal or water reservoir for maintenance or due to dam failure should also be considered. For serviceability limit states [~~an average of~~] *less severe, more typical* water level or porewater pressure may be used.

(5) REC For slopes along waterfronts the most unfavourable hydraulic conditions are normally steady seepage for the highest possible groundwater level and rapid drawdown of the free water level.

(6)P REQ In deriving design distributions of porewater pressure, account *shall* be taken of the possible range of permeability anisotropy and variability of the ground.

11.4 Design and construction considerations

(1)P REQ The overall stability of a site and movements of natural or made ground shall be checked taking into account comparable experiences according to 1.5.2(2)P.

(2)P REQ The overall stability and movement of ground supporting existing buildings, new structures, slopes or excavations shall be considered.

(3) REC In cases where the stability of the ground cannot be clearly verified prior to design, additional investigations, monitoring and analysis should be specified according to the provisions of 11.7.

(4) REC Typical structures for which an analysis of overall stability should be performed are:

- ground retaining structures;
- excavations, slopes, embankments;
- foundations on sloping ground, natural slopes or embankments;
- foundations near an excavation, cut or buried structures, or shore.

~~(5) REC~~ NOTE: Stability problems or creep movements occur primarily in cohesive soils with a sloping ground surface. However, instability can also occur in cohesionless soils and fissured rocks in slopes where the inclination, *which may be determined by erosion*, is close to the angle of ~~[internal friction, often shaped by erosion]~~ shearing resistance. Increased movements are often observed at elevated porewater pressures or close to the ground surface during the freezing and thawing cycles.

(5)P REQ If the stability of a site cannot readily be verified or the movements are found to be not acceptable for the site's intended use, the site shall be judged to be unsuitable without stabilizing measures.

(6)P REQ The design shall ensure that all construction activities in and on the site can be planned and executed such that the occurrence of an ultimate or serviceability limit state is sufficiently improbable.

(7)P REQ Slope surfaces exposed to potential erosion shall be protected if required to ensure that the safety level is retained.

(8) REC Slopes should be sealed or planted, or protected artificially. For slopes with berms, a drainage system within the berm should be considered.

(9)P REQ Construction processes shall be taken into account as far as they might affect the overall stability or the magnitude of movement.

(10) REC Potentially unstable slopes may be stabilized by:

- a concrete cover with or without anchorage;
- an abutment of gabions, either of steel net or geotextile cages;
- ground nailing;
- vegetation;
- a drainage system;
- a combination of the above.

(11) REC The design should follow the general principles of sections 8 and 9.

11.5 Ultimate limit state design

11.5.1 Stability analysis for slopes

(1)P REQ The overall stability of slopes including existing, affected or planned structures, shall be verified in ultimate limit states (GEO and STR) with design values of actions, resistances and strengths, ~~[in accordance with 2.4.7.1(2)P]~~ where the partial factors should be selected from Annex A (Tables A.2.1, A.2.2 and A.2.3.6).

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

(2)P REQ In analysing the overall stability of the ground, on soil or rock, all relevant modes of failure shall be taken into account.

(3) REC When choosing a calculation method the following should be considered:

- soil layering;
- occurrence and inclination of discontinuities;
- seepage and porewater pressure distribution;
- short- and long-term stability;
- creep deformations due to shear;
- type of failure (circular or non-circular surface; toppling; flow);
- use of numerical methods.

(4) REC The mass of soil or rock bounded by the failure surface should normally be treated as a rigid body or as several rigid bodies moving simultaneously. *Failure surfaces or interfaces between rigid bodies may have a variety of shapes including planar, circular and more complicated shapes.* Alternatively, stability may be checked by *limit analysis* ~~[finding a statistically admissible field]~~ or using the finite element method. ~~[Failure surfaces or interfaces between rigid bodies may have a variety of shapes including planar, circular and more complicated shapes].~~

(5) REC Where ground or embankment material is relatively homogeneous and isotropic, ~~it is usually sufficient to assume~~ circular failure surfaces *should normally be assumed.*

(6) REC For slopes in layered soils with considerable variations of shear strength, special attention should be paid to the layers with lower shear strength. This may require analysis of non-circular failure surfaces.

(7) REC In jointed materials, including hard rock and layered or fissured soils, the shape of the failure surface ~~may~~*can* partly or fully be governed by discontinuities. ~~In this case this will normally require analysis of three-dimensional wedges should normally be made.~~

(8) REC Existing failed slopes which can potentially be reactivated *should* usually *be analysed*, ~~require the analysis of using~~ non-circular failure surfaces. Partial factors normally used for overall stability analyses then ~~may~~*need* not be appropriate.

(9) REC ~~Where problem geometry deviates significantly from plane strain, it may be required if the failure surface is significantly non-planar, the use of to consider three-dimensional failure surfaces should be considered.~~

(10)[PER] REC ~~[In ground which does not exhibit marked strength anisotropy, a method of vertical slices may be used] The method A slope analysis should verify the overall moment and vertical stability of the sliding mass. If horizontal equilibrium is not checked, inter-slice forces should be assumed to be horizontal.~~

(11)P REQ In cases where a combined failure of structural members and the ground could occur, the ground structure interaction shall be considered by allowing for the difference in their relative stiffnesses. Such cases include failure surfaces intersecting structural members such as piles and flexible walls.

NOTE: In analysing natural slopes it is generally an advantage to make a first calculation using characteristic values, to get an idea of the global factor of safety before starting any design. Experiences with comparable cases including investigation procedures should be applied.

(12) REC Since a distinction between favourable and unfavourable gravity loads is not possible in assessing the most adverse slip surface, any uncertainty about soil weight density should be considered by applying upper and lower characteristic values of it.

(13)P REQ The design shall show that the deformation of the ground under design actions due to creep or regional settlements will not cause *unacceptable* damage to structures ~~transportation network or services or infrastructure~~ sited on, in or near the particular ground.

11.5.2 Slopes and cuts in rock masses

(1)P REQ The stability of slopes and cuts in rock masses shall be checked against translational and rotational modes of failure involving isolated rock blocks or large portions of the rock mass, and also against rock falls. *Particular attention shall be given to the pressure caused by blocked seepage water in joints and fissures.*

(2)P REQ Stability analyses shall be based on reliable knowledge of the pattern of discontinuities intersecting the rock mass and of the shear strength of the intact rock and of the discontinuities.

(3) REC Account should be taken of the fact that failure of slopes and cuts in hard rock masses, with a well defined pattern of discontinuities, will generally involve:

- sliding of blocks or rock wedges;
 - toppling of blocks or slabs;
 - a combination of toppling and sliding,
- depending on the orientation of the slope face in relation to that of the discontinuities.

(4) REC *It should be considered that failure of slopes and cuts in highly fissured rock masses and in soft rocks and cemented soils may develop along circular or almost circular slip surfaces passing through portions of intact rock.*

(5) REC Sliding of isolated blocks and wedges ~~can~~ *should* usually be prevented by reducing the inclination of the slope by providing berms, and installing anchors, bolts and internal drainage. In cutting slopes sliding ~~may~~ *should* be prevented by selecting the direction and orientation of the slope face so that movements of isolated blocks are kinematically impossible.

(6) REC *To prevent toppling failures, anchoring or bolting and internal drainage should normally be applied.*

(7) REC When considering the long-term stability of slopes and cuts the detrimental effects of vegetation and environmental or polluting agents on the shear strength of discontinuities and on the strength of the intact rock should be taken into account.

(8) REC In highly fractured rock masses in steep slopes and slopes susceptible to toppling, spalling, ravelling and slumping, the possibility of rock falls should always be analysed.

(9) REC In cases where reliable provisions to prevent rock falls are not feasible, ~~it may be appropriate to allow~~ rock falls *should be allowed* to occur *with the provision of* nets, barriers and other suitable provision to trap the falling rock.

(10) REC The design of provisions to trap rock blocks and debris falling down a rock slope should be based on a thorough investigation of the possible trajectories of the falling material.

11.5.3 Stability of excavations

(1)P REQ The overall stability of the ground close to an excavation, including excavation spoil and existing structures, roads and services shall be checked (see section 9).

(2)P REQ The bottom stability of an excavation shall be checked in relation to the design porewater pressure in the ground. For the analysis of hydraulic failure see Section 10.

(3)P REQ ~~The~~ Heave of the bottom of deep excavations due to unloading shall be considered.

[11.5.4 – moved to 11.4]

[~~(1) REC Potentially unstable slopes may be stabilized by:~~

- ~~— a concrete cover with or without anchorage;~~
- ~~— an abutment of gabions, either of steel net or geotextile cages;~~
- ~~— ground nailing;~~
- ~~— vegetation;~~
- ~~— a drainage system;~~
- a combination of the above]

11.6 Serviceability limit state design

(1)P REQ The design shall show that the deformation of the ground under characteristic actions will not cause a serviceability limit state in structures, ~~roads or services~~ *infrastructure* on or near the particular ground.

(2) REC Subsidence of the ground due to the following causes should be considered:

- change in groundwater conditions and corresponding porewater pressures;
- long-term creep under drained conditions;
- volume loss of deep soluble strata;
- mining or similar works such as gas extraction.

(3) REC Since the analytical and numerical methods available at present do not usually provide reliable predictions of the deformation of a natural slope, the occurrence of *serviceability* limit states should be avoided by *one of the following*:

- limiting the mobilised shear strength; or
- observing the movements and specifying actions to reduce or stop them if necessary.

11.7 Monitoring

(1)P REQ The ground shall be monitored using appropriate equipment if:

- it is not possible to prove by calculation or by prescriptive measures that the occurrence of the limit states given in 11.2 is sufficiently unlikely;
- the assumptions made in the calculations are not based on reliable data.

(2) REC Monitoring should be planned to provide knowledge of:

- groundwater levels or porewater pressures in the ground, so that effective stress analyses can be carried out or checked;
- lateral and vertical ground movements, in order to predict further deformations;
- the depth and shape of the moving surface in a developed slide, in order to derive the ground strength parameters for the design of remedial works;
- rates of movement, in order to give warning of impending danger; in such cases a remote digital readout for the instruments or a remote alarm system may be appropriate.

~~[(3) REC The construction of embankments on soft soil with low permeability should be monitored and controlled by means of porewater pressure measurements in the soft layers and settlement measurements of the fill] (moved to 12.7)~~

Section 12 Embankments

12.1 General

(1)P REQ The provisions of this section *shall* apply to embankments ~~[for dikes,]~~ for small dams and for ~~transportation networks infrastructure~~.

(2) REC For placement and compaction of fill ~~are the provisions considered~~ in section 5 *should be applied*.

12.2 Limit states

(1)P REQ A list shall be compiled of limit states for the embankment to be designed.

(2) REC The following limit states should be checked:

- loss of overall site stability;
- failure in the embankment slope or crest;
- failure caused by internal erosion;
- failure caused by surface erosion or scour;
- deformations in the embankment leading to loss of serviceability, e.g. excessive settlements or cracks;
- settlements and creep displacements leading to damages or loss of serviceability in nearby structures or utilities;
- excessive deformations in transition zones, e.g. the access embankment of a bridge abutment;
- loss of serviceability of traffic areas by climatic influences such as freezing and thawing or extreme drying;
- creep in slopes during the freezing and thawing period;
- degradation of base coarse material due to high traffic loads;
- deformations caused by hydraulic actions;
- changes of environmental conditions such as pollution of surface or groundwater, noise or vibrations.

12.3 Actions and design situations

(1)P REQ In selecting the actions for the calculation of limit states, the list in 2.4.2.(4)P shall be considered.

~~[(2)P REQ Traffic loads for roads and railroads shall be taken from EN1991-3]~~

(2) REC When deriving the actions that embankments impose on adjacent structures or any reinforced parts of the ground, the differences in the stiffnesses should be considered.

(3)P REQ Design situations shall be selected in accordance with the principles stated in 2.2.

(4)P REQ In addition, the following special design situations shall be taken into account, if relevant:

- the effects of the construction process, such as excavations close to the embankment fill and vibrations caused by blasting, pile driving or heavy equipment;

- the effects of structures to be constructed on or close to the embankment;
- the erosion effects of overtopping, ice, waves and rain on the slopes and crest;
- temperature effects such as shrinkage;
- human activities.

(5)P REQ The design free water level on the downstream embankment slope and the design groundwater level, or their combination, shall be based on available hydrological data to give the most unfavourable conditions that could occur in the design situation considered. The possibility of failure of drains, filters or seals shall be considered.

(6) REC For shore embankments, ~~like dikes~~ the most unfavourable hydraulic conditions *should be considered*. These are normally steady seepage for the highest possible groundwater level and rapid draw-down of the free water level.

(7)P REQ In deriving design distributions of porewater pressure, account shall be taken of the possible range of anisotropy and heterogeneity of the soil.

(8)P REQ When designing the embankment with respect to settlement, the effective stress decrease in the ground due to submergence of the dry crust or the fill shall be taken into account.

12.4 Design and construction considerations

(1)P REQ Embankments shall be designed taking into account experience with embankments on similar ground and made of similar fill material.

(2)P REQ When assessing the foundation level for an embankment, the following shall be considered, when applicable:

- reaching an adequate bearing stratum, or applying stabilising measures where this is not practicable;
- providing sufficient protection against adverse, climatic effects on the bearing capacity of the ground;
- the groundwater level with respect to the drainage of the embankment;
- avoiding adverse effects on adjacent structures and utilities;
- reaching layers with adequate low permeability.

(3) REC Design of embankments should ensure that:

- the bearing capacity of the subsoil is satisfactory;
- the drainage of the various fill layers is satisfactory;
- the permeability of the fill material in dams [~~and dikes~~] is as low as required;
- filters or [~~membranes~~] *geosynthetics* are specified where necessary to fulfill filter criteria;
- the fill material is specified according to the criteria in 5.3.2.

(4)P REQ For embankments on ground with low strength and high compressibility, the construction process shall be specified so as to ensure that the bearing capacity is not exceeded and that excessive settlements or movements do not occur during construction (see 5.3.3(2)P).

(5) REC When the embankment on compressible ground is raised in layers, piezometer measurements should be specified to ensure that porewater pressures have dissipated to acceptably low values before the next fill layer is placed.

(6)P REQ For embankments ~~[of dams and dikes]~~ *retaining water at different water levels*, the foundation level shall be chosen with respect to the permeability of the ground or measures *shall be taken* to make the structure watertight.

(7) REC If ground improvement is specified, the volume of ground to be improved ~~shall~~ *should* be designed with sufficient areal extent to avoid harmful deformations ~~[at the crest of the embankment]~~.

(8) REC When determining the weight of the embankment from the weight density of fill (see 3.3.3), care should be taken to include fill particles of size > 20 mm to 60 mm in the density tests. They are often not included but ~~may~~ *can* have a considerable effect on the weight density.

(9)P REQ Embankment slope surfaces exposed to erosion shall be protected. If berms are designed, a drainage facility shall be specified for the berm.

(10) REC The slopes should be sealed during embankment construction and planted thereafter where appropriate.

(11) REC For embankments carrying traffic, icing on the surface of the pavement should be avoided. The thermal capacity of the pavement on an insulation layer or a lightweight fill may be high enough to avoid this.

(12) REC Frost penetration on the crest of an earth dam should be restricted to an appropriate level.

(13) REC Design of the embankment slope should consider that creep movements may occur in slopes during freezing and thawing irrespective of the slope stability under dry conditions. This is especially important in transition zones, e.g. at bridge abutments.

12.5 Ultimate limit state design

(1)P REQ In analysing the stability of part or all of an embankment, all possible failure modes shall be considered, as stated in section 11.

(2) REC Since embankments are often constructed in different phases with different load conditions, the analysis should be done phase by phase and provisions specified accordingly in the geotechnical design report.

(3)P REQ Where lightweight fill materials such as expanded polystyrene, expanded clay or foamed concrete are used, the possibility of buoyancy effects shall be considered (*see section 10*).

(4)P REQ Any analysis of embankments containing different fill materials shall adopt strength values that have been determined at compatible strains in the materials.

(5) REC Where roads or watercourses cross an embankment, special attention should be paid to the spatial interaction of the various structural elements.

(6) REC When analysing the stability of improved ground, the effect of the improving process, e.g. the disturbance of soft sensitive clay, should be considered. As the effect of the improvement is time-dependent, it should not be taken into account until a steady state has been reached.

(7)P REQ To avoid ultimate limit states caused by surface erosion, internal erosion or hydraulic pressure, the provisions in sections 10 and 11 shall be fulfilled.

12.6 Serviceability limit state design

(1)P REQ The design shall show that the deformation of the embankment when subjected to characteristic actions will not cause a serviceability limit state in the embankment or in structures, roads or services sited on, in or near the embankment.

(2) REC The settlement of an embankment on compressible ground ~~may~~ *should* be calculated using the principles of 6.6.1. Special attention should be paid to the time dependency of the settlements due to both consolidation and secondary settlements.

(3) REC The possibility of deformations due to changes in the groundwater conditions should be taken into account.

(4) REC In cases where the deformations are difficult to predict, the methods of preloading or trial embankments should be considered, especially in cases where serviceability limit states shall be prevented.

12.7 Supervision and monitoring

(1)P REQ Supervision and monitoring of embankments shall follow the provisions in section 4.

(2) REC Monitoring should be applied to embankments *in one or more of the following situations*:

- [~~classified as Geotechnical Category 3; or~~]
- *when using* the observational method, see 2.7;
- ~~used as dams~~ **[and dikes]** and where the stability of an embankment acting as a dam to a large degree depends on the pore water pressure distribution in and beneath the embankment;
- where records of pollution effects of fill or traffic are required;
- where control of adverse effects on structures or utilities is required;
- where surface erosion is a considerable risk.

(3)P REQ In cases where a supervision and monitoring programme is required, the designer shall present it in the geotechnical design report, see 2.8. It shall be specified that the monitoring records are to be evaluated and acted upon as necessary.

(4) REC A monitoring programme for an embankment ~~may~~ *should* contain the following records:

- porewater pressure measurements in and beneath the embankment;

- settlement measurements for the whole or parts of the embankment and influenced structures;
- measurements of horizontal displacements;
- checks on strength parameters of fill material during construction;
- chemical analyses before, during and after construction, if pollution control is required;
- observations of erosion protection;
- checks on permeability of fill material and of foundation soil during construction;
- depth of frost penetration of the crest of an embankment.

(5) REC The construction of embankments on soft soil with low permeability should be monitored and controlled by means of porewater pressure measurements in the soft layers and settlement measurements of the fill.

(moved from 11.7(3))

Annex A
(normative)
**Partial factors for ultimate limit states
and recommended values**

A.0 Partial factors and their recommended values

(1)P REQ The partial factors γ for ultimate limit states in persistent and transient situations, and the correlation factors ξ for pile foundations, shall be those shown in the Tables in this annex.

(2) REC The recommended values for these partial factors should be taken from the Tables in this Annex.

NOTE: The partial factor values may be set by the National annex.

A.1 Partial factors for equilibrium *limit state* (EQU) verification

A.1.1 Partial factors on actions (γ_F)

Action	Symbol	value
Permanent Unfavourable ¹	$\gamma_{G,dst}$	1,10
Favourable ²	$\gamma_{G,stab}$	0,90
Variable Unfavourable ¹	$\gamma_{Q,dst}$	1,50
Favourable ²	$\gamma_{Q,stab}$	0
¹ Destabilising; ² Stabilising		

A.1.2 Partial factors for soil parameters (γ_M)

Soil parameter	Symbol	value
Shearing resistance ¹	γ_ϕ	1,25
Effective cohesion	$\gamma_{c'}$	1,25
Undrained strength	γ_{cu}	1,40
Unconfined strength	γ_{qu}	1,40
Weight density	γ_γ	1,00
¹ This factor is applied to $\tan \phi'$		

A.2 Partial factors [~~on actions (γ_F) or action effects (γ_E)~~] for structural (STR) and geotechnical (GEO) limits states verification

A.2.1 Partial factors on actions (γ_F) or the effects of actions (γ_E)

Action		Symbol	Set	
			A1	A2
Permanent	Unfavourable	γ_G	1,35	1,0
	Favourable		1,0	1,0
Variable	Unfavourable	γ_Q	1,5	1,3
	Favourable		0	0

A.2.2 Partial factors for soil parameters(γ_M)

Soil parameter	Symbol	Set	
		M1	M2
Shearing resistance	γ_ϕ^1	1,0	1,25
Effective cohesion	γ_c	1,0	1,25
Undrained strength	γ_{cu}	1,0	1,4
Unconfined strength	γ_{qu}	1,0	1,4
Unit weight density	γ_σ	1,0	1,0

¹ This symbol is related to $\tan \phi'$

A.2.3 Partial resistance factors (γ_R)

A.2.3.1 Spread foundations

Resistance	Symbol	Set		
		R1	R2	R3
Bearing	γ_{Rv}	1,0	1,4	1,0
Sliding	γ_{Rh}	1,0	1,1	1,0

A.2.3.2 Pile foundations

A.2.3.2.1 Driven piles

Resistance	Symbol	Set			
		<i>R1</i>	<i>R2</i>	<i>R3</i>	<i>R4</i>
Base	γ_b	1,0	1,1	1,0	1,3
Shaft (compression)	γ_s	1,0	1,1	1,0	1,3
Total/combined (compression)	γ_t	1,0	1,1	1,0	1,3
Shaft in tension	$\gamma_{s,t}$	1,25	1,15	1,1	1,6

A.2.3.2.2. Bored piles

Resistance	Symbol	Set			
		<i>R1</i>	<i>R2</i>	<i>R3</i>	<i>R4</i>
Base	γ_b	1,25	1,1	1,0	1,6
Shaft (compression)	γ_s	1,0	1,1	1,0	1,3
Total/combined (compression)	γ_t	1,15	1,1	1,0	1,5
Shaft in tension	$\gamma_{s,t}$	1,25	1,15	1,1	1,6

A.2.3.2.3. Continuous flight auger (CFA) piles

Resistance	Symbol	Set			
		<i>R1</i>	<i>R2</i>	<i>R3</i>	<i>R4</i>
Base	γ_b	1,1	1,1	1,0	1,45
Shaft (compression)	γ_s	1,0	1,1	1,0	1,3
Total/combined (compression)	γ_t	1,1	1,1	1,0	1,4
Shaft in tension	$\gamma_{s,t}$	1,25	1,15	1,1	1,6

A.2.3.3 Correlation factors for pile foundations

A.2.3.3.1 Correlation factors ξ to derive characteristic values from static pile load tests (n - number of tested piles)

ξ for $n =$	1	2	3	4	≥ 5
ξ_1	1,40	1,30	1,20	1,10	1,00
ξ_2	1,40	1,20	1,05	1,00	1,00

A.2.3.3.2 Correlation factors ξ to derive characteristic values from semi-empirical method (n - the number of profiles of tests)

ξ for $n =$	1	2	3	4	5	7	10
ξ_3	1,40	1,35	1,33	1,31	1,29	1,27	1,25
ξ_4	1,40	1,27	1,23	1,20	1,15	1,12	1,08

A.2.3.3.3 Correlation factors ξ to derive characteristic values from dynamic impact tests^{1,2,3,4,5} (n - number of tested piles)

ξ for n	≥ 2	≥ 5	≥ 10	≥ 15	≥ 20
ξ_5	1,60	1,50	1,45	1,42	1,40
ξ_6	1,50	1,35	1,30	1,25	1,25

¹ The ξ -values in the table are valid for dynamic impact tests.
² The ξ -values may be multiplied with a model factor = 0,85 when using dynamic impact tests with signal matching.
³ The ξ - values should be multiplied with a model factor = 1,10 when using a pile driving formula with measurement of the quasi-elastic pile head displacement during the impact.
⁴ The ξ -values shall be multiplied with a model factor = 1,20 when using a pile driving formula without measurement of the quasi-elastic pile head displacement during the impact.
⁵ The number n of test piles should be determined among similar piles in the foundations

A.2.3.4 Partial resistance factors for pre-stressed anchorages

Resistance	Symbol	Set			
		R1	R2	R3	R4
Temporary	$\gamma_{a,t}$	1,1	1,1	1,0	1,1
Permanent	$\gamma_{a,p}$	1,1	1,1	1,0	1,1

A.2.3.5 Partial resistance factors for retaining structures

Resistance	Symbol	Set		
		R1	R2	R3
Bearing capacity	$\gamma_{R,v}$	1,0	1,4	1,0
Sliding resistance	$\gamma_{R,h}$	1,0	1,1	1,0
Earth resistance	$\gamma_{R,e}$	1,0	1,4	1,0

A.2.3.6 Partial resistance factors for slopes and overall stability

Resistance	Symbol	Set		
		R1	R2	R3
Earth resistance	$\gamma_{R,e}$	1,0	1,1	1,0

A.3 Partial factors for uplift *limit state* (UPL) verifications**A.3.1 Partial factors on actions (γ_F)**

Action	Symbol	Value
Permanent Unfavourable ¹	$\gamma_{G,dst}$	1,00
Favourable ²	$\gamma_{G,stab}$	0,90
Variable Unfavourable ¹	$\gamma_{Q,dst}$	1,50
¹ Destabilising; ² Stabilising		

A.3.2 Partial factors for soil parameters (γ_M)

Soil parameter	Symbol	Value
Shearing resistance	γ_ϕ^1	1,25
Effective cohesion	γ_c	1,25
Undrained strength	γ_{cu}	1,40
Tensile pile resistance	$\gamma_{s,t}$	1,40
Anchorage	γ_R	1,40
¹ This symbol is related to $\tan\phi'$		

A.4 Partial factors for hydraulic heave *limit state* (HYD) verification

Action	Symbol	Value
<i>Permanent</i>		
<i>Unfavourable</i> ¹	$\gamma_{G,dst}$	1,35
<i>Favourable</i> ²	$\gamma_{G,stab}$	0.90
<i>Variable</i>		
<i>Unfavourable</i> ¹	$\gamma_{Q,dst}$	1.50
¹ <i>Destabilising;</i> ² <i>Stabilising</i>		

Annex B
(informative)
**Background information on partial factors for
Design Approaches 1, 2 and 3**

B.1 General

(1) For limit state types STR and GEO *in persistent and transient situations*, three Design Approaches are outlined in 2.4.7.3.4. They differ in the way they distribute partial factors between actions, the effects of actions, material properties and resistances. In part, this is due to differing approaches to the way in which allowance is made for uncertainties in modeling the effects of actions and resistances.

(2) In Design Approach 1, for all designs, checks are, in principle, required for two sets of factors, applied in two separate calculations. Where it is obvious that one of these sets governs the design, it will not be necessary to carry out calculations for the other set. Generally, factors are applied to actions, rather than to the effects of actions, though with one noted exception [~~2.4.7.3(3)~~] (2.4.7.3.2(2)). In many cases, factors are applied to [~~material strengths~~] *ground parameters*, but for the design of piles and anchors they are applied to resistances.

(3) In Approaches 2 and 3, a single calculation is required for each part of a design, and the way in which the factors are applied is varied according to the calculation considered.

(4) In Design Approach 2, factors are applied either to actions or effects of actions and to resistances. Design Approach 2 is an action (effect) and resistance factor approach.

(5) In Design Approach 3, factors are applied to actions or effects of actions from the structure and to ground strength parameters. Design Approach 3 is an action (effect) and material factor approach.

B.2 Factors on actions and the effects of actions

(1) EN 1990 states that γ_f is a partial factor for an action and takes account of the possibility of unfavourable deviations of the action value from its characteristic value. Likewise γ_{sd} is a partial factor taking account of uncertainties in modeling the actions and in modeling the effects of actions.

(2) EN 1990 allows γ_{sd} and γ_f to be combined into one factor multiplying F_k :

$$\gamma_F = \gamma_{sd} \cdot \gamma_f \quad (\text{B.1})$$

(3) The various approaches in EN 1997-1 require that factors be applied either to actions or the effects of actions. Since the use of model factors γ_{sd} for actions from the ground will

remain exceptional and is therefore left to national determination, γ_F is used throughout for simplicity for actions and γ_E for effects of actions in geotechnical design, see Annex A, Tables A.1.1 and A.2.1.

This enables national authorities to select alternative values of the combination $\gamma_{sd} \times \gamma_F$

(4) Equation (2.6) includes X_k / γ_M in the calculation of actions because ground material properties may affect the values of geotechnical actions in some cases.

(5) In **Design Approach 1**, checks are required for two ~~[sets]~~ *combinations of sets* of factors, applied in two separate calculations.

In ~~[Set 1.4]~~ *Combination 1*, factors $\neq 1$ are generally applied to actions, with factors =1 on the effects of actions. Thus $\gamma_F \neq 1$ and $\gamma_E = 1$ are applied in equation (2.6).

An exception to this is noted in 2.4.7.3.2(2): in cases where it would be physically unreasonable to apply $\gamma_F \neq 1$ (example: tank with fixed fluid level), then $\gamma_F = 1$ and $\gamma_E \neq 1$ are used.

In ~~[Set 1.2 or Sets 1.2 and 1.3]~~ *Combination 2*, ~~(for pile or anchor design)~~ $\gamma_E = 1$ is always used, with $\gamma_F \neq 1$ only for variable actions.

Thus, except as noted in 2.4.7.3.2(2), for Design Approach 1 equation.2.6 reduces to

$$E_d = E \{ \gamma_F F_{rep}, X_k / \gamma_M, a_d \} \quad (B.2).$$

(6) In **Design Approach 2**, a single calculation is required for each part of a design, and the way in which the factors are applied either to actions or the effects of actions is varied according to the calculation considered and chosen according to national determination.

Either $\gamma_E \neq 1$ and $\gamma_F = 1$, or $\gamma_F \neq 1$ and $\gamma_E = 1$ are applied. Since $\gamma_M = 1$ is used, equation. 2.6 reduces to

$$E_d = \gamma_E \cdot E \{ F_{rep}, X_k, a_d \}, \text{ or,} \quad (B.3.1)$$

$$E_d = E \{ \gamma_F F_{rep}, X_k, a_d \} \quad (B.3.2)$$

(7) In **Design Approach 3**, a single calculation is required. However, in this Design Approach a difference is made between actions F_{rep} from the structure and actions from or through the ground calculated from X_k . Either $\gamma_E \neq 1$ and $\gamma_F = 1$ or $\gamma_E = 1$ and $\gamma_F \neq 1$ are applied. Thus equation.2.6 remains:

$$E_d = E \{ \gamma_F F_{rep}, X_k / \gamma_M, a_d \} \text{ or,} \quad (B.4.1)$$

$$E_d = \gamma_E E \{ F_{rep}, X_k / \gamma_M, a_d \} \quad (B.4.2)$$

B.3 Factors on material strengths and resistances

(1) Equation (6.6) of EN 1990 and Equation (2.7) of EN 1997-1 are equivalent:

$$R_d = \frac{1}{\gamma_{Rd}} R \{ X_{d,i}; a_d \} = \frac{1}{\gamma_{Rd}} R \left\{ \eta_i \frac{X_{k,i}}{\gamma_{m,i}}; a_d \right\}, \quad (\text{EN 1990, equation. 6.6}) \quad (\text{B.5.1})$$

$$R_d = \frac{1}{\gamma_R} \left\{ \gamma_F F_{rep}, \frac{X_k}{\gamma_M}, a_d \right\}, \quad (\text{EN 1997-1, equation.2.7}) \quad (\text{B.5.2})$$

(2) Note that EN 1997-1, Equation 2.7, includes $\gamma_F F_{rep}$ in the calculation of design resistances because the magnitudes of actions may affect the values of geotechnical resistances in some cases, e.g. earth resistance.

(3) The value of the conversion factor η is taken to be 1,0 in EN 1997-1 because the characteristic material strengths are defined to be those relevant to the field situation, thereby including η in the characteristic value.

(4) The various approaches in this standard require that factors be applied either to material strengths (X) or to resistances (R). These factors combine the roles of the material factors γ_m and the resistance model factors (γ_{Rd}) in various ways. For simplicity, the factors applied to material strengths (X) are denoted (γ_M), and the factors applied to resistances (R) are denoted γ_R .

(5) In **Design Approach 1**, checks are required for *combinations of sets* of factors for two separate calculations.

In [~~Set 1.4~~] Combination 1, factors =1 are applied to material strength and resistances. Thus $\gamma_M = \gamma_R = 1$ in equation.(2.7).

In [~~Set 1.2~~] Combination 2, *except for piles and anchorages*, $\gamma_M > 1$, $\gamma_R = 1$.

Thus in most cases [~~factor sets 1.1 and 1.2 are used for~~] Design Approach 1 *adopts* Equation(2.7a) thus:

$$R_d = R \{ \gamma_F F_{rep}, X_k / \gamma_M, a_d \} \quad (\text{B.6.1.1})$$

But, *in Combination 2 for piles and anchors*, [~~Sets 1.1 and 1.3 are used and~~] $\gamma_M = 1$ and $\gamma_R > 1$ are used in equation.(2.7b) thus:

$$R_d = \frac{1}{\gamma_R} R \{ \gamma_F F_{rep}, X_k, a_d \} \quad (\text{B.6.1.2})$$

(6) In **Design Approach 2**, factors =1 are generally applied to material strengths, with factors >1 applied to resistances. Thus $\gamma_M = 1$; $\gamma_R > 1$ are used in equation.(2.7). Since $\gamma_F = 1$ is generally also used, equation.(2.7b) is used thus:

$$R_d = \frac{1}{\gamma_R} R \{ F_{rep}, X_k, a_d \} \quad (\text{B.6.2})$$

(7) In **Design Approach 3**, $\gamma_M > 1$ and $\gamma_R = 1$ are generally applied. Equation.(2.7a) is used thus:

$$R_d = R\{\gamma_F F_{rep}, X_k / \gamma_M, a_d\} \quad (\text{B.6.3.1})$$

But, note that sometimes there is also a need to have $\gamma_R > 1$ (piles in tension, for example), so that equation. (2.7a) is used thus:

$$R_d = R\{\gamma_F F_{rep}, X_k / \gamma_M, a_d\} / \gamma_R \quad (\text{B.6.3.2})$$

Annex C
(informative)

**Sample procedures to determine limit values
of earth pressures on vertical walls**

C.1 Limit values of earth pressure

(1) (REC) The limit values of earth pressure on a vertical wall, caused by weight density γ , uniform vertical surface load (q) and ground cohesion (c) should be calculated as follows:

– active limit state:

$$\begin{aligned}\sigma_a(z) &= K_{a,h} \cdot [\gamma \cdot z + q] - 2c \cdot \sqrt{K_{a,h}} \\ \tau_a(z) &= \sigma_a \cdot \tan \delta + a \quad (\text{positive for downward movement of ground})\end{aligned}\tag{C.1}$$

– passive limit state:

$$\begin{aligned}\sigma_p(z) &= K_{p,h} \cdot [\gamma \cdot z + q] + 2c \cdot \sqrt{K_{p,h}} \\ \tau_p(z) &= \sigma_p \cdot \tan \delta + a \quad (\text{positive for upward movement of ground})\end{aligned}\tag{C.2}$$

where:

a	is the adhesion (between ground and wall)
c	is the ground cohesion
$K_{a,h}$	the coefficient of horizontal active earth pressure
$K_{p,h}$	the coefficient of horizontal passive earth pressure
q	the vertical surface load
z	the distance down the face of the wall
β	the slope angle of the ground behind the wall (upward positive)
δ	the angle of shearing resistance between ground and wall
γ	weight density of retained ground
$\sigma_a(z)$	the stress normal to the wall at depth z (active limit state)
$\sigma_p(z)$	the stress normal to the wall at depth z (passive limit state)
$\tau_a(z)$	the stress tangential to the wall at depth z (active limit state)
$\tau_p(z)$	the stress tangential to the wall at depth z (passive limit state)

(2) (PER) Equations (C.1) and (C.2) may be applied, either in terms of total, or effective stress as appropriate.

(3) (PER) Values of the earth pressure coefficients may be taken from Figures C.1 for $K_{a,h}$ and C.2 for $K_{p,h}$. They are approximate on the safe side.

(4) (REC) In layered soils, the coefficients K should normally be determined by the shear strength parameters at depth z only, independent of the values at other depths.

(5) (PER) Intermediate values of active earth pressure between the state at rest and the limit state may be obtained by linear interpolation.

(6) (PER) Intermediate values of passive earth pressure between the state at rest and the limit state may be obtained by parabolic interpolation as shown in Figure C.3.

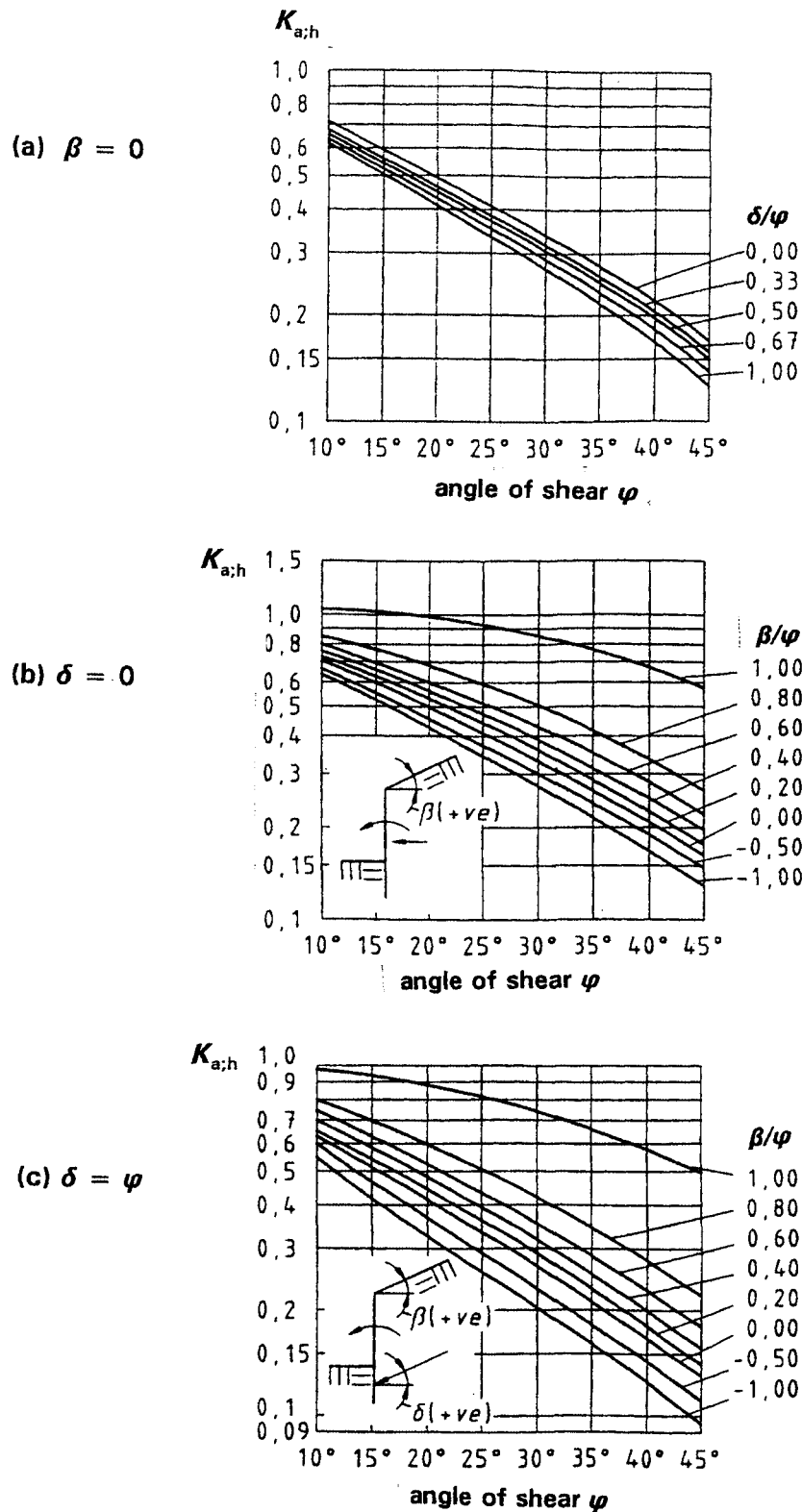


Figure C.1: Coefficients $K_{a,h}$ of active earth pressure
 (a) horizontal retained surface
 (b) inclined retained surface, $\delta = 0$
 (c) inclined surface, $\delta = \varphi$

143

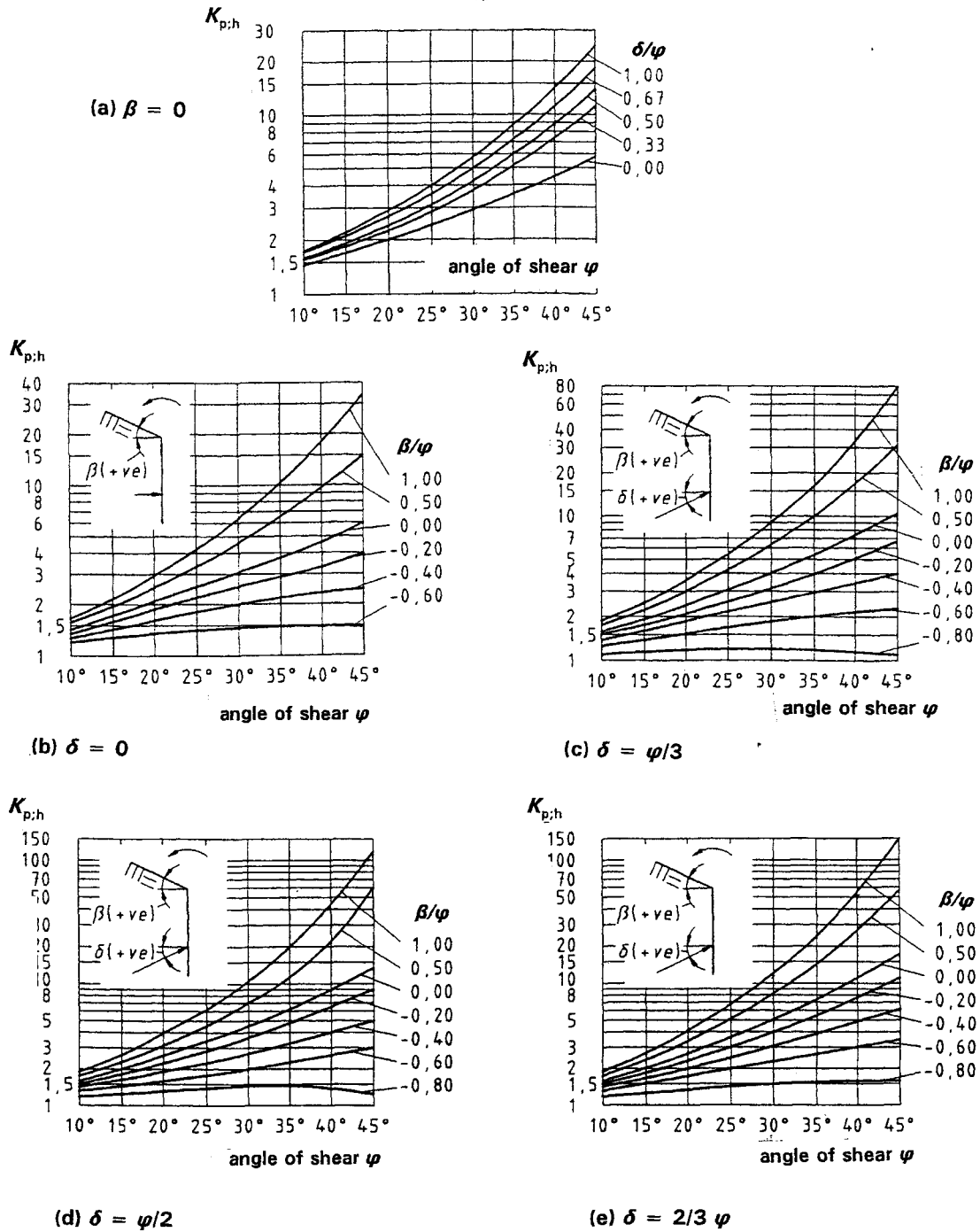


Figure C.2: Coefficients $K_{p,h}$ of earth resistance: (a) horizontal retaining surface, (b) to (e) inclined retaining surface and variable values of δ

C.2 Movements to mobilize limit earth pressures

(1) REC The movement needed for development of an active limit state in non-cohesive soil behind a vertical wall retaining horizontal ground should be considered depending on the kind of wall movement and the density of the soil. Table C.1 gives the order of magnitude of the ratio v_a/H .

Table C.1 Ratios v_a/h

Kind of wall movement	v_a/h loose soil %	v_a/h dense soil %
a)	0,4 to 0,5	0,1 to 0,2
b)	0,2	0,05 to 0,1
c)	0,8 to 1.0	0,2 to 0,5
d)	0,4 to 0,5	0,1 to 0,2
where: v_a is the wall motion to mobilise active earth pressure h is the height of the wall		

(2) REC Account should be taken that movement needed for development of a passive limit state of earth pressure in non-cohesive soil behind a vertical wall retaining horizontal ground is much larger than the one for the active earth pressure. Table C.2 gives the order of magnitude of the ratio v_p/H for the full passive earth pressure and, in brackets, for half of the limit value.

(3) REC The movement ratios in table C.2 should be increased by a factor of 1.5 to 2.0 if subaqueous ground *below the water table* is considered.

Table C.2 Ratios v_p/h

Kind of wall movement	v_p/h loose soil %	v_p/h dense soil %
a)	7 (1,5) to 25 (4,0)	5 (1,1) to 10 (2,0)
b)	5 (0,9) to 10 (1,5)	3 (0,5) to 6 (1,0)
c)	6 (1,0) to 15 (1,5)	5 (0,5) to 6 (1,3)
where: v_p is the wall motion to mobilise passive earth pressure h is the height of the wall		

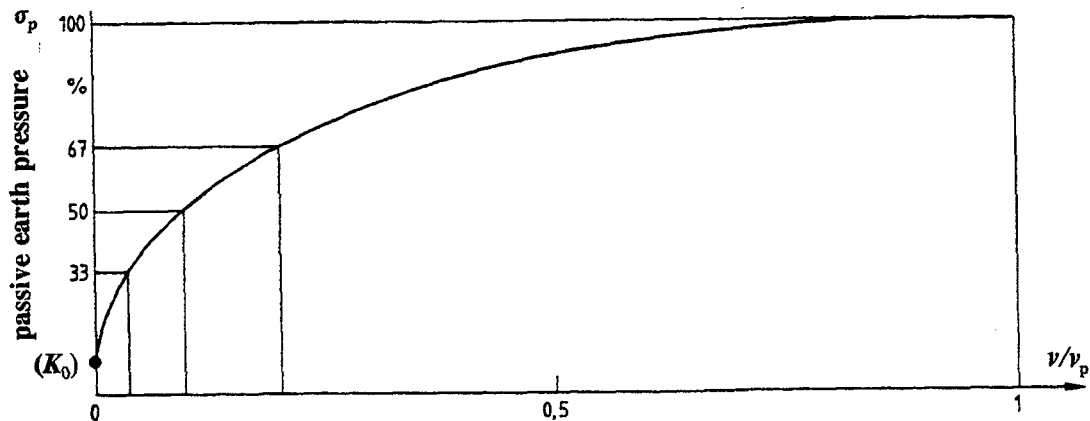


Figure C.3: Mobilization of passive earth pressure of non-cohesive soil versus wall displacement v_p (DIN 4085-100)

Annex D
(informative)
A sample analytical method
for bearing resistance calculation

D.1 Symbols used in Annex D

(1) The following symbols are used in annex D in addition to those in 1.7 and 1.8:

$A' = b' \times L'$	the design effective foundation area;
b	the design values of the factors for the inclination of the base, with subscripts c , q and γ ;
b	the foundation width;
b'	the effective foundation width (fig. D.1);
d	the embedment depth;
e	the eccentricity of the resultant action, with subscripts b and L (fig.D.1)
i	the inclination factors of the load, with subscripts cohesion c , surcharge q and weight density γ ;
L	the foundation length;
L'	the effective foundation length (fig. D.1);
m	exponent in formulas of i .
N	the bearing capacity factors, with subscripts for c , q and γ ;
q'	the design effective overburden pressure at the level of the foundation base;
s	the shape factors of the foundation base, with subscripts for c , q and γ ;
V	the vertical load;
α	the inclination of the foundation base to the horizontal;
δ	the design base friction angle;
γ'	the design effective weight density of the soil below the foundation level, reduced to $\gamma' = \gamma - \gamma_w(1+i)$ in the case of an upward hydraulic gradient i ;
γ	the design total weight density of the soil below the foundation level;
γ_w	the design weight density of groundwater;
θ	direction angle of H .

D.2 General

(1) *PER* Approximate equations for the design vertical bearing resistance, derived from plasticity theory and experimental results, may be used. Allowance should be made for the effects of the following:

- the strength of the ground, generally represented by the design values of c_u , c' and ϕ' ;
- eccentricity and inclination of design loads;
- the shape, depth and inclination of the foundation;
- the inclination of the ground surface;
- groundwater pressures and hydraulic gradients;
- the variability of the ground, especially layering.

D.3 Undrained conditions

(1) PER The design bearing resistance may be calculated from:

$$R/A' = (\pi+2) \times c_u \times b_c \times s_c \times i_c + q \quad (D.1)$$

with the dimensionless factors for:

- the inclination of the foundation base: $b_c = 1 - 2\alpha / (\pi+2)$

- the shape of the foundation:

$s_c = 1 + 0,2 (b'/L')$, for a rectangular shape;

$s_c = 1,2$, for a square or circular shape.

- the inclination of the load, caused by a horizontal load H :

$$i_c = \frac{1}{2} \left(1 + \sqrt{1 - \frac{H}{A' c_u}} \right)$$

with $H \leq A' \times c_u$

D.4 Drained conditions

(1) PER The design bearing resistance may be calculated from:

$$R/A' = c' \times N_c \times b_c \times s_c \times i_c + q' \times N_q \times b_q \times s_q \times i_q + 0,5 \times \gamma' \times B' \times N_\gamma \times b_\gamma \times s_\gamma \times i_\gamma \quad (D.2)$$

with the design values of dimensionless factors for:

- the bearing resistance:

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

$$N_c = (N_q - 1) \cot \phi'$$

$$N_\gamma = 2 (N_q - 1) \tan \phi', \text{ where } \delta \geq \phi'/2 \text{ (rough base)}$$

- the inclination of the foundation base:

$$b_c = b_q - (1 - b_q)/N_c \times \tan \phi'$$

$$b_q = b_\gamma = (1 - \alpha \times \tan \phi')^2$$

- the shape of foundation:

$$s_q = 1 + (b'/L') \sin \phi', \text{ for a rectangular shape;}$$

$$s_q = 1 + \sin \phi', \text{ for a square or circular shape;}$$

- $s_\gamma = 1 - 0,3 (b'/L')$, for a rectangular shape;

$$s_\gamma = 0,7, \text{ for a square or circular shape}$$

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

- the inclination of the load, caused by a horizontal load H :

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

$$i_q = [1 - H/(V + A'c' \cot \varphi')]^m$$

$$i_\gamma = [1 - H/(V + A'c' \cot \varphi')]^{m+1}$$

where:

$m = m_b = [2 + (b' / L')] / [1 + (b' / L')]$ when H acts in the direction of b' ;

$m = m_L = [2 + (L' / b')] / [1 + (L' / b')]$ when H acts in the direction of L' .

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

$$m = m_\theta = m_L \times \cos^2 \theta + m_b \times \sin^2 \theta.$$

Fig. D.1 Notations

Annex E

(informative)

A sample semi-empirical method for bearing resistance estimation

(1) To estimate the design bearing resistance of a foundation on soil field tests such as the pressuremeter test may be used.

(2) When using the pressuremeter, the design bearing resistance of a foundation subjected to a vertical load, R_d , is related to the limit pressure of the soil by the linear function:

$$R_d / A' = \sigma_{v0} + k \times p_{le}^* \quad (E.1)$$

where:

p_{le}^* is the design net equivalent limit pressure (from the pressure meter test)

and the other symbols defined in 1.8.

(3) Numerical values of the bearing resistance factor k are in the range of 0,8 to 3,0 depending on the type of soil, the embedment depth and the shape of the foundation.

(4) The design net equivalent limit pressure p_{le}^* is derived from the net limit pressure p_1^* which is defined for a pressuremeter test as the difference ($p_1 - p_0$) between the limit pressure p_1 and the at rest horizontal earth pressure p_0 at the level of the test; p_0 may be determined from an estimate of the at rest earth pressure coefficient K_0 and from the values of the effective overburden pressure q' and the pore water pressure u as $p_0 = K_0 q' + u$.

Annex F
(informative)
Sample methods for settlement evaluation

F.1 Stress-strain method

(1) PER The total settlement of a foundation on cohesive or non-cohesive soils may be evaluated using the stress-strain calculation method as follows:

- computing the stress distribution in the ground due to the loading from the foundation; this may be derived on the basis of elasticity theory, generally assuming homogeneous isotropic soil and a linear distribution of bearing pressure;
- computing the strain in the ground from the stresses using stiffness moduli values or other stress-strain relationships determined from laboratory tests (preferably calibrated against field tests) or field tests;
- integrating the vertical strains to find the settlements; to use the stress-strain method a sufficient number of points within the ground beneath the foundation should be selected and the stresses and strains computed at these points.

F.2 Adjusted elasticity method

(1) PER The total settlement of a foundation on cohesive or non-cohesive soil may be evaluated using elasticity theory and an equation of the form:

$$s = p \times b \times f / E_m \tag{F.1}$$

where:

E_m is the design value of the modulus of elasticity

f is the settlement coefficient

p is the bearing pressure, linearly distributed on the base of the foundation

and the other symbols are defined in 1.8.

(2) The value of the settlement coefficient f depends on the shape and dimensions of the foundation area, the variation of stiffness with depth, the thickness of the compressible formation, Poisson's ratio, the distribution of the bearing pressure and the point for which the settlement is calculated.

(3) If no useful settlement results measured at neighbouring similar structures in similar conditions are available, the design drained modulus E_m of the deforming stratum for drained conditions may be estimated from the results of laboratory or in situ tests.

(4) REC The adjusted elasticity method should only be used if the stresses in the ground are such that no significant yielding occurs and if the stress-strain behaviour of the ground may be considered to be linear. Great caution is required when using the adjusted elasticity method in the case of non-homogeneous ground.

F.3 Settlements without drainage

(1) PER The short-term components of settlement of a foundation, which occur without drainage, may be evaluated using either the stress-strain method or the adjusted elasticity method. The values adopted for the stiffness constants (such as E_m) and Poisson's ratio should in this case represent the undrained behaviour.

F.4 Settlements caused by consolidation

(1) PER To calculate the settlement caused by consolidation, a confined one-dimensional deformation of the soil may be assumed and the consolidation test curve is then used. Addition of settlements in the undrained and consolidation state often leads to an overestimate of the total settlement, and empirical corrections may be applied.

F.5 Time-settlement behaviour

(1) PER With cohesive soils the rate of consolidation settlement before the end of primary consolidation may be estimated approximately using consolidation parameters obtained from a compression test. However, the rate of consolidation settlement should preferably be obtained using permeability values obtained from in situ tests.

Annex G

(informative)

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

(1) For weak and broken rocks with tight joints, including chalk with porosity less than 35 %, presumed bearing resistance may be derived from Figure G.1. This is based on the grouping given in Table G.1 with the assumption that the structure can tolerate settlements equal to 0,5 % of the foundation width. Values of presumed bearing resistance for other settlements may be derived by direct proportion. For weak and broken rocks with open or infilled joints, reduced values of presumed bearing pressure should be used.

Table G.1 Grouping of weak and broken rocks

Group	Type of rock
1	Pure limestones and dolomites Carbonate sandstones of low porosity
2	Igneous Oolitic and marly limestones Well cemented sandstones Indurated carbonate mudstones Metamorphic rocks, including slates and schists (flat cleavage/foliation)
3	Very marly limestones Poorly cemented sandstones Slates and schists (steep cleavage/foliation)
4	Uncemented mudstones and shales

Fig. G.1 Presumed bearing resistance for square pad foundations bearing on rock (for settlements not exceeding 0,5 % of foundation width). For types of rock in each of four groups, see Table G.1. Presumed bearing resistance in hatched areas to be assessed after inspection and/or making tests on rock
(from: BS 8004)

Annex H
(informative)
**Limiting values of structural deformation
and foundation movement**

(1) The components of foundation movement which should be considered include settlement, relative (or differential) settlement, rotation, tilt, relative deflection, relative rotation, horizontal displacement and vibration amplitude. Definitions of some terms for foundation movement and deformation are given in Figure H.1

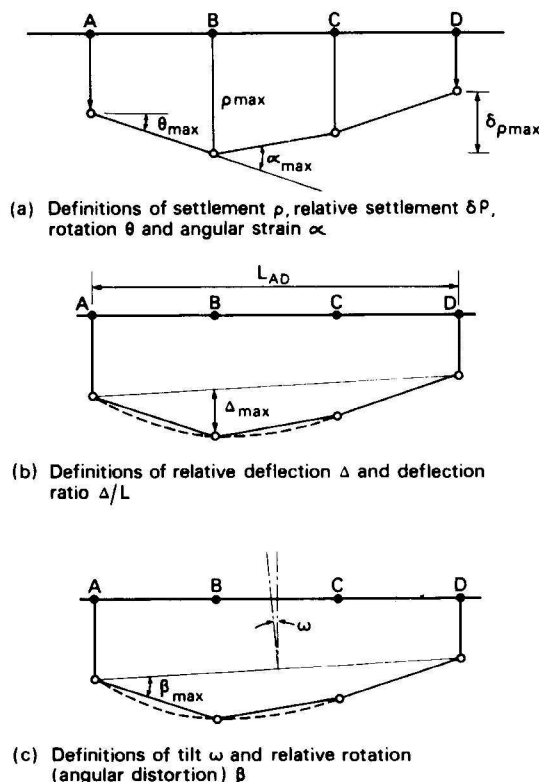


Fig. H.1 Definitions of foundation movement

(2) The maximum acceptable relative rotations for open framed structures, infilled frames and load bearing or continuous brick walls are unlikely to be the same but are likely to range from about 1/2000 to about 1/300, to prevent the occurrence of a serviceability limit state in the structure. A maximum relative rotation of 1/500 is acceptable for many structures. The relative rotation likely to cause an ultimate limit state is about 1/150.

(3) The ratios given in H.2 apply to a sagging mode, as illustrated in fig.H.1. For a hogging mode (edge settling more than part between), the value should be halved.

(4) For normal structures with isolated foundations, total settlements up to 50 mm are often acceptable. Larger settlements may be acceptable provided the relative rotations remain within acceptable limits and provided the total settlements do not cause problems with the services entering the structure, or cause tilting etc.

(5) These guidelines concerning limiting settlements apply to normal, routine structures. They should not be applied to buildings or structures which are out of the ordinary or for which the loading intensity is markedly non-uniform.

Annex J
(informative)
**Checklist for construction supervision
and performance monitoring**

J.1 General

The list which follows contains the more important items that should be considered when supervising construction or monitoring the performance of the completed structure. The importance of the items will vary from project to project. The list is not exhaustive. Items which refer to specific aspects of geotechnical engineering or to specific types of works have been reported in the chapters of this code.

J.2 Construction supervision

J.2.1 General items to be checked

- (1) Verification of ground conditions and of the location and general lay-out of the structure.
- (2) Groundwater flow and pore pressure regime; effects of dewatering operations on groundwater table; effectiveness of measures taken to control seepage inflow; internal erosion processes and piping; chemical composition of groundwater; corrosion potential.
- (3) Movements, yielding, stability of excavation walls and base; temporary support systems; effects on nearby buildings and utilities; measurement of soil pressures on retaining structures; measurement of pore water variations resulting from excavation or loading.
- (4) Safety of workmen with due consideration of geotechnical limit states.

J.2.2 Water flow and pore pressures

- (1) Adequacy of system to ensure control of pore water pressures in all aquifers where excess pressure could affect stability of slopes or base of excavation, including artesian pressures in an aquifer beneath the excavation; disposal of water from dewatering systems; depression of groundwater table throughout entire excavation to prevent boiling or quick conditions, piping and disturbance of formation by construction equipment; diversion and removal of rainfall or other surface waters.
- (2) Efficient and effective operation of dewatering system throughout the entire construction period considering encrusting of well screens, silting of wells or sumps; wear in pumps; clogging of pumps.
- (3) Control of dewatering to avoid disturbance of adjoining structures or areas; observations of piezometric levels; effectiveness, operation and maintenance of water recharge systems if required.
- (4) Settlement of adjoining structures or areas.
- (5) Effectiveness of subhorizontal borehole drains.

J.3 Performance monitoring

(1) Settlement at established time intervals of buildings and other structures including those due to effects of vibrations, metastable soils.

(2) Lateral displacement, distortions especially those related to fills and stockpiles; soil supported structures, such as buildings or large tanks; deep excavation channels.

(3) Piezometric levels under buildings or in adjoining areas, especially if deep drainage or permanent dewatering systems are installed or if deep basements are constructed.

(4) Deflection or displacement of retaining structures considering: normal backfill loadings; effects of stockpiles, fills or other surface loadings; water pressures.

(5) Flow measurements from drains.

(6) Special problems.

- High temperature structures such as boilers, hot ducts, etc.; desiccation of clay or silt soils: monitoring of temperatures; movements.
- Low temperature structures, such as cryogenic installations or refrigerated areas: monitoring of temperature; ground freezing; frost heave, displacement; effects of subsequent thawing.

(7) Watertightness.

(8) Vibration measurements.