DRAFT FOR DEVELOPMENT

DD ENV 1997-1:1995

Eurocode 7: Geotechnical design —

Part 1: General rules —

(together with United Kingdom National Application Document)

Committees responsible for this Draft for Development

The preparation of the National Application Document for use in the UK with ENV 1997-1:1994 was entrusted to Technical Committee B/526, Geotechnics, upon which the following bodies were represented:

Association of Consulting Engineers
Association of Geotechnical Specialists
Department of the Environment (Property and Buildings Directorate)
Department of Transport
Federation of Civil Engineering Contractors
Federation of Piling Specialists
Institution of Civil Engineers
Institution of Structural Engineers

This Draft for Development, having been prepared under the direction of the Sector Board for Building and Civil Engineering, was published under the authority of the Standards Board and comes into effect on 15 July 1995

The following BSI reference relates to the work on this Draft for Development: Committee reference B/526

ISBN 0 580 24511 X

Amendments issued since publication

Amd. No.	Date	Comments

Contents

National foreword

This publication comprises the English language version of ENV 1997-1:1994 *Eurocode 7: Geotechnical design — Part 1: General rules*, as published by the European Committee for Standardization (CEN) plus the National Application Document (NAD) to be used with the ENV for design of foundations and geotechnical structures to be constructed in the United Kingdom.

ENV 1997-1 results from a programme of work sponsored by the European Commission to make available a common set of rules. The full range of codes covers the basis of design and actions, the design of structures in concrete, steel, composite construction, aluminium, timber and masonry, and geotechnics and seismic design.

An ENV is made available for provisional application during a period of trial use of 3 years, but does not have the status of a fully agreed European Standard (EN). At the end of the trial period the aim is to use the experience gained to modify the ENV so that it can be approved as an EN.

The values of some of the parameters in the ENV Eurocodes may be set by member states so as to meet the requirements for safety in national regulations. The values to be used in the UK are given in clause **4** of this NAD.

The NAD contains references to alternative, supporting documents, pending the publication of relevant European Standards. These references are given in Annex A of this NAD.

The purpose of the NAD is to provide essential information, particularly in relation to safety, necessary for provisional application of the ENV and it therefore constitutes an essential part of this publication in the UK. The recommendations of the NAD take precedence in the UK over the corresponding provisions in the ENV.

Compliance with ENV 1997-1 and the NAD does not in itself confer immunity from legal obligations.

Users of this document are invited to comment on its technical content, ease of use and any ambiguities or anomalies. These comments will be taken into account when preparing the UK national response to CEN on the question of whether the ENV can be converted to an EN.

Comments should be made in writing to BSI, 389 Chiswick High Road, London W4 4AL, quoting this document, the reference to the relevant clause, and if possible, a proposed revision, within 2 years of the issue of this document.

Textual errors. When implementing the English language version of ENV 1997-1:1994 as the national prestandard, the textual errors listed below were discovered. They have been reported to CEN in a proposal to amend the text of the European Prestandard.

In line 1 of item 4 in **6.5.3** "(**6.5**)" should read "(**6.2**)".

In line 3 of item 3 in **6.6** "(**2.4.5**)" should read "(**2.4.6**)".

In line 4 of item 4 in **8.8.5** "pile" should read "anchorage".

In equation G.2 of Annex G "a" should read "a'".

Summary of pages

This document comprises a front cover, an inside front cover, pages i and ii, the National Application Document title page, pages ii to x, the ENV title page, pages 2 to 88 and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

National Application Document for use in the UK with ENV 1997-1:1994

Contents of National Application Document

Introduction

This National Application Document has been prepared by Technical Committee B/526 to enable ENV 1997-1 (Eurocode 7-1) to be used for the design of geotechnical structures to be constructed in the United Kingdom. It has been developed from:

a) a textual examination of ENV 1997-1; and

b) trial calculations, including parametric calibration against relevant UK codes and standards, to assess its ease of use and to provide numerical factors that produce designs in general conformity with UK practice.

1 Scope

This National Application Document (NAD) provides information required to enable ENV 1997-1 to be used for most routine designs for geotechnical structures that are to be constructed in the UK.

2 References

2.1 Normative references

This National Application Document incorporates, by dated or undated reference, provisions from other publications. These normative references are made at the appropriate places in the text and the cited publications are listed on the inside back cover. For dated references, only the edition cited applies: any subsequent amendments to or revisions of the cited publication apply to this National Application Document only when incorporated in the reference by amendment or revision. For undated references, the latest edition of the cited publication applies, together with any amendments.

2.2 Informative references

This National Application Document refers to other publications that provide information or guidance. Editions of these publications current at the time of issue of this standard are listed on the inside back cover, but reference should be made to the latest editions.

3 Definitions

For the purposes of this National Application Document the following definitions apply.

NOTE ENV 1997-1 uses terminology that may not be wholly familiar to UK engineers, such as “action” and “execution”. Definitions of these terms may be found in ENV 1991-1 and ENV 1997-1 and are reproduced here for convenience.

3.1
action

force (load) applied to the structure (direct action); or an imposed or constrained deformation (indirect action)

NOTE For example caused by temperature changes, moisture variation or uneven settlement.

3.2
execution

activity of creating a building or civil engineering works

NOTE The term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.

4 Values of partial factors

a) In this clause, values of partial factors currently do not differ from those used in ENV 1997-1.

NOTE In the state of development of this NAD at July 1995, no deviations from boxed values are proposed.

b) *Clause **2.4.2 (14) P*** Table 2.1 should be replaced by Table 1 of this NAD. For accidental situations all numerical values of partial factors for actions should be 1.0.

c) *Clause **7.6.3.2 (6) P*** For the derivation of the ultimate characteristic bearing resistance of piles in compression, the factors to be used should be those given in Table 2 of this NAD, which should be substituted for Table 7.1.

d) *Clause **7.6.3.2 (10) P*** For the derivation of the design ultimate bearing resistance of piles in compression, the values of partial factors should be those given in Table 3 of this NAD, which should be substituted for Table 7.2.

e) *Clause **7.6.3.3*** The value 1.5 should be substituted for the bracketed value [1.5].

f) *Clause* ***7.7.2.2*** *(2) P* For the derivation from pile load tests of ultimate characteristic values of the resistance of piles in tension, the values of partial factors to be applied to the measured ultimate tensile resistance should be those given in Table 4 of this NAD, which should be substituted for Table 7.3.

g) *Clause* ***7.7.2.2*** *(4) P* The factor to derive the design value from the characteristic value should be 1.6.

h) *Clause* ***8.8.5*** *(4) P* For the derivation from assessment tests of the ultimate characteristic resistance of anchorages, the values of partial factors should be those given in Table 5 of this NAD, which should be substituted for Table 8.1.

i) *Clause* ***8.8.5*** *(5) P* For the derivation from characteristic resistance of the design resistance of anchorages, the value of partial factor should be 1.25 for temporary anchorages and 1.5 for permanent anchorages.

Table 1 — Partial factors — ultimate limit states in persistent and transient situations

Case	Actions			Ground properties			
	Permanent		Variable				
	Unfavourable	Favourable	Unfavourable	tan φ	c'	c_u	q_u[a]
A	1.0	0.95	1.5	1.1	1.3	1.2	1.2
B	1.35	1.0	1.5	1.0	1.0	1.0	1.0
C	1.0	1.0	1.3	1.25	1.6	1.4	1.4

[a] Compressive strength of soil or rock.

Table 2 — Factors to derive the ultimate characteristic bearing resistance

	Number of load tests		
	1	2	> 2
Factor ξ on mean R_{cm}	1.5	1.35	1.3
Factor ξ on lowest R_{cm}	1.5	1.25	1.1

Table 3 — Factors to derive the ultimate design bearing resistance

Component factors	γ_b	γ_s	γ_t
Driven piles	1.3	1.3	1.3
Bored piles	1.6	1.3	1.5
CFA piles	1.45	1.3	1.4

Table 4 — Factors to derive characteristic ultimate tensile pile resistance from tests

	Number of load tests		
	1	2	> 2
Factor ξ on mean R_m	1.5	1.35	1.3
Factor ξ on lowest R_m	1.5	1.25	1.1

Table 5 — Factors to derive ultimate characteristic resistance from anchorage tests

	Number of assessment tests		
	1	2	> 2
Factor ξ on mean R_{am}	1.5	1.35	1.3
Factor ξ on lowest R_{am}	1.5	1.25	1.1

5 Reference standards

ENV 1997-1 does not call up supporting standards since none yet exist (see Annex A).

6 Additional recommendations

Annex B lists points that should be noted when designing to ENV 1997-1.

Annex A (informative) References to supporting standards in Eurocode 7

Table A.1 provides guidance on British Standards which support clause references in EC7 on standards for site investigation and laboratory and field testing.

Table A.1 — References in EC 7 to other codes and standards

Reference location in EC7	Reference	UK equivalent document no.
3.1 (3)P	"internationally recognized standards and recommendations"	BS 5930 BS 1377-1 to BS 1377-9
3.2.3 (6)P	"standardized procedures"	BS 5930 BS 1377-1 to BS 1377-9
3.3.4 **3.3.5**	"standard laboratory procedures"	BS 1377-1 to BS 1377-9

Annex B (normative) Additional recommendations

B.1 General

For the geotechnical design of trunk roads and motorways within the UK, reference should be made to the following documents of the Department of Transport, the Scottish Office Industry Department, the Welsh Office: Y Swyddfa Gymreig and the Department of the Environment for Northern Ireland:

a) *Design Manual for Roads and Bridges* [1];

b) *Manual for Contract Documents for Highway Works*, Volumes 1 and 2 [2].

B.2 Guidance on EC7

a) *Clause **2.4.2***

In **2.4.2** (12)P Case B, care is required particularly in the selection of the partial factor γ_F when the design of structural elements is critical. Whether earth pressures acting on the structure are considered to be favourable or unfavourable will substantially affect the outcome (for example, in the design of a cantilever retaining structure both the active and passive pressure distributions will usually be unfavourable to the design of the structural section). The application rules in **2.4.2** (17) are particularly important; the same value of γ_F (1.35 or 1.0) is applied to all earth and water pressures, depending on whether the combined effect of them all is favourable or unfavourable.

The bending moments and shear forces derived from factored earth pressures should be regarded as design values when using EC2 or other of the structural Eurocodes for the structural calculations.

If the application of $\gamma_F = 1.35$ leads to a physically unreasonable situation, then the factor (1.35) should be treated as a model factor and applied to the action effects (e.g. resultant bending moment and shear force) which are then treated as design values in EC2 or other of the structural Eurocodes.

b) *Clause **5***

Reference should be made to BS 6031, BS 8006[1)] and the forthcoming CEN documents from CEN/TC 288/WG6 *Execution of special geotechnical works — Grouting* and CEN/TC 288/WG7 *Execution of special geotechnical works — Jet grouting.*

c) *Clause **6***

Reference should be made to BS 8004.

d) *Clause* **7**

Reference should be made to BS 8004, BS 5573 and to the forthcoming CEN documents from CEN/TC288/WG3 *Execution of special geotechnical works — Bored piles*, CEN/TC 288/WG4 *Execution of special geotechnical works — Sheet pile walls*, CEN/TC 288/WG5 *Execution of special geotechnical works — Displacement piling.*

[1)] In preparation.

e) *Clause* ***7.6.3.3***

The adhesion factor a provides a correlation between the results of static load tests and ground test results. If the values adopted for a and c_u are such that the resulting value of shaft adhesion $a \cdot c_u$ represents the mean of the test results, then the shaft resistance will be given by:

$q_{sik} = a \cdot c_u/1.5$

then

$R_{sd} = R_{sk}/\gamma_s = \Sigma q_{sik} \cdot A_{si}/\gamma_s = \Sigma a \cdot c_u \cdot A_{si}/(1.5 \cdot \gamma_s)$.

With

$\gamma_s = 1.3$, the factor on $a \cdot c_u \cdot A_{si}$ becomes 1.95.

f) *Clause* ***8***

Reference should be made to BS 8002, BS 8006 and to the forthcoming document from CEN/TC288/WG2 *Execution of special geotechnical works — Ground anchors*.

g) *Clause* ***9***

Reference should be made to BS 6031.

blank

List of references (see clause **2**)

Normative references

BSI publications

BRITISH STANDARDS INSTITUTION, London

BS 5573:1978, *Code of practice for safety precautions in the construction of large diameter boreholes for piling and other purposes.*

BS 6031:1981, *Code of practice for earthworks.*

BS 8002:1994, *Code of practice for earth retaining structures.*

BS 8004:1986, *Code of practice for foundations.*

BS 8006, *Code of practice for strengthened/reinforced soils and other fills*[2)].

Other references

[1] *Design Manual for Roads and Bridges*[3)].

[2] *Manual for Contract Documents for Highway Works*, Vols 1 and 2[3)].

Informative references

BSI publications

BRITISH STANDARDS INSTITUTION, London

BS 1377, *Methods of test for soils for civil engineering purposes.*

BS 1377-1:1990, *General requirements and sample preparation.*

BS 1377-2:1990, *Classification tests.*

BS 1377-3:1990, *Chemical and electro-chemical tests.*

BS 1377-4:1990, *Compaction-related tests.*

BS 1377-5:1990, *Compressibility, permeability and durability tests.*

BS 1377-6:1990, *Consolidation and permeability tests in hydraulic cells and with pore pressure measurement.*

BS 1377-7:1990, *Shear strength tests (total stress).*

BS 1377-8:1990, *Shear strength tests (effective stress).*

BS 1377-9:1990, *In-situ tests.*

BS 5930:1981, *Code of practice for site investigations.*

CEN publication

EUROPEAN COMMITTEE FOR STANDARDIZATION (CEN), Brussels

ENV 1991-1:1994, *Eurocode 1 — Basis of design and actions on structures — Part 1: Basis of design.*

[2)] In preparation.
[3)] Published by and available from HMSO.

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EUROPEAN PRESTANDARD
PRÉNORME EUROPÉENNE
EUROPÄISCHE VORNORM

ENV 1997-1

October 1994

ICS 91/060.00;91.120.20

Descriptors: Soils, computation, buildings codes, rules of calculation

English version

Eurocode 7: Geotechnical design — Part 1: General rules

Eurocode 7: Calcul geotechnique —
Partie 1: Règles générales

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

This European Standard (ENV) was approved by CEN on 1993-05-25 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into a European Standard.

CEN members are required to announce the existance of this ENV in the same way as for an EN and to make the ENV available promptly at national level in an appropriate form. It is permissible to keep conflicting national standards in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

CEN members are the national standards bodies of Austria, Belgium, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

CEN

European Committee for Standardization
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B-1050 Brussels

Ref. No. ENV 1997-1:1994 E

Foreword

1 Objectives of the Eurocodes

(1) The structural Eurocodes comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.

(2) They are intended to serve as reference documents for the following purposes:

a) As a means to prove compliance of building and civil engineering works with the essential requirements of the Construction Products Directive (CPD)

b) As a framework for drawing up harmonised technical specifications for construction products.

(3) They cover execution and control only to the extent that is necessary to indicate the quality of the construction products, and the standard of the workmanship, needed to comply with the assumptions of the design rules.

(4) Until the necessary set of harmonised technical specifications for products and for methods of testing their performance is available, some of the Structural Eurocodes cover some of these aspects in informative annexes.

2 Background to the Eurocode programme

(1) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonised technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various Member States and would ultimately replace them. These technical rules became known as the "Structural Eurocodes".

(2) In 1990, after consulting their respective Member States, the CEC transferred work of further development, issue and updates of the Structural Eurocodes to CEN and the EFTA Secretariat agreed to support the CEN work.

(3) CEN Technical Committee CEN/TC 250 is responsible for all Structural Eurocodes.

3 Eurocode programme

(1) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:

EN 1991, Eurocode 1: Basis of design and actions on structures.

EN 1992, Eurocode 2: Design of concrete structures.

EN 1993, Eurocode 3: Design of steel structures.

EN 1994, Eurocode 4: Design of composite steel and concrete structures.

EN 1995, Eurocode 5: Design of timber structures.

EN 1996, Eurocode 6: Design of masonry structures.

EN 1997, Eurocode 7: Geotechnical design.

EN 1998, Eurocode 8: Design of structures for earthquake resistance.

EN 1999, Eurocode 9: Design of aluminium alloy structures.

(2) Separate sub-committees have been formed by CEN/TC 250 for the various Eurocodes listed above.

(3) This part of the Structural Eurocode for Geotechnical design which had been finalised and approved for publication under the direction of CEC, is being issued by CEN as a European Prestandard (ENV) with an initial life of three years.

(4) This Prestandard is intended for experimental practical application in the design of the building and civil engineering works covered by the scope as given in **1.1.2** and for the submission of comments.

(5) After approximately two years CEN members will be invited to submit formal comments to be taken into account in determining future action.

(6) Meanwhile, feedback and comments on this Prestandard should be sent to the Secretariat of sub-committee CEN/TC250/SC7 at the following address:

NNI

P.O.Box 5059

NL-2600 GB Delft

The Netherlands

or to a national standards organisation.

4 National application documents

(1) In view of the responsibilities of authorities in member countries for the safety, health and other matters covered by the essential requirements of the CPD, certain safety elements in this ENV have been assigned indicative values which are identified by [ ]. The authorities in each member country are expected to assign definitive values to these safety elements.

(2) Many of the supporting standards, including those giving values for actions to be taken into account and measures required for fire protection, will not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document giving definitive values for safety elements, referencing compatible supporting standards and giving national guidance on the application of this Prestandard will be issued by each Member State or its Standard Organisation. This Prestandard should be used in conjunction with the National Application Document valid in the country where the building and civil engineering works is to be constructed.

It is intended that this Prestandard is used in conjunction with the NAD valid in the country where the building or civil engineering works are located.

5 Matters specific to this prestandard

(1) The scope of eurocode 7 is defined in **1.1.1** and the scope of this Part of eurocode 7 is defined in **1.1.2**. Additional Parts of Eurocode 7 which are planned are indicated in **1.1.3**; these will cover additional technologies or applications, and will complement and supplement this Part.

(2) In using this Prestandard in practice, particular regard should be paid to the underlying assumptions and conditions given in **1.3**.

(3) The nine chapters of this Prestandard are complemented by seven annexes which have informative status.

Contents

Section 1. General

1.1 Scope

1.1.1 Scope of Eurocode 7

(1)P This prestandard applies 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**.

(2)P This prestandard is concerned with the requirements for strength, stability, serviceability and durability of the structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

(3)P This prestandard shall be used in conjunction with ENV 1991-1 "Basis of Design" of Eurocode 1 "Basis of Design and Actions on Structures" which establishes the principles and requirements for safety and serviceability, describes the basis for design and verification and gives guidelines for related aspects of structural reliability.

(4)P This prestandard gives the rules to calculate actions originating from the ground such as earth pressures. Numerical values of actions on buildings and civil engineering works to be taken into account in the design are provided in ENV 1991 Eurocode 1 "Basis of Design and Actions on Structures" applicable to the various types of construction.

(5)P In this prestandard execution is covered to the extent that is necessary to indicate the quality of the construction materials and products which should be used and the standard of workmanship on site needed to comply with the assumptions of the design rules. Generally, the rules related to execution and workmanship are to be considered as minimum requirements which may have to be further developed for particular types of buildings or civil engineering works and methods of construction.

(6)P This prestandard does not cover the special requirements of seismic design. Eurocode 8, "Designprovisions for earthquake resistance of structures" provides additional rules for seismic design which complete or adapt the rules of this prestandard.

1.1.2 Scope of ENV 1997-1

(1)P This prestandard gives a general basis for the geotechnical aspects of the design of buildings and civil engineering works.

(2)P The following subjects are dealt with in ENV 1997-1 Eurocode 7: "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: Retaining Structures*;

— *Section 9: Embankments and Slopes*.

1.1.3 Further Parts of Eurocode 7

(1)P This prestandard will be supplemented by further Parts which will complete or adapt it for particular aspects of special types of buildings and civil engineering works, special methods of construction and certain other aspects of design which are of general practical importance.

1.2 References

This European Prestandard 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 Prestandard only when incorporated in it by amendment or revision.

ISO 1000:1981, *SI-units and Recommendations for the use of their multiples and of certain other units.*

ISO 3898:1987, *Bases for design of structures. Notations. General symbols.*

1.3 Distinction between, Principles and Application Rules

(1)P Depending on the character of the individual clauses, distinction is made in this prestandard between Principles and Application Rules.

(2)P The Principles comprise:

— general statements and definitions for which there is no alternative, as well as;

— requirements and analytical models for which no alternative is permitted unless specifically stated.

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

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

(5)P It is permissible to use alternative rules different from the Application Rules given in this Eurocode, provided it is shown that the alternative rules accord with the relevant Principles.

1.4 Assumptions

(1)P The following assumptions apply:

— data required for design are collected, recorded and interpreted;

— 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 is 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 Eurocode or in the relevant material or product specifications;

— the structure will be adequately maintained;

— the structure will be used in accordance with the purpose defined for the design.

1.5 Definitions

1.5.1 Terms common to all Eurocodes

(1)P The terms used in common for all Eurocodes are defined in ENV 1991-1 Basis of design.

1.5.2 Special terms used in ENV 1997-1

(1)P The following terms are used in ENV 1997-1 with the following meanings:

— **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;

— **ground**: soil, rock and fill existing in place prior to the execution of the construction works;

— **structure**: as defined in ENV 1991-1 “Basis of design”, including fill placed during execution of the construction works;

1.6 S.I. units

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

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

— forces	kN, MN
— moments	kNm
— mass density	kg/m^3, Mg/m^3, (t/m^3)
— unit weight	kN/m^3
— stresses, pressures and strengths	kN/m^2 (kPa)
— stiffness	MN/m^2 (MPa)

— coefficient of permeability m/s, (m/year)
— coefficient of consolidation m^2/s, (m^2/year)

1.7 Symbols common to all Eurocodes

(1)P The symbols used in common for all Eurocodes are defined in ENV 1991-1 “Basis of design”.

1.8 Symbols used in Eurocode 7

(1)P The symbols commonly used in ENV 1997-1 are defined in the following sections. Other symbols are defined where they are used locally in the text. The notation of the symbols used is based on ISO 3898:1987.

1.8.1 Latin upper case letters

B width
D diameter
F axial or transverse load on pile
H horizontal action or force
K earth pressure coefficient
N bearing resistance factor
R vertical resistance (in units of force) of a foundation element
V vertical action or force

1.8.2 Latin lower case letters

a adhesion
c' cohesion intercept in terms of effective stress
c_u undrained shear strength
i hydraulic gradient
k permeability
q overburden or surcharge pressure
s settlement
u pore water pressure

1.8.3 Greek lower case letters

γ unit weight
δ angle of shearing resistance between ground and structure
σ total normal stress
σ' effective normal stress
τ shear stress
ϕ angle of shearing resistance
ϕ' angle of shearing resistance in terms of effective stress

1.8.4 Subscripts

a	active earth pressure
a	anchor
b	base of pile
c	compression
d	design value
k	characteristic value
p	passive earth pressure
s	shaft of pile
t	tensile
t	total
tr	transverse
w	water
o	at rest or initial condition

Section 2. Basis of geotechnical design

2.1 Design requirements

(1)P A structure shall be designed in compliance with the general design principles given in ENV 1991-1 Eurocode 1 "Basis of Design".

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

— light and simple structures and small earthworks 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 for property and life;

— other geotechnical structures.

(3) For projects of low geotechnical complexity and risk, such as defined above, simplified design procedures are acceptable.

(4)P The following factors shall be taken into consideration when determining the geotechnical design requirements:

— nature and size of the structure and its elements, including any special requirements;

— conditions with regard to its surroundings (neighbouring structures, traffic, utilities, vegetation, hazardous chemicals, etc.);

— ground conditions;

— groundwater situation;

— regional seismicity;

— influence of the environment (hydrology, surface water, subsidence, seasonal changes of moisture).

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

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 eventually changed at each stage of the design and construction process.

The various design aspects of a project may require treatment in different geotechnical categories. It is not necessary to treat the whole of the project according to the highest of these categories.

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

Geotechnical Category 1

This category only includes 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 for property and life.

Geotechnical Category 1 procedures will only be sufficient in ground conditions which are known from comparable experience to be sufficiently straight-forward that routine methods may be used for foundation design and construction.

Geotechnical Category 1 procedures will be 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 straight-forward.

The following are examples of structures or parts of structures complying with Geotechnical Category 1:

— simple 1 and 2 storey houses and agricultural buildings with a maximum design column load of 250 kN and 100 kN/m for walls and using conventional types of spread and piled foundations;

— retaining walls and excavation supports where the difference in ground levels does not exceed 2m;

— small excavations for drainage works, pipe-laying, etc.

Geotechnical Category 2

This category includes conventional types of structures and foundations with no abnormal risks or unusual or exceptionally difficult ground or loading conditions. Structures in Geotechnical Category 2 require quantitative geotechnical data and analysis to ensure that the fundamental requirements will be satisfied, but routine procedures for field and laboratory testing and for design and execution may be used.

The following are examples of structures or parts of structures complying with Geotechnical Category 2:

Conventional types of:

— spread foundations;

— raft foundations;

— piled 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.

Geotechnical Category 3

This category includes structures or parts of structures which do not fall within the limits of Geotechnical Categories 1 and 2.

Geotechnical Category 3 includes very large or unusual structures, structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions and structures in highly seismic areas.

(6)P For each geotechnical design situation it shall be verified that no relevant limit state is exceeded.

(7) This design requirement may be achieved by:

— 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**.

These four approaches may be used in combination. 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.

(8)P The interaction between structure and ground shall be considered.

(9) Compatibility of strains in the materials involved at a limit state should be considered, especially for materials which are brittle or which have strain-softening properties. Examples include over-reinforced concrete, dense granular soils, cemented soils and clays which exhibit low residual strength. 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.

(10)P Buildings shall be protected against the penetration of groundwater or the transmission of vapour or gases to the inner surfaces.

(11)P When possible the design results shall be checked against comparable experience.

2.2 Design situations

(1)P In geotechnical design the detailed specifications of design situations shall include, as appropriate:

— the general suitability of the ground on which the structure is located;

— the disposition and classification of the various zones of soil, rock and elements of construction which are involved in the calculation model;

— dipping bedding planes;

— mine workings, caves or other underground structures;

— in the case of structures resting on or near rock, the following shall be included:

— interbedded hard and soft strata;

— faults, joints and fissures;

— solution cavities, such as swallow holes or fissures filled with soft material, and continuing solution processes;

— the actions, their combinations and load cases;

— 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;

— variations in groundwater levels, including the effects of dewatering, possible flooding, failure of drainage systems, 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;

— subsidence due to mining or other causes;

— the tolerance of the structure to deformations;

— the effect of the new structure on existing structures or services.

2.3 Durability

(1)P In geotechnical design the internal and external environmental conditions shall be estimated at the design stage to assess their significance in relation to durability and to enable provisions to be made for the protection or adequate resistance of the materials.

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

— for concrete: aggressive agents, such as acidic conditions or sulphate salts, in the ground water;

— 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; pitting type of corrosive attack to 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;

— for timber: fungi and aerobic bacteria in the presence of oxygen;

— for synthetic fabrics: the aging effects of UV exposure or ozone degradation or the combined effects of temperature and stress, secondary effects due to chemical degradation.

2.4 Geotechnical design by calculation

2.4.1 Introduction

(1)P Design by calculation shall be in accordance with section 9 "Verification by the partial factor method" in ENV 1991-1 Eurocode 1 "Basis of Design". This method involves:

— calculations models;

— actions, which may be either imposed loads or imposed displacements;

— properties of soils, rocks and other materials;

— geometrical data;

— limiting values of deformations, crackwidth, vibrations etc.

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

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

(4) Limit states involving the formation of a mechanism in the ground are readily checked using this approach. For limit states defined by deformation considerations, the deformations should be calculated or otherwise assessed if this approach is used.

(5)P Calculation models shall consist of:

— a method of analysis, often based on an analytical model including simplifications;

— if needed, a modification to the results of the analysis to ensure that the results of the design calculation model are either accurate or err on the side of safety.

(6) The modification to the results of the analysis should take account of the following factors:

— the range of uncertainty in the results of the method of analysis on which the design calculation model is based;

— any systematic errors known to be associated with the method of analysis.

(7)P When no reliable calculation model is available for a specific limit state, analysis of the other limit states shall be carried out using factors to ensure that this limit state is sufficiently improbable.

(8)P Whenever possible, the calculation model shall be correlated with field observations of previous designs, model tests or more reliable analyses.

(9) The calculation model may consist of an empirical relationship between test results and design requirements, used in place of an analytical model. In this case the empirical relationship shall be clearly established for the relevant ground conditions.

2.4.2 Actions in geotechnical design

(1)P For any calculation the values of actions are known quantities. Actions are not unknowns in the calculation model.

(2)P Before any calculation is carried out, the designer shall choose the forces and imposed displacements which will be treated as actions in that calculation. Some forces and imposed displacements shall be treated as actions in certain calculations and not in others. Downdrag (negative skin friction) and earth pressures are examples of such forces.

(3) For loads applied to foundations by structures, an analysis of the interaction between the structure and the ground may be needed in order to determine the actions to be adopted in the design of foundations.

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

— the weights of soil, rock and water;

— in situ stresses in the ground;

— free water pressures;

— ground water pressures;

— seepage forces;

— dead, imposed and environmental loads from structures;

— surcharges;

— mooring forces;

— removal of load or excavation of ground;

— traffic loads;

— movements caused by mining;

— swelling and shrinkage caused by vegetation, climate or moisture changes;

— movements due to creeping or sliding soil masses;

— movements due to degradation, decomposition, self-compaction and solution;

— movements and accelerations caused by earthquakes, explosions, vibrations and dynamic loads;

— temperature effects, including frost heave;

— ice loading;

— imposed prestress in ground anchors, or struts.

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

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

(7)P Actions which are applied cyclically with high frequency shall be identified for special consideration with regard to dynamic effects.

(8)P Special consideration shall be given to the safety evaluation of a geotechnical structure where hydrostatic forces are the predominant forces. This is due to the fact that deformations, fissuring and variable permeability with inherent risk of erosion may give rise to changes in the level of the water table which could be vitally important to the safety.

(9)P 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)P For limit states with severe consequences (generally ultimate limit states), design values for water pressures and seepage forces shall represent the most unfavourable values which could occur in extreme circumstances. 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.

(11) The risk of unfavourable water levels may be caused by changes in the water catchment, and reduced drainage possibilities (owing to blockage or freezing), etc. should be considered.

Unless the adequacy of the drainage system can be demonstrated and its maintenance ensured, it will often be necessary to assume that the groundwater table could rise to ground level in extreme circumstances. In some cases this could be considered as an accidental action.

(12)P The design shall be verified for each of the three Cases, A, B and C separately as relevant.

(13) Cases A, B and C have been introduced in order to ensure stability and adequate strength in the structure and in the ground in accordance with Table 9.2 of ENV 1991-1, Eurocode 1 Basis of Design.

(14)P The values of partial factors for Permanent and variable actions given in Table 2.1 shall generally be used for verification of ultimate limit states of conventional types of structures and foundations in persistent and transient situations. More severe values shall be considered in cases of abnormally great risk or unusual or exceptionally difficult ground or loading conditions. Where it can be justified on the basis of the possible consequences, less severe values may be used for temporary structures or transient situations. For accidental situations all numerical values of partial factors for actions shall be taken equal to [1,0].

Table 2.1 — Partial factors — ultimate limit states in persistent and transient situations

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

[a] Compressive strength of soil or rock.

(15) For ground properties, different partial material factors are to be used with Cases A, B and C, (see **2.4.3** and Table 2.1)

Where it is clear that one of the three cases is most critical to the design, it will not be necessary to carry out calculations for other cases. However, different cases may be critical to different aspects of a design.

In this prestandard, Case A is only relevant to buoyancy problems, where hydrostatic forces comprise the main unfavourable action. The values given in Table 2.1 are only valid for such situations. For buoyancy problems it is often more appropriate to use a structural solution (e.g. overflow arrangements) associated with partial factor values close to unity, rather than to rely on larger values which are less appropriate.

Case B is often critical to the design of the strength of structural elements involved in foundations or retaining structures. Where there is no strength of structural materials involved, Case B is irrelevant.

Case C is generally critical in cases, such as slope stability problems, where there is no strength of structural elements involved. Case C is often critical to the sizing of structural elements involved in foundations or retaining structures, and sometimes to the strength of structural elements. Where there is no strength of ground involved in the verification, Case C is irrelevant.

The design strengths of the structural materials and the ground will not necessarily both be fully mobilised in the same case.

In structural design of elements such as footings, piles, retaining walls, etc., a model factor γ_{sd} may be introduced as relevant.

(16)P Permanent actions shall include self weight of structural and non-structural components and those actions caused by ground, groundwater and free water.

(17) In calculation of design earth pressures for Case B, the partial factors given in Table 2.1 are applied to characteristic earth pressures. Characteristic earth pressures comprise characteristic water pressures together with stresses which are admissible in relation to the characteristic ground properties and characteristic surface loads.

All permanent characteristic earth pressures on both sides of a wall are multiplied by [1.35] if the total resulting action is unfavourable and by [1.00] if the total resulting action effect is favourable. Thus, all characteristic earth pressures are treated as being derived from a single source as defined in ENV 1991-1.

In some situations, the application of partial factors to characteristic earth pressures could lead to design values which are unreasonable or even physically impossible. In these situations, the partial factors for actions given in Table 2.1 may be treated as model factors. They are then applied directly to the action effects (i.e. internal structural forces and bending moments) derived from characteristic earth pressures.

In calculation of design earth pressures for Case C, the partial factors given in Table 2.1 are applied to the characteristic strength of the ground and to the characteristic surface loads.

(18)P For the verification of serviceability limit states, partial factors of unity shall be used for all permanent and variable actions except where specified otherwise.

(19)P Design values of actions due to ground and groundwater may also be derived by methods other than the use of partial factors. The partial factors set out in Table 2.1 indicate the level of safety considered appropriate for conventional design in most circumstances. These shall be used as a guide to the required level of safety when the method of partial factors is not used.

(20) Where design values for ultimate limit state calculations are assessed directly, they should be selected such that a more adverse value is extremely unlikely to affect the occurrence of the limit state.

Direct assessment of design values is particularly appropriate for actions or combinations of actions for which values derived using Table 2.1 are clearly impossible.

2.4.3 Ground properties

(1)P Design values of ground properties, X_d, shall either be derived from characteristic values, X_k, using the equation:

$$X_d = X_k/\gamma_m \tag{2.1}$$

where:

γ_m is the safety factor for the ground property or shall be assessed directly.

(2)P The selection of characteristic values for soil and rock properties shall be based on the results of laboratory and field tests. Account shall be taken of the possible differences between the properties measured in the tests and the soil and rock properties governing the behaviour of the geotechnical structure due to factors such as:

— presence of fissures, which may play a different role in the test and in the geotechnical structure;

— time effects;

— the brittleness or ductility of the soil and rock tested.

(3) A conversion factor shall be applied where necessary to convert the laboratory and field test results into values which can be assumed to represent the behaviour of the soil and rock in the ground.

(4)P Selection of characteristic values of soil and rock properties shall take account of the following:

— geological and other background information, such as data from previous projects;

— the variabilities of the property values;

— the extent of the zone of ground governing the behaviour of the geotechnical structure at the limit state being considered;

— the influence of workmanship on artificially placed or improved soils;

— the effect of construction activities on the properties of in-situ ground.

(5)P The characteristic value of a soil or rock parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state.

(6) The extent of the zone of ground governing the behaviour of a geotechnical structure at a limit state is usually much larger than the extent of the zone in a soil or rock test and consequently the governing parameter is often a mean value over a certain surface or volume of the ground. The characteristic value is a cautious estimate of this mean value.

The governing zone of ground may also 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 is the mean strength over each individual zone of ground under a footing, if the building is unable to resist a local failure. If instead the building is stiff and strong enough, the governing parameter may be the mean of these mean values over the entire zone or part of the zone of ground under the building.

Statistical methods may be employed in the selection of characteristic values for ground properties. Such methods should allow apriori knowledge of comparable experience with ground properties to be taken into account for example by means of Baysian statistical methods.

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

(7)P Characteristic values may be lower values, which are less than the most probable values, or upper values, which are greater. For each calculation, the most unfavourable combination of lower and upper values for independent parameters shall be used.

(8)P The selection of characteristic values shall take account of the uncertainties in geometrical data and in the calculation model unless they are allowed for directly or in the calculation model.

(9)P For verification in persistant and transient situations of ultimate limit states the numerical values of partial factors for ground properties given in Table 2.1 for the cases A, B and C are generaly appropriate to be used with the partial factors for actions for the same cases for conventional design situations. For accidental situations all numerical values of partial factors shall be taken equal to [1,0].

(10)P For ultimate limit states in which soil strength acts in an unfavourable manner, the value of γ_m adopted shall be less than [1,0].

(11) The degree to which soil strength will be mobilised at the limit state may be taken into account by adopting design values which are less than the upper characteristic values divided by factors γ_m less than [1,0].

(12)P The partial factors for the resistance of a pile or an anchorage, determined on the basis of soil strength parameters, pile driving formulae or load tests, or anchorage tests are given in sections 7 and 8.

(13)P For serviceability limit states all values of γ_m are equal to [1.0].

(14)P Design values of ground properties may also be derived by methods other than the use of partial factors. The partial factors set out in the Table 2.1 indicate the level of safety considered appropriate for conventional designs. These shall be used as guidance to the required level of safety when the method of partial factors is not used.

(15) Where design values for ultimate limit state calculations are assessed directly, they should be selected such that a more adverse value is extremely unlikely to affect the occurrence of the limit state.

2.4.4 Design strength of structural materials

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

2.4.5 Geometrical data

(1)P Geometrical data include the level and slope of the ground surface, water levels, levels of interfaces between strata, excavation levels, the shape of the foundation, etc.

(2)P In cases where variations of the geometrical data are not important, they shall be allowed for in the selection of design values for material properties or actions. In other cases it is generally advisable to allow for these uncertainties directly.

(3)P For limit states with severe consequences, design values for geometrical data shall represent the most unfavourable values which could occur in practice.

2.4.6 Limiting values for movements

(1)P A limiting value for a particular deformation is the value at which an ultimate or serviceability limit state is deemed to occur.

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

(3) The components of foundation movement which may need to be considered include settlement, relative (or differential) settlement, rotation, tilt, relative deflection, relative rotation, horizontal displacement and vibration.

(4)P The design values for the limiting movements shall be agreed with the designer of the supported structure.

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

— the confidence with which the acceptable value of the movement can be specified;

— 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.

(6)P The differential settlements and relative rotations for foundations shall be estimated to ensure that these do not lead to the occurrence of an ultimate limit state or a serviceability limit state, such as unacceptable cracking or the jamming of doors, in the supported structure.

(7) The maximum acceptable relative rotations for open frames, infilled frames and load bearing or continuous brick walls are unlikely to be the same but are likely to range from about 1/2 000 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.

For normal structures with isolated foundations, total settlements up to 50 mm and differential settlements between adjacent columns up to 20 mm are often acceptable. Larger total and differential 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.

The above guides 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.

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

— random or systematic variations in ground properties;

— the loading distribution;

— the construction method;

— the stiffness of the structure.

(9) For the majority of ground conditions, including alluvia, silts, loess, fills, peat and residual soils, consideration should be given to the possibility of a component of differential settlement due to variation in the ground properties across the site.

2.5 Design by prescriptive measures

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

(2) Design by prescriptive measures may be used where comparable experience, as defined in **1.4.2** (1)P, makes design calculations unnecessary. It may also be used to ensure durability against front 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 Results of load tests or tests on experimental models may be used to justify a design, provided that the following features are 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) Tests may be carried out on a sample of the actual construction or on full scale or smaller scale models.

2.7 The observational method

(1)P Because prediction of geotechnical behaviour is often difficult, it is sometimes appropriate to adopt the approach known as "the observational method", in which the design is reviewed during construction. When this approach is used, the following four requirements shall all 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 state; 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.

(2)P During construction, the monitoring shall be carried out as planned, and additional or replacement monitoring shall be undertaken if this becomes necessary. The results of the monitoring shall be assessed at appropriate stages and the planned contingency actions shall be put in operation if this becomes necessary.

2.8 The Geotechnical Design Report

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

(2) 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. 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 of the level of acceptable risks;

— geotechnical design calculations and drawings;

— a note of items to be checked during construction or requiring maintenance or monitoring.

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

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

— the object of each set of observations or measurements;

— the parts of the structure which are to be monitored and the stations at which observations are to be made;

— the frequency with which readings are to be taken;

— the way 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.

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

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

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

(4)P The sampling, transportation and storage procedures shall be reported and their influence shall be considered when interpreting the tests results.

3.2 Geotechnical investigations

3.2.1 Introduction

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

(2) The ground conditions which may influence the decision about the geotechnical category should be determined as early as possible in the investigation as the character and extent of the investigations is related to the geotechnical category of the structure.

For Geotechnical Category 1 situations, the following apply:

As a minimum requirement all design assumptions should be verified at the latest during the supervision of the works. The investigation should include visual inspection of the construction site and also shallow pits, penetration tests or auger borings.

Geotechnical investigations for Geotechnical Category 2 and 3 situations normally include the following three phases which may overlap:

— preliminary investigations (see **3.2.2**);

— design investigations (see **3.2.3**);

— control investigations (see **4.3**).

3.2.2 Preliminary investigations

(1)P Preliminary investigations shall be carried out:

— to assess the general suitability of the site;

— to compare alternative sites, if relevant;

— to estimate the changes 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.

(2) The following items should be considered for inclusion in a preliminary investigation:

— field reconnaissance;

— topography;

— hydrology, especially pore pressure distribution;

— examination of neighbouring structures and excavations;

— geological and geotechnical maps and records;

— previous site investigations and constructional experience in the vicinity;

— aerial photographs;

— old maps;

— regional seismicity;

— any other relevant information.

3.2.3 Design investigations

(1)P Design investigations shall be carried out:

— to provide the information required for an adequate and economic design of the permanent and temporary works;

— to provide the information required to plan the method of construction;

— to identify any difficulties that may arise during construction.

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

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

(4) The following items should be considered for inclusion in a design investigation of the relevant ground:

— geological stratigraphy;

— strength properties of all relevant ground;

— deformation properties of all relevant ground;

— pore pressure distribution through the ground profile;

— permeability conditions;

— possible instability of subsoil;

— compactibility of the ground;

— possible aggressiveness of the ground and the ground water;

— possibility of ground improvement;

— frost susceptibility.

(5)P In order to ensure that the design investigation-covers all relevant ground formations particular attention shall be paid to the following geological features:

— cavities;

— degradation of rocks, soils or fill materials;

— geohydrological effects;

— faults, joints and other discontinuities;

— creeping soil and rock masses;

— expansive and collapsible soils and rocks;

— presence of waste or man-made materials.

(6)P A suitable combination of routine investigation methods shall be used to identify the geotechnical features concerning the ground. These methods shall involve generally available commercial tests performed according to generally accepted or standardized procedures.

(7) A routine investigation should normally include in-situ tests, borings and laboratory tests. Where soundings and/or other indirect methods are used, it is normally necessary to carry out borings in order to identify the ground in which these methods are used. If the geological features of the site are well known, such borings may be omitted.

(8)P The investigation shall be carried out at least through the formations which are assessed as being relevant to the project and below which the ground will have no substantial influence on the behaviour of the structure.

(9)P The distance between the exploration points and the depth of exploration shall be selected on the basis of information on the geology of the area, the ground conditions, the size of the site and the type of structure.

(10) For Geotechnical Category 2 investigations, the following apply:

— In case of structures covering a large area, the exploration points may be placed in a grid. The mutual distance between the points should normally be 20 – 40 m. In uniform soil conditions the borings or excavation pits may be partially replaced by penetration tests or geophysical soundings.

— For pad and strip foundations the depth of soundings or borings below the anticipated foundation level should normally be between 1 and 3 times the width of the foundation elements. Greater depths should usually be investigated in some of the exploration points to assess the settlement conditions and possible ground water problems.

— For rafts the depth of in-situ tests or borings should normally be equal to or greater than the foundation width unless bedrock is encountered within this depth.

— For filled areas and embankments the minimum investigation depth should include all compressible soil strata whose contribution to the settlement is important. The investigation depth may be limited to a level below which the contribution to the settlement is less than 10 % of the total settlement. The distance between neighbouring exploration points should normally be 100 – 200 m.

For piled foundations, borings, penetration or other in-situ tests should normally be performed to explore the ground conditions to a depth to ensure safety, which normally means 5 times the diameter of the shaft of the pile. However, there will be cases when substantially deeper soundings or borings are needed. It is also a requirement that the investigation depth is greater than the smaller side of the rectangle circumscribing the group of piles forming the foundation at the level of the pile toes.

(11)P The existing ground water pressures acting during the investigation shall be established. The extreme levels of any free water which might influence the ground water pressures shall be established and the free water levels during the investigation shall be recorded.

(12) For Geotechnical Category 2 investigations the following apply:

— The investigation of the pore pressure distribution should normally include;

— observations of the water levels in borings and standpipes and their fluctuations with time;

— an evaluation of the hydrogeology of the site including such features as artesian or perched water tables or tidal variation.

— In order to assess excavations for uplift, the pore water pressures should be investigated to a depth below the excavation which equals at least the depth of the excavation below the ground water level. In situations where the upper layers have a low unit weight, investigations to even greater depths may required.

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

(14)P 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 the extent of the investigation shall at least be sufficient to meet the requirements specified above.

(15) For such Geotechnical Category 3 investigations, the following apply:

— Additional investigations of a more specialized nature will often be necessary and shall be undertaken where necessary.

— Whenever test procedures of a specialized or unusual nature are applied, the test procedures and test interpretations shall be documented. Furthermore, references to the tests shall be given.

3.3 Evaluation of geotechnical parameters

3.3.1 General

(1) P Properties of soil and rock and rock masses are quantified by geotechnical parameters which are used in design calculations. They shall be derived from the results of field and laboratory tests and other relevant data. These shall be interpreted in a manner appropriate to the limit state being considered.

(2) 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)P In order to establish reliable values of geotechnical parameters, the following items shall be considered:

— many soil parameters are not true constants, but depend on factors such as stress level, mode of deformation, etc.;

— in interpreting test results, published information relevant to the use of each type of test in the appropriate ground conditions shall be considered;

— testing schedules shall include a sufficient number of tests to provide data for the derivation and variation of the various parameters which are relevant to the design;

— the value of each parameter shall be compared with relevant published data and local and general experience. Published correlations between parameters shall also be considered, if relevant;

— whenever available, the results of large scale field trials and measurements from full scale constructions shall be analysed;

— whenever available, correlations between the results from more than one type of test shall be checked.

3.3.2 Characterization of soil and rock type

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

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

(3) In addition to the above mentioned visual inspection, the following properties may be used for identification purposes:

for soils:

— grain size distribution;

— grain shape;

— grain surface roughness;

— relative density;

— unit weight;

— natural water content;

— Atterberg limits;

— carbonate content;

— organic matter content.

for rocks:

— mineralogy;

— petrography;

— water content;

— unit weight;

— porosity;

— sound velocity;

— quick water absorption;

— swelling;

— slake-durability index;

— uniaxial compression strength.

The strength obtained from uniaxial compressive tests allows the classification of rocks, but simpler testing procedures like the Point Load test may also be used.

3.3.3 Unit weight

(1)P The unit weight shall be determined with sufficient accuracy to establish design values of the actions which derive from it.

(2)P Natural or man-made variations or layering shall be considered when using tests to determine the unit weight.

(3) Knowing the soil type and grading, the in-situ unit weight of sand and gravel may be estimated with sufficient accuracy from the results of tests, such as penetration tests, or observations which indicate the strength of the soil.

3.3.4 Relative density

(1)P The relative density shall express the degree of compactness of a cohesionless soil with respect to the loosest and densest conditions as defined by standard laboratory procedures.

(2) A direct measure of the relative density of a soil may be obtained by comparing an accurate measurement of its in-situ unit weight with laboratory values of its unit weight after standard reference tests. An indirect measure of the relative density of a soil may be obtained from penetration tests.

3.3.5 Degree of compaction

(1)P The degree of compaction shall be expressed as the ratio between its dry unit weight and the maximum dry unit weight obtained from a standard compaction test.

(2) The compaction tests most frequently used are the Standard and the Modified Proctor Tests corresponding to different standard energies of compaction. The compaction test also gives the optimum water content, i.e., the soil water content at a state of maximum dry unit weight for a certain energy of compaction.

3.3.6 Undrained shear strength of cohesive soils

(1)P In assessing the undrained shear strength, c_u of saturated, fine grained soils, the influence of the following features is important and shall be considered:

— differences between the stress states in-situ and in a test;

— sample disturbance, especially for laboratory tests on samples obtained from boreholes;

— anisotropy of strength, especially in clays of low plasticity;

— fissures, especially in stiff clays. Test results may represent the strength either of the fissures or of the intact clay, and either of these may govern field behaviour. Sample size may be important;

— rate effects. Tests carried out too quickly tend to yield higher strengths;

— large strain effects. Most clays exhibit a loss of strength at very large strains and on preformed slip surfaces;

— time effects. 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;

— inhomogeneity of samples, such as inclusions of gravel or sand within a sample of clay;

— degree of saturation;

— the level of confidence in the theory used to derive the undrained shear strength from the test results, especially for in-situ tests.

3.3.7 Effective shear strength parameters for soils

(1)P In assessing the effective shear strength parameters c' and ϕ', the following features shall be considered:

— the stress level of the problem imposed;

— the accuracy of the in-situ determination of the unit weight;

— the disturbance during sampling.

(2)P The values of c' and ϕ' may be assumed constant only within the range of stresses for which they have been evaluated.

(3)P When effective strength parameters c' and ϕ' are obtained from undrained tests with pore pressure measurements attention shall be paid that the samples are fully saturated.

(4) Soils generally exhibit a slightly higher value of ϕ' when tested in plane strain than when tested under triaxial conditions.

3.3.8 Soil stiffness

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

— drainage conditions;

— level of mean effective stress;

— level of imposed shear strain or induced shear stress, this latter often normalized with respect to the shear strength at failure;

— stress and strain history.

(2) These factors are the most important in controlling the stiffness of soils. Other factors influencing the deformation moduli of soils that may be taken into account include:

— direction of soil stressing with respect to the orientation of the principal consolidation stress;

— time and strain rate effects;

— size of the specimen tested in relation to the particle size and macrofabric feature of the soil.

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 stiffness of the soil in-situ. Analysis of observations of the behaviour of previous constructions is therefore recommended.

It is sometimes convenient to assume a linear or log-linear relationship between stress and strain for a limited range of stress change. However, this must always be adopted with caution since the actual behaviour of soil is generally significantly non-linear.

3.3.9 Quality and properties of rocks and rock masses

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

— spacing;

— orientation;

— aperture;

— persistence (continuity);

— tightness;

— roughness, including the effects of previous movements on the joints;

— filling.

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

— in situ stresses;

— water pressure;

— pronounced variations in properties between different layers.

(3) Rock quality may be quantified using the Rock Quality Designation (RQD) which is an indicator of a rock mass for engineering purposes.

Estimation of whole rock properties, such as strength and stiffness, may be obtained by using the concept of rock mass classifications originally developed in connection with tunnelling.

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

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

— some porous soft rocks, degrade rapidly to soils of low strength, especially if exposed to the effects of 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.

3.3.9.1 *Uniaxial compressive strength and deformability of rock materials*

(1)P In assessing the uniaxial compression 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, e.g. 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;

— the geometry of the tested specimens.

(2) Unaxial compressive strength and deformability in uniaxial compression are mainly intended for classification and characterization of intact rock.

3.3.9.2 ***Shear strength of joints***

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

— orientation of the test specimen regarding the rock mass and the assumed actions;

— orientation of the shear test;

— number of specimens tested;

— dimension of the sheared area;

— pore water pressure conditions;

— possibility of progressive failure governing the behaviour of the rock in the ground.

(2) Sheared planes normally coincide with planes of weakness in the rock (joints, planes of bedding, schistosity, cleavage) or with the interface between soil and rock or concrete and rock. Measured shear strength of joints is mainly used for the limit equilibrium analysis of rock masses.

3.3.10 Permeability and consolidation parameters

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

— the effect of non-homogeneous ground conditions;

— the effect of anisotropy in the ground;

— the effect of fissures or faults in the ground, especially in rock;

— the effect of stress changes under the proposed loading.

(2) Permeability measurements made on small laboratory samples may not be representative of the in-situ conditions. Whenever possible, in-situ tests which measure average properties of a large ground volume should therefore be preferred. However, consideration must be given to possible changes in the permeability with increases in effective stress above the insitu value. Sometimes permeability may be evaluated on the basis of a knowledge of the grain size and its distribution.

3.3.11 Cone parameters

(1) In assessing values of the cone resistance, the sleeve friction and, possibly, the pore pressure during penetration the following features should be considered:

— the detailed design of the cone and friction sleeve may affect the results significantly. Allowance must therefore be made for the type of cone in use;

— the results can only be interpreted with confidence when the soil succession is established. In many situations, borings will therefore be needed in conjunction with the penetration tests;

— the effects of the groundwater conditions and the overburden pressure in the soil should be considered when interpreting the results;

— in inhomogeneous soils for which widely fluctuating results are recorded, the penetration values which represent the part of the soil matrix relevant to the design in hand must be considered;

— established correlations with other test results, such as density measurements and other forms of penetration testing, should be considered when available.

3.3.12 Blow count from standard penetration tests and dynamic probing

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

— type of test;

— detailed description of the execution of the test (lifting method, shoe or cone, mass of the falling weight, the drop height, the diameter of the casing and the rods, etc.);

— groundwater conditions;

— the influence of the overburden pressure;

— the nature of the ground, particularly when cobbles or coarse gravel are encountered.

3.3.13 Pressuremeter parameters

(1) In assessing the values of the limit pressure and the pressuremeter modulus, the following features should be considered:

— the type of equipment and, most important;

— the procedure used to install the pressuremeter in the ground.

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.

3.3.14 Dilatometer parameters

(1) In assessing the flat dilatometer values the installation procedure should be considered.

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

The dilatometer modulus should normally be used as a basis for determining the constrained modulus of the soil.

3.3.15 Compactibility

(1)P In assessing the compactibility of a fill material, the following features shall be considered:

— type of soil or rock;

— grain size distribution;

— grain shape;

— the inhomogeneity of the material;

— the degree of saturation or moisture content;

— type of plant to be used.

(2) In order to obtain a direct measure of the compactibility of a soil or a rock fill, a trial compaction test with the intended type of material, thickness of fill layer and type of compaction equipment should be made. The density thus obtained is related to laboratory values for standard procedures of compaction and is also related to field values for the intended site control device or procedure (e.g. soundings, dynamic compaction testing, plate load tests, settlement records).

3.4 Ground Investigation Report

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

(2) The Ground Investigation Report should normally consist of the following two parts:

— presentation of available geotechnical information including geological features and relevant data;

— geotechnical evaluation of the information, stating the assumptions made in the derivation of the geotechnical parameters.

These parts may be combined into one report or divided between several reports.

3.4.1 Presentation of geotechnical information

(1)P The presentation of geotechnical information shall include a factual account of all field and laboratory work and documentation of the methods used to carry out the field investigations and the laboratory testing.

(2) In addition to the above, the factual report should include the following information, if relevant:

— purpose and scope of the geotechnical investigation;

— brief description of the project for which the geotechnical report is being compiled giving information about the location of the project, its size and geometry, anticipated loads, structural elements, materials of construction etc.;

— a statement of the anticipated Geotechnical Category of the structure;

— dates between which field and laboratory work were performed;

— procedures used for sampling, transportation and storage;

— types of field equipment used;

— survey data;

— names of all consultants and subcontractors;

— field reconnaissance of the general area of the project noting particularly:

  — evidence of ground water;

  — behaviour of neighbouring structures;

  — faulting;

  — exposures in quarries and borrow areas;

  — areas of instability;

  — difficulties during excavation;

— history of the site;

— geology of the site;

— information from available aerial photographs;

— local experience in the area;

— information about the seismicity of the area;

— 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 ground water 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;

— grouping and presentation of field and laboratory test results in appendices.

3.4.2 Evaluation of geotechnical information

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

— review of the field and laboratory work. In cases where there are limited or partial data, it shall be stated. If the data are defective, irrelevant, insufficient or inaccurate, it shall be pointed out and commented upon accordingly. The sampling, 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 whether they are misleading or represent a real phenomenon that must be accounted for in the design;

— submission of proposals for further field and laboratory work, if deemed necessary, with comments justifying the need for this extra work. Such proposals shall be accompanied by a detailed programme for the types of extra investigations to be carried out with specific reference to the points which have to be answered.

(2) In addition to the above, 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;

— determination of the depth of the ground water 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 compressibility and strength characteristics. Comments on irregularities such as pockets and cavities;

— grouping and presentation of the range of values of the geotechnical data for each stratum. This presentation should be in a comprehensible form which will enable the most appropriate ground parameters to be selected for the design.

Section 4. Supervision of construction, monitoring and maintenance

4.1 General requirements

(1)P In order to ensure the safety and quality of a structure, the following shall be undertaken, as appropriate:

- — the construction processes and workmanship shall be supervised;
- — the performance of the structure shall be monitored during and after construction;
- — the structure shall be adequately maintained.

(2)P Supervision of the construction process and workmanship and monitoring of the performance of the structure during and after construction shall be carried out in accordance with the specifications in the Geotechnical Design Report.

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

- — 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.

Monitoring the performance of the structure during and after construction should involve, as appropriate, the following:

- — observations and measurements to monitor the behaviour of the structure and its surroundings during construction so as to identify the need for remedial measures, alterations to the construction sequence, etc.;
- — observations and measurements to monitor and evaluate the long-term performance of the structure and its surroundings.

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

(5) The inspection, control, field and laboratory testing required for supervising construction and monitoring performance should be planned during the design stage. In the case of unexpected events, the extent and frequency of monitoring should be increased.

Annex A gives a checklist for construction supervision and performance monitoring.

4.2 Supervision

4.2.1 Plan of supervision

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

(2) The plan of supervision 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 design modifications or of implementing corrective measures during construction.

4.2.2 Inspection and control

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

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

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

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

(3)P 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 on the environmental conditions;

— unforeseen events.

(4) Records of temporary works should also be kept. Interruptions to the works, and their conditions on recommencement, should be recorded.

(5)P The results of the inspection and control shall be made available to the designer before the decisions, which may be their consequence, are taken.

4.2.3 Assessment of the design

(1)P The suitability of the construction procedures and the sequence of operations shall be reviewed against the ground conditions which are encountered and 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 control. If necessary, the structure shall be redesigned.

(2) 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 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) 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.

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

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

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.

(3)P Deviations from the ground type and properties assumed in the design shall be reported without delay to the person responsible for the project.

(4)P 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 As appropriate, the groundwater levels, pore pressures and groundwater chemistry encountered during execution shall be checked and compared with those assumed in the design. More thorough checks are needed for sites on which significant variations of ground type and permeability are known or suspected to exist.

(2) For Geotechnical Category 1, controls are usually based on previously documented experience in the area or on indirect evidence.

For Geotechnical Categories 2 and 3, direct observations are normally made of the groundwater conditions if these greatly affect either the method of construction or the performance of the structure.

Groundwater flow characteristics and the pore pressure regime may 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.

If pore pressure changes occur during construction which 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.

For structures below groundwater level which may float, porewater pressures should be monitored until the weight of the structure is sufficient to rule out the possibility of floating.

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

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

(4)P Deviations from the groundwater features assumed in the design shall be reported without delay to the person responsible for the project.

(5)P 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 Site operations shall be checked for compliance with the method of construction assumed in the design and stated in the Geotechnical Design Report.

(2) For Geotechnical Category 1, a formal construction schedule is not normally included in the Geotechnical Design Report. The sequence of construction operations is normally decided by the contractor.

For Geotechnical Categories 2 and 3, the Geotechnical Design Report may give the sequence of construction operations envisaged in the design. Alternatively, the Geotechnical Design Report may state that the sequence of construction is to be decided by the contractor.

(3)P Subsequent 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 and shall be reported without delay to the person responsible for the project.

(4)P It shall be checked that the principles used in design are appropriate for the sequence of construction operations which are used.

4.5 Monitoring

(1)P The aims of monitoring are:

— 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 The inspection and measurements required to monitor the performance of the completed structure shall be specified to the owner/client.

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

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

Measurements may include the following:

— deformations of the ground affected by the structure;

— values of actions;

— values of contact pressure between ground and structure;

— pore water pressures and their variation with time;

— stresses and deformations (vertical or horizontal movements, rotations or distortions) in structural members.

Results of measurements may be integrated with qualitative observations including architectural appearance.

The length of the post-construction monitoring period may be altered as a result of observations obtained 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 may be required for more than ten years after construction is complete, or throughout the life of the structure.

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

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

For Geotechnical Category 2, the evaluation of performance may be based on measurements of movements of selected points of the structure.

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.

(7)P 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.

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

(2) 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 The provisions in this section apply where adequate ground conditions are achieved by:

— placing soil or granular material;

— dewatering;

— treating ground;

— reinforcing ground.

(2) Situations where soil or granular material is placed for engineering construction 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 dykes and transportation networks.

Dewatering of ground may be temporary or permanent.

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

(3)P Design procedures for geotechnical structures which may include the use of fill, dewatering, improvement and reinforcement are dealt with in sections 6 to 9.

5.2 Fundamental requirements

(1)P The fundamental requirements to be satisfied are that fill and dewatered, improved or reinforced ground be capable of sustaining the actions caused by loads, seeping water, vibrations, temperature, rain, etc.

(2)P The 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 The adequacy of the fill shall be based on good material handling possibilities and achieving adequate engineering properties after compaction.

(2) Transport and placement should be considered during design.

5.3.2 Selection of fill material

(1)P The criteria for selecting 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) Suitable fill materials 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, can also be used in some circumstances. Some cohesive materials may be suitable but require particular care.

(3)P The following aspects shall be considered when selecting a fill material:

— grading;

— resistance to crushing;

— compactibility;

— plasticity;

— organic content;

— chemical agressivity;

— pollution effects;

— solubility;

— susceptibility to volume changes (swelling clays and collapsible materials);

— the effect of frost;

— resistance to weathering;
— the effect of excavation, transportation and placement;
— the possibility of cementation occurring after placement (e.g. blast furnace slags).

(4) If local materials are not suitable for use as fill in their natural state it may 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)P When the selected material contains potentially aggressive or polluting chemicals, adequate provisions shall be adopted to prevent these attacking structures or services or polluting the groundwater. Such materials shall only be used in large amounts in permanently monitored locations.

(6)P In case of doubt, the source of fill material shall be tested 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.

(7) In Geotechnical Category 1, visual inspection of the material will often be sufficient.

(8)P The fill material shall not contain foreign matter such as snow, ice or peat in any significant amount.

5.3.3 Selection of fill placement and compaction procedures

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

(2)P The procedures for fill placement and compaction shall be selected 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 Selection of the compaction procedure for fills will depend 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.

(4) In order to develop an appropriate procedure for compaction, a trial field test should be made 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)P Where there is a possibility of rainfall during the placement of cohesive fill material, the fill surface shall at all stages be profiled so as to drain adequately.

(6)P Frozen, expansive or soluble soils shall normally not be used as a fill material.

(7) Placement of fill at temperatures below the freezing point may require heating of the fill to be placed and frost protection of the fill surface.

(8)P If backfill is used around foundations, the compaction procedure shall be selected so that later subsidence around foundations and beneath floor slabs does not cause damage.

5.3.4 Checking the fill

(1)P The compaction work shall be checked by inspecting or testing in order to ensure that the nature of the fill material, its placement water content and the compaction procedures are consistent with those prescribed.

(2) For some combinations of materials and compaction procedures, testing after compaction may not be needed. In particular, testing may be replaced by checking that compaction has been performed according to the procedure deduced from a trial field test or from comparable experience.

Testing of compaction should use 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 modulus, etc. Such measurement may not be able to determine whether satisfactory compaction has been achieved in cohesive soils.

Structural fills on which foundations are built shall be made with suitable materials (see **5.3.2** (1)P) for which an appropriate density, a 100 % Proctor density (2.5 kg rammer, height of fall 0.3 m) as an average and a 97 % Proctor density as a lower limit shall be assured and the risk of collapse and excessive differential settlements shall be prevented.

With rock fill or fill containing a large amount of coarse particles, the use of Proctor density to test compaction is not applicable. Checking of compaction may then be made by:

— checking that compaction has been performed according to the procedure deduced form a trial field test or from comparable experience;

— checking that the additional settlement induced by an additional pass of the compaction equipment is lower than a specified value;

— seismic methods.

(3)P In cases where overcompaction is not acceptable, an upper bound limit for the compaction shall be specified.

(4) Overcompaction may cause the following undesirable effects:

— the development of slickenslides and high soil stiffnesses in dykes 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 Any scheme for removing water from the ground or for lowering the water pressure shall be based on the results of a geotechnical investigation.

(2) 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.

Part of the dewatering system can be a system of recharge wells at some distance from the excavation.

(3)P The dewatering scheme shall satisfy the following conditions as appropriate:

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

— the scheme adopted shall not lead to excessive settlements or damage to nearby structures;

— the scheme adopted 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 shall be available in the case of breakdown to facilitate maintenance;

— 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 adopted shall not lead to excessive transport of contaminated water to the excavation;

— the scheme adopted shall not lead to excessive extraction of water in a drinking water catchment area.

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

(5)P If a pumping operation is to extend over a long period of time, the groundwater should 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. Bacterial action can also be a problem procuring clogging deposits in long term dewatering.

5.5 Ground improvement and reinforcement

(1)P Before any ground improvement or reinforcement method is chosen or used, a geotechnical investigation shall be carried out to obtain an adequate knowledge of the initial ground conditions.

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

— the thickness and properties of the in situ ground strata or fill material;

— the magnitude of water pressure in the various strata;

— the nature, size and position of the structure to be supported by the ground;

— the prevention of damage to adjacent structures or services;

— whether 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 materials or changes in groundwater level;

— the long-term effects with respect to deterioration of materials.

(3) In many cases ground improvement and reinforcement works should be classified in Geotechnical Category 3.

(4)P The effectiveness of the ground improvement shall be checked against the acceptance criteria by determining the changes in the appropriate ground properties or conditions resulting from the improvement method.

Section 6. Spread foundations

6.1 General

(1)P The provisions in this chapter apply to spread foundations including pads, strips and rafts. Some of the provisions may also apply to deep foundations such as caissons.

6.2 Limit states

(1)P A list of limit states to be considered shall be compiled. The following limit states shall be considered:

— loss of overall stability;

— bearing resistance failure;

— failure by sliding;

— combined failure in ground and in structure;

— structural failure due to foundation movement;

— excessive settlements;

— excessive heave;

— unacceptable vibrations.

6.3 Actions and design situations

(1)P In selecting the actions for calculations of limit states, the actions listed in **2.4.2** shall be considered.

(2) When structural stiffness is significant, an analysis of the interaction between structure and ground may be needed in order to determine the distribution of actions.

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

(4) When selecting the design situations for spread foundations it is especially important to assess the level of groundwater table.

6.4 Design and construction considerations

(1)P 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 required for construction 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.

(2)P In addition to fulfilling the performance requirements, the foundation width shall be designed taking account of practical considerations related to economic excavation, setting out tolerances, working space requirements and the dimensions of the wall or column supported by the foundation.

(3)P When designing a spread foundation, one of the following design methods shall be used:

— a direct method, in which separate analyses are carried out for each limit state using calculation models and design values for the actions and the ground parameters. 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 deformation analysis shall be used.

— presumed bearing resistance, estimated empirically 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.

Calculation models for ultimate and serviceability limit state design of spread foundations on soil are given in **6.5** and **6.6**. Presumed bearing pressures for the design of spread foundations on rock are given in **6.7**.

6.5 Ultimate limit state design

6.5.1 Overall stability

(1)P Failures due to loss of overall stability shall be checked in particular for foundations in the following situations:

— near or on an inclined site, a natural slope or an embankment;

— 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.

For such situations, it shall be demonstrated by the principles described in section 9 that a stability failure of the ground mass containing the foundation is sufficiently improbable.

6.5.2 Bearing resistance failure

6.5.2.1 *General*

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

$$V_d \leqslant R_d \tag{6.1}$$

where

V_d is the ultimate limit state design load normal to the foundation base including the weight of the foundation and of any backfill material. In drained conditions water pressures shall generally be included as actions in calculating V_d.

R_d is the design bearing resistance of the foundation against normal loads, taking into account the effect of any inclined or eccentric load. R_d shall be calculated from design values of the relevant parameters chosen in accordance with **2.4.3** and **3.3**.

(2) When water pressures around the footing are hydrostatic, calculation of V_d may be simplified by using buoyant weights of structural elements below the water table.

6.5.2.2 *Analytical method*

(1)P In an analytical evaluation of the design vertical bearing resistance, R_d, of a spread foundation both short-term and long-term situations shall be considered particularly in fine-grained soils where changes of pore water pressure may lead to changes in shear strength.

(2) Annex B gives a sample analytical method for bearing resistance calculation.

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

(4)P When calculating the design bearing resistance of a foundation on highly layered deposits, the design values of the ground parameters for each layer shall be determined.

(5) Where a strong formation underlies a weak formation, the bearing resistance may be calculated by applying the shear parameters of the weak formation.

6.5.2.3 *Semi-empirical method*

(1)P The design bearing resistance of a spread foundation may be estimated semi-empirically from the results of in situ tests taking account of comparable experience.

(2) Annex C gives a sample semi-empirical method for bearing resistance estimation.

6.5.3 Failure by sliding

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

(2)P For safety against failure by sliding on a horizontal base, the following inequality shall be satisfied:

$$H_d \leqslant S_d + E_{pd} \qquad (6.2)$$

where:

H_d is the horizontal component of the design load including the design active earth forces;

S_d is the design shear resistance between the base of the foundation and the ground;

E_{pd} is the design resisting earth pressure on the side of the footing which can be mobilized with the displacement appropriate to the limit state considered and which is available throughout the life of the structure.

(3) The values of both S_d and E_{pd} 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.

(4)P For sloping bases, a similar requirement to the inequality (**6.5**) shall be applied.

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

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

(7)P For drained conditions, the design shear resistance, S_d, shall be calculated using the following equation:

$$S_d = V'_d \tan \delta_d \qquad (6.3)$$

where:

V'_d is the design effective load, normal to the foundation base;

δ_d is the design friction angle on the foundation base.

(8) The design friction angle δ_d may be assumed equal to the design angle of shearing resistance ϕ'_d for cast-in-situ concrete foundations and equal to 2/3 ϕ'_d for smooth precast foundations. Any effective cohesion c' should be neglected.

(9)P For undrained conditions, the design shearing resistance, S_d, shall in general be limited by:

$$S_d = A' c_u \qquad (6.4)$$

where:

A' is the effective base area, as in **6.5.2.2**;

c_u is the undrained shear strength.

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:

$$S_d \leq 0{,}4\ V_d \qquad (6.5)$$

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 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 deviations in the actual works.

(2) Unless special care is taken during the works differences up to 0.10 m should be considered.

(3)P The above conservative design value for the location of the foundation edge shall be used in the bearing resistance check.

6.5.5 Structural failure due to foundation movement

(1)P Differential vertical and horizontal displacements of the foundations for a structure under the ultimate limit state design loads and ground deformation parameters shall be considered to ensure that these do not lead to an ultimate limit state occurring in the supported structure.

(2) The second method outlined in **6.4**.(3)P may be adopted using design bearing pressure for which displacements will not cause an ultimate limit state in the structure.

(3)P 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

(1)P Foundation displacements caused by the superstructure shall be considered both in terms of displacement of the entire foundation and differential displacements of different parts of the foundation.

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

(3)P The possible range of vertical and horizontal displacements of the foundation shall be assessed and compared with the limiting values for movements indicated in **2.4.5**.

(4)P Displacements caused by the actions on the foundation, such as those listed in **2.4.2**, shall be considered.

(5)P Methods which may be used to calculate vertical displacements (settlements) caused by loads on the foundation are given in **6.6.1**.

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

6.6.1 Settlement

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

(2) For the calculation of settlements in saturated soils, the following three components of settlement should be considered:

— settlement without drainage, for fully saturated soil due to shear deformation at constant volume, s_0;

— settlement caused by consolidation, s_1;

— settlement caused by creep, s_2.

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.

The depth to which compressible soil layers should be taken into consideration depends on the size and shape of the foundation, the variation in soil stiffness with depth and the spacing of foundation elements.

Normally, this depth may be taken as the depth at which the effective vertical stress due to the foundation load amounts to 20 % of the effective overburden stress.

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.

(3)P Any possible additional settlement caused by self compaction of the soil shall also be assessed.

(4) The following should be considered:

— in fill and collapsible soils, the possible effects of self-weight, flooding and vibration;

— in crushable sands, the effects of stress changes.

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

(6) Annex D gives sample methods for settlement evaluation.

(7)P The differential settlements and relative rotations shall be assessed, taking account of both the distribution of loads and the possible variability of the ground, to ensure that these do not lead to the occurrence of a serviceability limit state.

(8) Differential settlements calculated without taking account of the stiffness of the structure tend to be overpredictions. An analysis of ground-structure interaction may be used to justify reduced values of differential settlements.

Differential settlement caused by variability of the ground should be allowed for unless it is prevented by the stiffness of the structure. For spread foundations on natural ground, its magnitude may typically be up to 10 mm, but not usually exceeding 50 % of the calculated total settlement.

The tilting of an eccentrically loaded foundation shall 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.

6.6.2 Vibration analysis

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

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

(3)P Vibrations caused by earthquakes shall be considered according to ENV 1998-1.

6.7 Foundations on rock: Additional design considerations

(1)P 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 and other discontinuities and their characteristics (e.g. fillings, 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, as underground works, slopes etc. being near to the foundation.

(2) Spread foundations on rock may normally be designed using the method of presumed bearing pressures described in **6.4**.(3)P.

For strong intact igneous rocks, gneissic rocks, limestones and sandstones, the presumed bearing pressure is limited by the compressive strength of the concrete foundation.

Annex E gives a sample method for deriving the presumed bearing resistance for spread foundations on rock.

The settlement of a foundation may be assessed on the basis of comparable experience related to rock mass classification, cf **3.3.9**.

6.8 Structural design of spread foundations

(1)P Spread foundations shall be verified against structural failure in accordance with **2.4**.

(2) For rigid foundations, the bearing pressure may be assumed to be distributed linearly. More detailed analysis of soil-structure interaction may be used to justify a more economic design, following the principles of **2.1** (8)P.

For a flexible foundation, the distribution of the contact pressure may be derived by modelling the foundation as a beam or slab resting on a deforming continuum or series of springs with appropriate stiffness and strength.

(3)P 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.

(4) Normally the bearing pressure may be assumed to be distributed linearly.

For design situations with concentrated forces 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 be assessed by a settlement analysis with an appropriate estimate of the bearing pressure distribution. The moduli should be adjusted so that the computed bearing pressures do not exceed values for which linear behaviour may be assumed.

Total and differential settlements of the structure as a whole should be calculated in accordance with **6.6.1**. 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.

Section 7. Pile foundations

7.1 General

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

7.2 Limit states

(1)P A list of limit states to be considered shall be compiled. The following limit states shall be considered:

— 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 settlements;

— excessive heave;

— unacceptable vibrations.

7.3 Actions and design situations

7.3.1 General

(1)P For calculations of limit states, the actions listed in **2.4.2** shall be considered.

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

(3) An analysis of the interaction between structure and ground may be needed in order to determine the actions from the structure to be adopted in the design of the pile foundations. It may be necessary to consider both low and high characteristic values of the deformation parameters in the interaction analysis.

7.3.2 Actions due to ground displacement

7.3.2.1 *General*

(1)P Ground in which piles are located may be subject to displacement caused by consolidation, swelling, adjacent loads, creeping soil, landslides or earthquakes. These phenomena affect the piles by causing downdrag (negative skin friction), heave, stretching, transverse loading and displacement. For these situations, the design values if the strength and stiffness of the moving ground shall usually be upper values.

(2)P 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 If design calculations are carried out treating the downdrag force as an action, its value shall be the maximum which could be generated by large settlement of the ground relative to the pile.

(2) Calculation of maximum downdrag forces should take account of the shear resistance of the soil along the shaft of the pile, the depth of compressible soil, the weight of soil and the surface load around each pile which is the cause of the settlement.

For a group of piles, an upper bound to the downdrag force may be calculated from the weight of the surcharge causing settlement with allowance for changes in groundwater pressure due to groundwater lowering, consolidation or pile driving.

(3)P 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. The design value of the settlement of the ground shall be derived taking account of material unit weights and compressibility in accordance with **2.4.3**.

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

7.3.2.3 ***Heave***

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

(2) 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 ground water content resulting from the removal of trees, cessation of abstraction from aquifers, prevention (by new construction) of evaporation and from accidents.

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 Transverse ground movements exert transverse loading on pile foundations. This transverse loading shall be considered if one or a combination of the following situations occur:

- — 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.

(2) Transverse loading on pile foundations should normally be evaluated by considering the piles as beams in a deforming soil mass.

When the horizontal deformation of the upper weak soil layers is large and the piles are widely spaced, the resulting transverse loading depends on the shear strengths of the weak soil layers.

7.4 Design methods and design considerations

7.4.1 Design methods

(1)P 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.

(2) Design values for parameters used in the calculations should be in general accordance with **3.3**, but the results of load tests may also be taken into account in selecting parameter values.

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

It is sometimes acceptable to use the observed performance of a comparable existing pile foundation in place of load tests provided that this approach is supported by the results of site investigation and ground testing.

Pile foundations for structures classified in Geotechnical Category 1 may be designed from comparable experience, without supporting load tests or calculations, provided the pile type and ground conditions remain within the area of experience and the ground conditions are checked and the installation of the piles is supervised in accordance with the principles of section 4.

7.4.2 Design considerations

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

(2)P 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 Planned future placing or removal of overburden or potential changes in the ground water regime shall be taken into account, both in calculations and in the use of load test results.

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

— the ground 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.

(5) In considering the aspects listed above, the following items require 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, especially under bentonite, to remove remoulded materials;

— local instability of a pile borehole during concreting which may cause a soil inclusion within the shaft;

— ingress of soil or water into the section of cast-in-situ piles;

— disturbance of the concrete before setting by groundwater flow;

— the effect of non-saturated sand layers around a pile extracting water from the concrete;

— the retarding influence of chemicals in the soil or the effect of the movement of groundwater on wet concrete in cast-in-situ piles which are not permanently cased;

— soil compaction due to the driving of displacement piles;

— soil disturbance due to the boring of pile shafts for bored piles.

7.5 Pile load tests

7.5.1 General

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

— when using a type of pile or installation method that is outside comparable experience and which has not been tested under comparable soil and loading conditions;

— when using a piling system which is outside the experience of the operatives carrying out the work;

— 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 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) 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;

— check the performance of individual piles and to allow judgement of the overall pile foundation.

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.

(3)P 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 bearing resistance.

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

(4)P 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 pressures have regained their initial values.

(5) In Some cases it may be necessary to record the pore 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 The pile load test procedure, 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 pile foundation from 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) Static load tests should be carried out in accordance with the ISSMFE Subcommittee on Field and Laboratory Testing's recommended procedure, "Axial Pile Loading Test, Suggested Method", published in the ASTM Geotechnical Testing Journal, June 1985, pp 79 – 90.

Devices for the determination of forces, stresses or strains and displacements should be calibrated prior to the test.

The direction of the applied force to compression or tension pile tests should coincide with the longitudinal axis of the pile.

In general, 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 normally not be used, especially in the case of severe transient loading.

7.5.2.2 *Trial piles*

(1)P The number of trial piles required to verify the design shall be selected based on the following aspects:

— the ground conditions and their variability across the site;

— the geotechnical category of the structure;

— 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 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 deformation behaviour, at least, to a depth of 5 times the pile diameter beneath the pile tip, unless sound rock or very hard soil is found at a lesser depth.

(3)P The method used for the installation of the trial piles shall be fully documented in accordance with **7.10**.

7.5.2.3 *Working piles*

(1)P The number of working pile load tests shall be selected on the basis of the recorded findings during construction.

(2) The selection of working test piles can be prescribed in the contract documents. This prescription may be related to the results of pile installation recordings.

(3)P The load applied to working test piles shall be at least equal to the design load governing the design of the foundation.

7.5.3 Dynamic load tests

(1)P The results of dynamic load tests may be used for design 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 under comparable soil conditions.

(2)P Dynamic test results shall always be considered in relation to each other.

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

7.5.4 Load test report

(1)P A report shall be made of 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;
- a description of the loading and measuring apparatus and the reaction system;
- calibration documents of the load cells, the jacks and the gauges;
- the installation record of the test piles;
- photographic records of the pile and the test site;
- test results in numerical form;
- time settlement plots for each applied load when a step loading procedure is used;
- the measured load-settlement behaviour;
- justification of the reasons for any departures from the above recommendations.

7.6 Piles in compression

7.6.1 Limit state design

(1)P The design shall demonstrate that the following classes of limit states are sufficiently improbable:

- ultimate limit states of overall stability failure;
- ultimate limit states of bearing resistance failure of the piled foundation;
- ultimate limit states of collapse or severe damage to a supported structure caused by displacement of the piled foundation;
- serviceability limit states in the supported structure caused by displacement of the piles.

(2) Overall stability should be checked in accordance with **7.6.2**. Normally the design process should then consider the margin of safety with respect to bearing resistance failure, which is the state in which piles are displaced indefinitely into the ground with negligible increase in resistance. This is the subject of **7.6.3**.

Settlement of piles is considered in **7.6.4**. For piles which require large settlements to reach their ultimate bearing resistance, ultimate limit states may occur in supported structures before the bearing resistance in the piles is fully mobilised. In these cases, the approach used in **7.6.3** for derivation of characteristic and design values should also be applied to the whole of the load-settlement curve, with the same numerical factors.

7.6.2 Overall stability

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

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

(3)P The clauses in **6.5.1**, relating to the overall stability of spread foundations, apply also to foundations involving piles in compression.

7.6.3 Bearing resistance

7.6.3.1 *General*

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

$$F_{cd} \leq R_{cd} \tag{7.1}$$

where:

F_{cd} is the ultimate limit state axial design compression load;

R_{cd} is the sum of all the ultimate limit state design bearing resistance components of the pile foundation against axial loads, taking into account the effect of any inclined or eccentric loads.

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

— downdrag is significant;

— the soil is very light, or;

— the pile extends above the surface of the ground.

(3)P For piles in groups, two failure mechanisms shall be considered:

— bearing resistance failure of the piles individually;

— bearing resistance failure of the piles and the soil contained between them acting as a block.

The design bearing resistance shall be taken as the lower of these two values.

(4) Generally, the bearing resistance of the pile group acting as a block may be calculated by treating the block as a single pile of large diameter. When the piles are used to reduce the settlement of a raft, their resistance corresponding to the creep load may be used in analysing the serviceability states of the structure.

(5)P The assessment of the ultimate bearing resistance of individual piles shall take account of the potential adverse effect of adjacent piles.

(6)P If the bearing stratum of the piles overlies a layer of weak soil, the effect of the weak layer on the bearing resistance of the foundation shall be considered.

(7)P When deriving the design bearing resistance of a pile group, the nature of the structure connecting the piles in the group shall be considered.

(8) If the piles support a flexible structure, it should be assumed that the bearing resistance of the weakest pile governs the occurrence of a limits rate.

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.

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

7.6.3.2 *Ultimate bearing resistance from pile load tests*

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

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

(3) 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 bearing resistance to be adopted.

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.

The approach should be used with caution for open-ended driven piles because of the influence of the diameter on the mobilisation of the end bearing resistance of a soil plug in the pile.

(4)P 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 by subtracting the measured or the most unfavourable design positive skin friction force in the compressible stratum from the forces measured at pile head.

(5) During the load test, positive skin friction will be developed along the total length of the pile and should be considered in accordance with **7.3.2.2**. The maximum load applied to the working test pile should be in excess of the sum of the design external load plus twice the downdrag.

(6)P When deriving the ultimate characteristic bearing resistance R_{ck} from values R_{cm} 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. As a minimum, both conditions (a) and (b) of Table 7.1 shall be satisfied using the equation:

$$R_{ck} = R_{cm}/\xi \tag{7.2}$$

Table 7.1 — Factors ξ to derive R_{ck}

Number of load tests	1	2	> 2
(a) Factor ξ on mean R_{cm}	[1.5]	[1.35]	[1.3]
(b) Factor ξ on lowest R_{cm}	[1.5]	[1.25]	[1.1]

(7) The systematic and random components of the variations in the ground should be distinguished in the interpretation of pile load tests. Account of the systematic components of ground variability can be made by considering different zones of homogeneous conditions or a trend of ground conditions with the position on the site. The records of the installation of the test pile(s) should be checked, and any deviation from the normal execution conditions should be accounted for. Such variations should be covered in part by a correct selection of the test piles.

(8)P In order to derive the ultimate design bearing resistance, the characteristic value, R_{ck}, should be divided into components of base resistance, R_{bk}, and shaft resistance, R_{sk}, such that:

$$R_{ck} = R_{bk} + R_{sk} \tag{7.3}$$

(9) The ratio of these components may be derived from the load test results, e.g. when measurements of these components have been performed, or estimated using the methods of **7.6.3.3**.

(10)P The design bearing resistance, R_{cd}, shall be derived from:

$$R_{cd} = R_{bk} / \gamma_b + R_{sk} / \gamma_s \tag{7.4}$$

where γ_b and γ_s, are taken from Table 7.2.

Table 7.2 — Values of γ_b, γ_s, and γ_t

Component factors	γ_b	γ_s	γ_t
Driven piles	[1.3]	[1.3]	[1.3]
Bored piles	[1.6]	[1.3]	[1.5]
CFA (Continuous flight auger) piles	[1.45]	[1.3]	[1.4]

(11) Normally the load test only provides the pile load versus settlement and time versus settlement diagrams without distinction between point and shaft resistance. Therefore, it is often not possible to distinguish between the partial factors for the assessment of the design value of base resistance and shaft resistance. Instead a partial factor on the ultimate characteristic pile resistance R_{ck} may be taken as the γ_t values given in Table 7.2.

7.6.3.3 ***Ultimate bearing resistance from ground test results***

(1)P The design bearing resistance, R_{cd}, of a pile shall be found from:

$$R_{cd} = R_{bd} + R_{sd} \tag{7.5}$$

where:

R_{bd} is the design base resistance;

R_{sd} is the design shaft resistance.

(2)P R_{bd} and R_{sd} should be derived from:

$R_{bd} = R_{bk} / \gamma_b$

and

$$R_{sd} = R_{sk} / \gamma_s \tag{7.6}$$

where:

$R_{bk} = q_{bk} \cdot A_b$

and $$R_{sk} = \sum_{i=1}^{n} q_{sik} A_{si} \tag{7.7}$$

where the following symbols are used:

R_{bk} and R_{sk}	the characteristic values of the base and shaft resistances;
A_b	the nominal plan area of the base of the pile;
A_{si}	the nominal surface area of the pile in soil layer i;
q_{bk}	the characteristic value of the resistance per unit area of the base;
q_{sik}	the characteristic value of the resistance per unit area of the shaft in layer i.

(3)P The values of γ_b and γ_s, shall be taken from Table 7.2.

(4)P The characteristic values q_{bk} and q_{sik} shall be derived from calculation rules based on established correlations between the results of static load tests and the results of field or laboratory ground tests. These calculation rules shall be devised such that ultimate bearing resistances using the characteristic values q_{bk} and q_{sik} do not exceed the measured ultimate bearing resistances used for establishing the correlation divided by [1.5], on average.

(5)P The calculation rules shall be established on the basis of comparable experience as defined in **1.4.2**.

(6) In assessing the validity of a calculation rule, the following items should be considered:

— soil type, including grading, mineralogy, angularity, density, preconsolidation, compressibility and permeability;

— installation of the pile, including method of boring or driving, (or other method of installation), length, diameter and material;

— method of ground testing.

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

(8) The zone of ground which influences the base resistance extends several diameters above and below the pile toe. In the design, account should be taken of weak ground in this zone which has a relatively large influence on the base resistance.

If weak ground is present at a depth of less than 4 times the base diameter below the base of the pile, a punching failure mechanism should be considered.

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

— the shearing resistance between the ground plug and the inside face of the tube or box;

— the base resistance derived using the cross sectional area of the base.

(10)P If piles with oversized base plates are installed, the possible adverse effect of the oversized plate on the base and shaft resistance of the piles shall be considered.

7.6.3.4 ***Ultimate bearing resistance from pile driving formulae***

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

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

(3)P 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 bearing resistance assessment. If redriving gives higher results, these may be taken into consideration.

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

7.6.3.5 ***Ultimate bearing resistance from wave equation analysis***

(1)P Where wave equation analysis is used to assess pile bearing resistance of individual compression piles the validity of the analysis shall have been demonstrated by previous evidence of acceptable performance on static load tests on the same pile type of similar length and cross section and in similar soil conditions. The input energy level during the dynamic load testing shall be high enough to allow for an appropriate interpretation of the pile capacity at a correspondingly high enough strain level.

(2)P The input parameters in the wave equation analysis may be subject to modification where dynamic pile testing of trial piles is carried out.

(3) Dynamic pile testing may supply a greater insight into actual hammer performance and dynamic ground parameters.

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

7.6.4 Settlement of pile foundations

(1)P Settlements under serviceability and ultimate limit state conditions shall be assessed and compared against the relevant limiting values for the movement given in **2.4.5**.

(2)P In cases where ultimate limit states may occur in supported structures before the ultimate bearing resistance of piles is fully mobilised, the procedures of **7.6.3** for derivation of characteristic and design values shall also be applied to the whole of the load-settlement curve, with the same numerical factors, and the same treatment of downdrag.

(3)P The assessment of settlement shall include the following components:

— the settlement of a single pile;

— the additional settlement due to group action.

The settlement analysis shall include an estimate of the differential settlements that may occur.

7.7 Piles in tension

7.7.1 General

(1)P The design of piles in tension shall be consistent with the design rules given in **7.6**, where applicable. Design rules specifically for foundations involving piles in tension are presented in this section.

7.7.2 Ultimate tensile resistance

7.7.2.1 ***General***

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

$$F_{td} \leq R_{td} \qquad (7.8)$$

where:

F_{td} is the ultimate limit state axial design tensile load and;

R_{td} is the ultimate limit state design tensile resistance of the pile foundation.

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

— pull out of the piles from the ground mass;

— uplift of the block of the ground containing the piles.

(3) For isolated tension piles or a group of tension piles, failure may occur by pulling out of a cone of ground especially for a pile with an oversized base or rock socket.

(4)P To demonstrate that there is adequate safety against failure of piles in tension by uplift of the block of soil containing the piles, as illustrated in Figure 7.1, the following inequality shall be satisfied for all ultimate limit state load cases and load combinations:

$$F_{td} \leq W_d - (U_{2d} - U_{1d}) + F_d \qquad (7.9)$$

where:

F_{td} is the design tensile force acting on the group of piles;

W_d is the design weight of the soil block (including the water) and the piles;

F_d is the design shear resistance at the boundary of the block of soil;

U_{1d} is the design downward force due to the water pressure on the top of the pile foundation;

U_{2d} is the design upward force due to the water pressure on the base of the soil block.

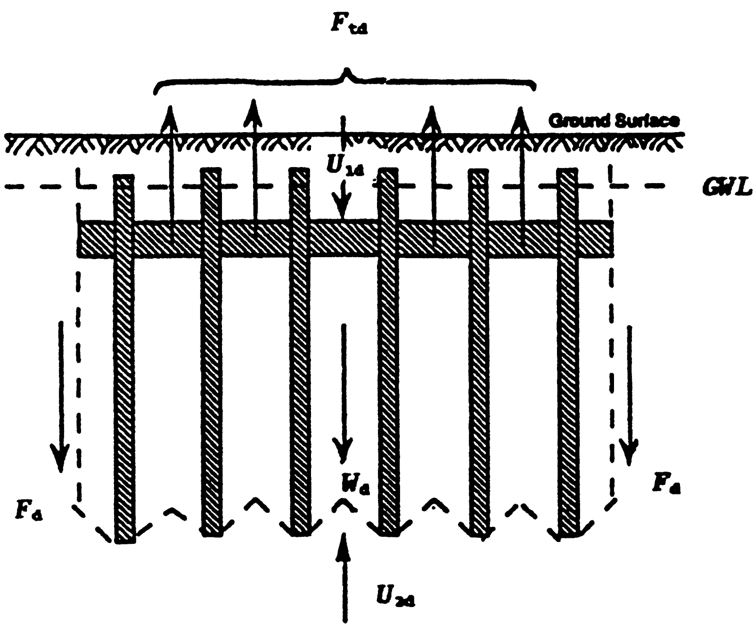

Figure 7.1 — Uplift failure of a group of piles in tension

(5) Normally the block effect will govern the design tensile resistance if the distance between the piles is equal to or smaller than the square root of the product of the pile length and the pile diameter.

(6)P The group effect, which may reduce the effective vertical stresses in the soil and hence the shaft resistance of individual piles in the group, shall be considered when assessing the tensile resistance of a group of piles.

(7)P The severe adverse effect on the tensile resistance of cyclic loading and reversals of load shall be considered.

(8) Comparable experience based on pile load tests should be applied to appraise this effect.

7.7.2.2 ***Ultimate tensile resistance from pile load tests***

(1)P Pile load tests to determine the ultimate tensile resistance R_t of an isolated pile shall be carried out in accordance with **7.5** and with regard to the clauses in **7.6.3.2**.

(2)P When deriving the ultimate characteristic tensile resistance R_{tk} from values R_{tm} 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. As a minimum, both conditions (a) and (b) of Table 7.3 shall be satisfied using the equation:

$$R_{tk} = R_{tm} / \xi \qquad (7.10)$$

Table 7.3 — Factors to derive R_{tk}

Number of load tests	1	2	> 2
(a) Factor ξ on mean R_{tm}	[1.5]	[1.35]	[1.3]
(b) Factor ξ on lowest R_{tm}	[1.5]	[1.25]	[1.1]

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

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.

(4)P The design tensile resistance, R_{td}, shall be derived from:

$$R_{td} = R_{tk} / \gamma_m \qquad (7.11)$$

where: γ_m = [1.6].

(5)P For pile groups, the effect of interaction shall be accounted for when deriving the tensile resistance from the results of load tests on individual test piles.

7.7.2.3 ***Ultimate tensile resistance from ground test results***

(1)P Calculation methods based on ground test results shall only be used when they have been proved by load tests on similar piles, of similar length and cross-section, under comparable soil conditions.

(2)P The design value of the tensile resistance of an isolated tension pile or of a group of tension piles from soil strength parameters shall be evaluated considering the shearing resistance between the pile and the soil in the strata which contribute to the tensile resistance of the pile.

(3) Annex G gives a sample calculation model for the tensile resistance of individual or grouped piles.

7.7.3 Vertical displacement

(1)P Vertical displacements under serviceability limit state conditions shall be assessed and compared against the relevant limiting values for the movement.

(2) This assessment should follow the general principles of **7.6.4**. In general, checking against the ultimate tensile resistance ensures that the vertical displacements will not cause damage to the structure and that a serviceability limit state will not occur. However, in some situations, very severe criteria may be given for the serviceability limit state and a separate check of the displacements may be required.

7.8 Transversely loaded piles

7.8.1 General

(1)P The design of piles subjected to transverse loading shall be consistent with the design rules given in **7.6**, where applicable. Design rules specifically for foundations involving piles subjected to transverse loading are presented in this section.

7.8.2 Ultimate transverse load resistance

7.8.2.1 *General*

(1)P 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:

$$F_{\mathrm{trd}} \leq R_{\mathrm{trd}} \tag{7.12}$$

where:

F_{trd} is the ultimate limit state design transverse load and;

R_{trd} is the ultimate limit state design resistance against transverse loads, taking into account the effect of any compressive or tensile axial loads.

(2) 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.

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

(4) A transverse load applied to a group of piles may result in a combination of compression, tension, and transverse forces in the individual piles.

7.8.2.2 *Ultimate transverse load resistance from pile load tests*

(1)P Pile head horizontal displacement tests shall be carried out in accordance with **7.5** and with regard to the clauses in **7.6.3.2**, where applicable.

(2) Contrary to the load test procedure described in **7.5**, it is normally unnecessary to continue 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 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) The data about 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 effect of interaction and head fixity should be accounted for when deriving the transverse resistance from the results of load tests on individual test piles.

7.8.2.3 *Ultimate transverse load resistance from ground test results and pile strength parameters*

(1)P The transverse resistance of a pile or pile group shall be calculated using a compatible set of structural axial forces, bending moments, shear forces, ground reactions and displacements.

(2)P The analysis of a transversely loaded pile shall include the possibility of structural failure of the pile in the ground in the zone below the ground surface, in accordance with **7.9**.

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

(4)P The degree of freedom of rotation of the piles at the connection with the structure shall be taken into account when assessing the transverse resistance of piles.

7.8.3 Transverse displacement

(1)P The assessment of the transverse displacement of the top 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.

7.9 Structural design of piles

(1)P Piles shall be verified against structural failure in accordance with **2.4**.

(2)P The structure of piles shall be designed to accommodate all the situations to which the piles will be subjected, both during construction, including transportation and driving when applicable, and in use. Piles subject to tension loads shall be designed to carry the full tensile force over their whole length when necessary.

(3)P The structural design shall accommodate construction tolerances as specified for the type of pile, the action components and the performance of the foundation.

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

(5) For piles which are completely embedded in the ground, failure by buckling is not likely to occur.

According to established practice, buckling should be checked for piles in soil layers with characteristic undrained strength of less than 15 kPa.

7.10 Supervision of construction

(1)P A pile installation plan shall be the basis for the construction work.

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

— the pile type with designation if standardized or Technical Approval otherwise;

— the location and inclination of each pile and tolerances on position;

— pile cross section;

— pile length;

— number of piles;

— 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 The installation of all piles shall be monitored and records shall be made at the site and as the piles are installed. A record signed by the supervisor of the work and the pile manufacturer shall be kept for each pile.

(4) The record for each pile should include the following, where appropriate:

— pile type and installation equipment;

— pile number;

— pile cross section, length and (for concrete piles) reinforcement;

— date and time of installation (including interruptions to the construction process);

— concrete mix, volume of concrete used and method of placing for cast-in-situ piles;

— specific weight, pH, Marsh viscosity and fines content of bentonite slurry (when used);

— pumping pressures of the grout or concrete, internal and external diameters, pitch of screw and penetration per revolution (for continuous flight auger piles or other injection piles);

— for driven 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 is critical;

— obstructions encountered during piling;

— deviations of positions and directions and as-built elevations.

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

(6)P If site observations or inspection of records reveal uncertainties with respect to the quality of installed piles, additional investigations shall be carried out to determine the as-built conditions of the piles and whether remedial measures are necessary. These investigations shall include either redriving or pile integrity tests, in combination with soil mechanics field tests adjoining the suspected piles, and static pile load tests.

(7)P 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) Dynamic low strain integrity tests can 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. Since defects like insufficient quality of concrete and thickness of concrete cover, affecting the long term performance of a pile, often cannot be found by dynamic tests, other tests such as sonic tests, vibration tests or coring may be needed in supervising the execution.

Section 8. Retaining structures

8.1 General

(1)P The provisions of this chapter apply to structures which retain ground, similar material or 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 are combined with soil or rock.

(2) In considering the design of retaining structures it may be appropriate to distinguish between the following three main types of retaining structures:

— gravity walls are walls of stone or plain or reinforced concrete, having a base footing with or without heel, ledge or buttress. The gravity of the wall itself, sometimes including stabilising soil or rock masses, 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.;

— embedded walls are relatively thin walls of steel, reinforced concrete or timber, supported by anchors, 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.;

— composite retaining structures include walls composed of elements from the above two types of walls. A large variety of such walls exists. Examples include double sheet pile wall cofferdams, earth structures reinforced by tendons, geotextiles or grouting, structures with multiple rows of ground anchors or soil nails, etc.

8.2 Limit states

(1)P A list of limit states to be considered shall be compiled. As a minimum the following limit states shall be considered for all types of retaining structures:

— loss of overall stability;

— failure of a structural element such as a wall, anchor, wale or strut or failure of the connection between such elements;

— combined failure in ground and in structural element;

— movements of the retaining structure which may cause collapse or affect the appearance or efficient use of the structure, nearby structures or services which rely on it;

— unacceptable leakage through or beneath the wall;

— unacceptable transport of soil grains through or beneath the wall;

— unacceptable change to the flow of groundwater.

(2)P In addition the following limit states shall be considered for gravity retaining structures and for composite retaining structures:

— bearing resistance failure of the soil below the base;

— failure by sliding at the base of the wall;

— failure by toppling of the wall.

and for embedded retaining structures:

— failure by rotation or translation of the wall or parts thereof;

— failure by lack of vertical equilibrium of the wall.

(3)P For all types of retaining structures combinations of the above mentioned limit states shall be considered.

(4) Design of gravity retaining structures often encounters the same type of problems encountered in the design of spread foundations and embankments and slopes. When considering the limit states for gravity retaining structures the principles of chapter 6 should therefore be applied as appropriate. Special care should be taken to account for bearing capacity failure of the ground below the base of the wall under loads with large eccentricities and inclinations, cf. **6.5.4**.

8.3 Actions, geometrical data and design situations

8.3.1 Actions

(1)P In selecting the actions for calculation of limit states, the actions listed in **2.4.2** shall be considered.

8.3.1.1 *Weight of backfill material*

(1)P Design values for the unit weight of backfill material shall be estimated on the basis of knowledge of the material available for backfilling. 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 in the design.

8.3.1.2 *Surcharges*

(1)P Determination of design values for surcharges shall take account of the presence on or near the ground surface of nearby buildings, parked or moving vehicles or cranes, stored granular material, goods, containers, etc.

(2) Care is needed in the case of repeated surcharge loading such as crane rails supported by a quay wall. The pressures induced by such surcharges may significantly exceed those due to the first loading or those resulting from static application of a load of equal magnitude.

8.3.1.3 *Weight of water*

(1)P Design values for unit weight 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.

(2) Local conditions such as salinity and the content of mud may significantly influence the unit weight of water.

8.3.1.4 *Wave forces*

(1)P Design values for wave and wave impact forces shall be selected on the basis of locally available data for the climatic and hydraulic conditions at the site of the structure.

8.3.1.5 *Supporting forces*

(1)P The components of forces caused by prestressing operations shall be regarded as actions. Design values shall be selected taking into account the effect of overstressing the anchor and the effect of a relaxation of the anchor.

8.3.1.6 *Collision forces*

(1)P Determination of design values for impact loads shall take account of the energy absorbed by the retaining system on impact.

(2) For lateral impacts on retaining walls, it is normally necessary to consider the increased stiffness exhibited by the retained ground when resisting an impact on the face of the wall. Furthermore the risk of the occurrence of liquefaction due to lateral impact on embedded walls should be investigated.

(3)P 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.

8.3.1.7 *Temperature effects*

(1)P Design of retaining structures shall take into account the effect of abnormal temperature differences over time and space.

(2) The effects of changes in temperature should especially be considered when determining the loads in struts and props.

Fire effects are dealt with in the Structural Fire Design Parts of the material related Eurocodes.

(3)P The design values for ice forces acting on retaining structures from a sheet of ice covering water shall be calculated taking into account:

- — the initial temperature of the ice before warming begins;
- — the rate at which the temperature increases;
- — the thickness of the ice.

(4)P 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.

8.3.2 Geometrical data

(1)P Design values for the geometrical data shall be derived in accordance with the principles stated in **2.4.5**.

8.3.2.1 ***Ground surfaces***

(1)P Design values for the geometrical data concerning the backfill behind the retaining structure 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) Where the stability of a retaining wall depends on the passive resistance of the ground in front of the structure, the ground level of the passive soil should be lowered by an amount Δ_a, in ultimate limit state calculations. For a cantilever wall, Δ_a should equal (10 %) of its height and for a supported wall, Δ_a should equal (10 %) of the height beneath the lowest support with Δ_a limited to a maximum of 0.5 m.

8.3.2.2 ***Water levels***

(1)P The selection of design values for the geometrical data which determine the regime of free water and the groundwater regime shall be made on the basis of locally available data for the hydraulic and hydrogeological conditions at the site of the retaining structure.

(2)P Account shall also be taken of the effect of variations of permeability on the groundwater regime. The possibility of adverse water pressure conditions due to the presence of perched or artesian water tables shall be considered.

8.3.3 Design situations

(1)P For the design of retaining structures the following items shall be considered:

- — variation in soil properties with time and space;
- — variations in water levels and pore water pressure in time;
- — variation in actions and in the way they are combined;
- — excavation, scour or erosion in front of the retaining structure;
- — backfilling behind the retaining structure;
- — the effect, if anticipated, of future structures and surcharge loadings;
- — ground movements due to subsidence, frost actions, etc.

(2) For waterfront structures ice forces and wave forces need to be applied simultaneously at the same point.

8.4 Design and construction considerations

(1)P In the design, both ultimate and serviceability limit states shall be considered, using a combination of the approaches mentioned in **2.1**.

(2) 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 these cases it is appropriate to use the observational method for the design of retaining structures.

For many earth retaining structures, a critical limit state occurs when the wall is displaced far enough to cause damage to nearby structures or services. Although collapse of the wall may not be imminent, the degree of damage caused in this way may considerably exceed a serviceability limit state in the supported structure. However, the design methods and factors of safety required by this code for ultimate limit state design are often sufficient to prevent the occurrence of this type of limit state provided the soils involved are at least medium dense or firm, and adequate construction methods and sequences are adopted. Special concern is, however, required by some highly overconsolidated clay deposits in which large at rest horizontal stresses may induce substantial movements in a wide area around excavations.

(3)P 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 and construction;
  - — disturbance of the ground due to driving or boring operations;

— provision of access for construction;

— required degree of watertightness of the finished wall;

— the practicability of constructing the wall to reach a stratum of low permeability and so to form a water cut-off. The resulting equilibrium ground water flow problem shall be assessed;

— the practicability of forming ground anchors in adjacent ground;

— the practicability of excavating between the propping of retaining walls;

— the ability of the wall to carry vertical load;

— ductility of structural components.

(4) As far as possible, retaining walls should be designed in such a way that adequate warning of danger (i.e. approaching an ultimate limit state) is given by visible signs. The design should guard against the occurrence of brittle failure, e.g. sudden collapse without conspicuous preliminary deformations;

— access for maintenance of the wall itself and any associated drainage measures;

— the appearance and durability of the wall and the 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 whilst 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**.

(5)P If the safety and serviceability of the design depends on the successful performance of drainage, the consequences of failure of the drainage system shall be considered, having regard for both damage to life 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 the water which will emerge, that the drainage system will operate adequately without maintenance.

(6) The seepage quantities, pressures and eventual chemical content of the emerging water should be considered.

8.5 Determination of earth and water pressures

8.5.1 Design earth pressures

(1)P Determination of design earth pressures shall take account of the mode and amount of movement and strain which is acceptable and which may take place for the retaining structure at the limit state under consideration.

(2) In the following context the words “earth pressure” will also denote the pressure from soft and weathered rocks and includes the pressure of groundwater.

Pressure from granular material stored in silos should be calculated from ENV 1991-4 Eurocode 1 Actions on silo’s.

(3)P Calculations of the magnitudes and directions of design earth pressures shall further 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 unit weight of the ground;

— the rigidity of the wall and the supporting system;

— the wall roughness.

(4) The amount of mobilised wall friction and adhesion is a function of:

— the strength parameters of the ground;

— the friction properties of the wall-ground interface;

— the direction of movements of the wall with respect to the ground and the amount of relative wall-ground movement;

— the ability of the wall to support the vertical forces implied by the wall friction and adhesion.

The amount of shear stress which can be mobilised at the wall-ground interface is limited by the wall-ground interface parameters δ and a. For a completely smooth wall $\delta = 0$ and $a = 0$ and for a completely rough wall $\delta = \phi$ and $a = c$.

A concrete or steel sheet piled wall supporting sand or gravel material normally may be assumed to have $\delta = k\phi$ and $a = 0$, where ϕ due to the disturbance at the ground wall interface should not exceed the critical state angle of friction of the ground and where k should not exceed the value 2/3 for precast concrete or steel sheet piling while it may assume the value 1 for concrete cast against soil. A steel sheet pile in clay under undrained conditions normally should be assumed to have $\delta = 0$ and $a = 0$ immediately after driving. Regeneration may take place over a period of time.

(5)P The design magnitudes and directions of earth pressures shall be calculated in accordance with the specifications given in Table 2.1 using design values of ground parameters which are appropriate to the limit state being considered.

(6) The design 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 can not be characterised by a single characteristic value.

(7)P In the case of retaining structures for rock masses, calculations of the ground pressure shall take account of the effect of discontinuities with particular attention to their orientation, aperture, roughness and the mechanical characteristics of any fill material.

(8)P In the case of retaining structures for swelling ground, calculation of the earth pressure shall take account of the swelling potential of the ground.

(9) The swelling pressures from cohesive soils depend on plasticity, placement water content and hydraulic boundary conditions.

8.5.2 At rest values of earth pressure

(1)P When no movement of the wall relative to the ground takes place, the earth pressure shall be calculated from the state of stress at rest. The determination of the at rest state shall take account of the stress history of the ground.

(2) At rest conditions will normally exist in the ground behind a retaining structure when the movement of the structure is less than $5 \cdot 10^{-4} \cdot H$ for normally consolidated soil.

For a horizontal ground surface, the at rest pressure coefficient K_o, which expresses the ratio between the horizontal and vertical (i.e. overburden) effective stresses, may be determined by:

$$K_o = (1 - \sin \phi') \cdot \sqrt{R_{oc}} \tag{8.1}$$

where R_{oc} denotes the overconsolidation ratio. The formula should not be used for extreme high values of R_{oc}.

If the ground is shelving upward from the wall by an angle $\beta \leq \phi'$ with the horizon, the horizontal component of the effective earth pressure σ_{ho} may be related to the effective overburden pressure by the ratio $K_{o}\beta$ taken equal to:

$$K_{o\beta} = K_o (1 + \sin \beta) \tag{8.2}$$

The direction of the earth pressure force may then be assumed to be parallel to the ground surface.

8.5.3 Limit values of earth pressure

(1)P Limit values of earth pressure are passive or active pressures which occur when the shear strength of the ground is fully mobilized and there is no impediment to the necessary type and amount of movement of ground or wall.

Annex G gives sample procedures for calculating limit values of earth pressure.

(2)P When struts, anchors or similar elements impose kinematical conditions on the structure, the limit values yield only a possible and not necessarily most adverse (or economical) distribution of earth pressures.

(3)P It shall be demonstrated that vertical equilibrium can be achieved for the assumed pressure distributions. If not, the wall friction parameters must be reduced on one side of the wall.

8.5.4 Intermediate values of earth pressure

(1)P Intermediate values of earth pressure occur when the wall movements are insufficient to mobilise the limit values. The determination of the intermediate value of earth pressure shall take account of the amount of wall movement and its direction relative to the ground.

(2) The movement needed for development of an active limit state in noncohesive ground of at least medium dense consistency is of the following order of magnitude:

- rotation about the top $0.002 \cdot H$
- rotation about the toe $0.005 \cdot H$
- translative motion $0.001 \cdot H$

where:

H is the height of the wall.

The intermediate values of earth pressures may be calculated using various empirical rules, spring constant methods, finite element methods, etc.

8.5.5 Compaction effects

(1)P An additional earth pressure is incurred if the wall is backfilled in layers and the fill is compacted. The determination of the additional earth pressure shall take account of the compaction procedure.

(2) Measurements indicate that the additional pressure depends on the applied energy, the thickness of the compacted layers, and the travel pattern of the compaction plant. However, compaction is reduced when the next layer is placed and compacted. When backfilling is complete, the excess pressure acts normally on the upper part of the wall.

(3)P Appropriate compaction procedures shall be applied during execution with the aim of avoiding excessive earth pressures which may lead to excessive movements for the structure.

8.5.6 Water pressures

(1)P Determination of design water pressures shall take account of the water levels above the ground and the groundwater level.

(2)P When checking the ultimate and serviceability limit states, the water pressures shall be accounted for in the combinations of actions in accordance with **2.4.2** and **8.4** (5)P.

(3)P For structures retaining soil of medium or low permeability (silts and clays), water pressures shall be assumed to act behind the wall corresponding to a water table assumed to be at a level no lower than the top face of the material with low permeability unless a reliable drainage system is installed or infiltration is prevented.

(4)P Where sudden changes in a free water level may occur, both the non-steady condition occurring immediately after the change in water table and the steady condition shall be examined.

(5)P Where no special drainage or flow prevention measures are taken, the possible effects of water filled tension or shrinkage cracks shall be considered.

(6) For these circumstances, in retained cohesive soils, the design total pressure should normally not be less than the pressure of water increasing hydrostatically from zero at the ground surface.

8.6 Ultimate limit state design

8.6.1 General

(1)P The design of retaining structures shall be checked at the ultimate limit state using the design actions and design situations appropriate to that state, as specified in **8.3**.

(2)P All relevant limit modes shall be considered.

(3) As a minimum, limit modes of the types illustrated in Figure 8.1 to Figure 8.6 for the most commonly used retaining structures should be considered.

(4)P Calculations for ultimate limit states shall establish that equilibrium can be achieved using the design actions and the design strengths as specified in **2.4.2** and **2.4.3**. Compatibility of deformations in the materials involved in a calculation shall be considered in assessing design strengths.

(5)P Upper or lower design values shall be used for the strength of the ground, whichever is more adverse.

(6)P Calculation methods may be used which redistribute earth pressure in accordance with the relative displacements and stiffnesses of ground and structural elements.

(7)P For fine grained soils, both short term and long term behaviour shall be considered.

(8)P For walls subject to differential water pressures, safety against failure due to hydraulic instability (erosion) shall be checked.

8.6.2 Overall stability

(1)P The principles in section 9 shall be used as appropriate to demonstrate that an overall stability failure will not occur and that the corresponding deformations are sufficiently small.

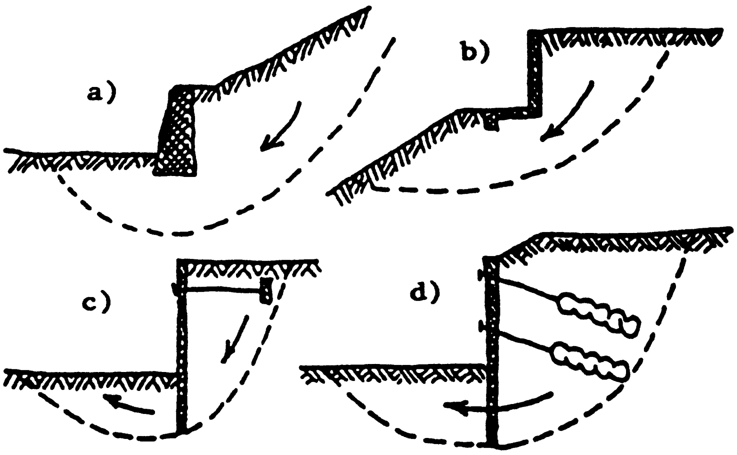

Figure 8.1 — Examples of limit modes for overall stability of retaining structures

(2) As a minimum, limit modes of the types illustrated in Figure 8.1 should be considered taking progressive failure and liquefaction into account.

8.6.3 Foundation failure of gravity walls

(1)P The principles of section 6 shall be used as appropriate to demonstrate that a foundation failure is sufficiently remote and that corresponding deformations will be small. Both bearing resistance and sliding shall be considered.

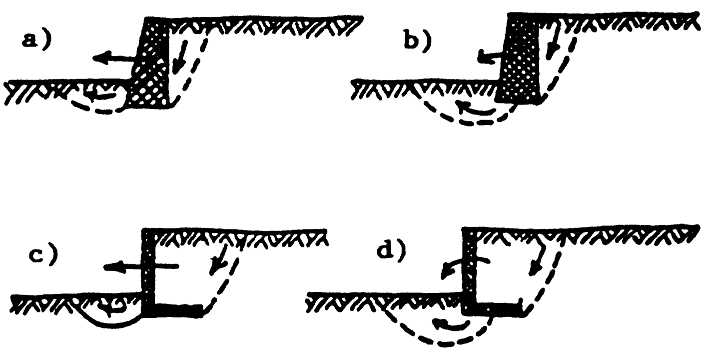

Figure 8.2 — Examples of limit modes for foundation failures of gravity walls

(2) As a minimum, limit modes of the types illustrated in Figure 8.2 should be considered.

8.6.4 Rotational failure of embedded walls

(1)P It shall be demonstrated by equilibrium calculations that embedded walls have sufficient penetration into the ground to prevent rotational failure.

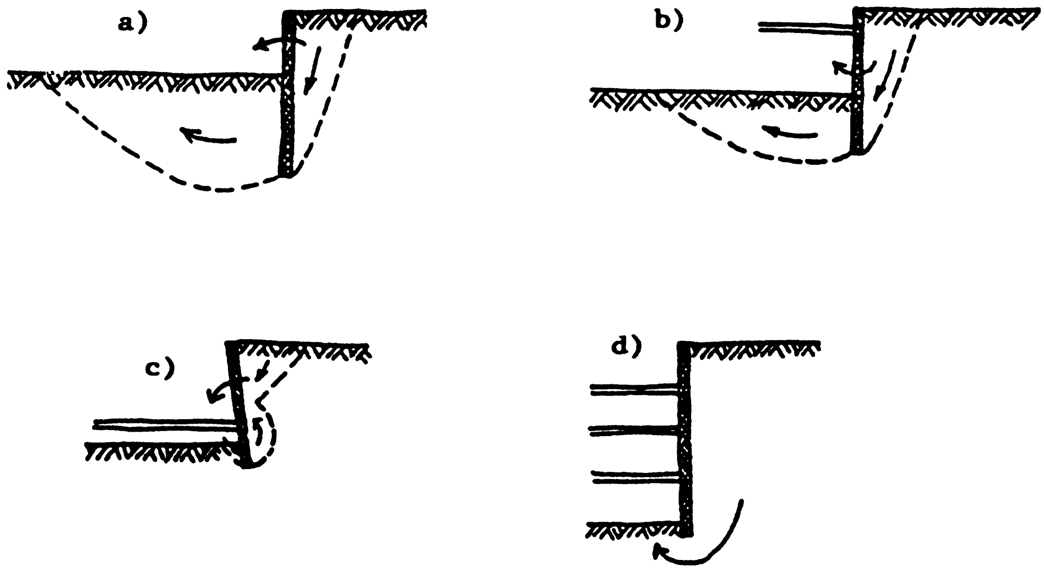

Figure 8.3 — Examples of limit modes for rotational failures of embedded walls

(2) As a minimum, limit modes of the types illustrated in Figure 8.3 should be considered.

(3)P 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.

8.6.5 Vertical failure of embedded walls

(1)P It shall be demonstrated that vertical equilibrium can be achieved using the design soil strengths and design vertical forces on the wall.

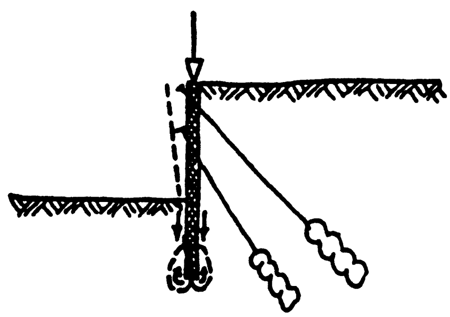

Figure 8.4 — An example of a limit mode for vertical failure of embedded walls

(2) As a minimum, the limit mode of the type illustrated in Figure 8.4 should be considered.

(3)P Where downward movement of the wall is considered, upper design values shall be used in this calculation for prestressing forces, such as those from ground anchors, which have a vertical downward component.

(4)P The design magnitude and direction of shear stress between the soil and the wall shall be consistent with the check for rotational failure.

(5) Vertical and rotational equilibrium should be checked with the same design values for shear stress on the wall.

(6)P If the wall acts as the foundation of a structure, vertical equilibrium shall be checked using the principles of section 7.

8.6.6 Structural design of retaining structures

(1)P Retaining structures including their supporting structural elements such as anchors and props shall be verified against structural failure in accordance with **2.4.2**.

It shall be demonstrated that equilibrium can be achieved without exceeding the design strengths of the wall and of supporting structural elements suchs as props and anchors.

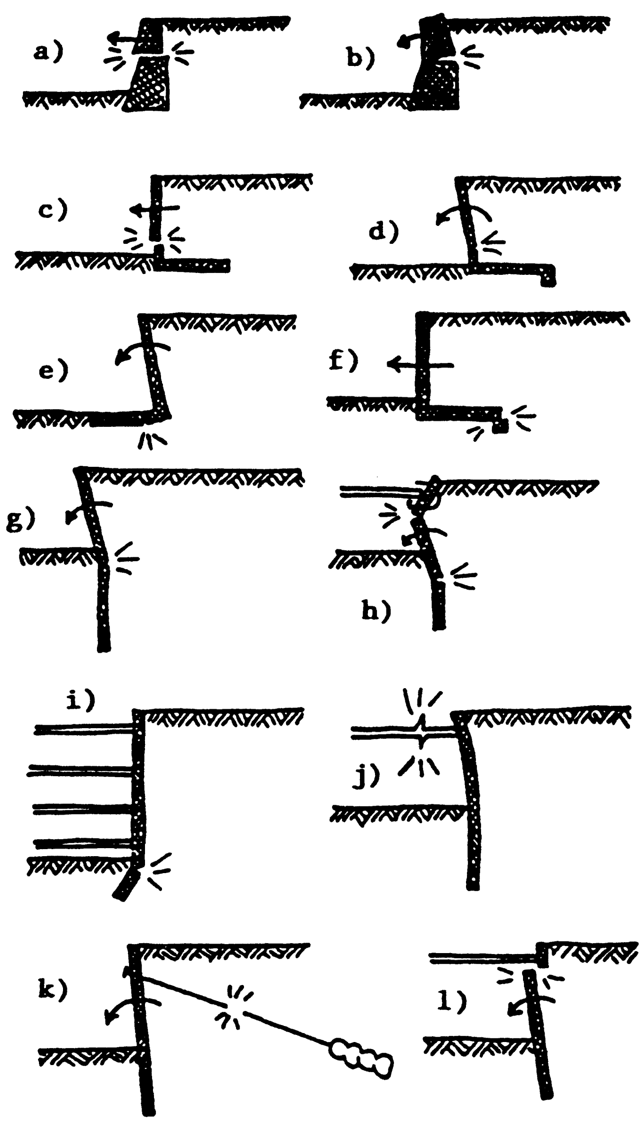

Figure 8.5 — Examples of limit modes for structural failure of retaining structures

(2) As a minimum, limit modes of the types illustrated in Figure 8.5 should be considered.

(3)P For each ultimate limit state it shall be demonstrated that the required strengths can be mobilised in the ground and the structure with compatible deformations.

(4) 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 the material-related Eurocodes. In the ground, loss of strength due to dilation in dense granular soils and formation of polished surfaces in clays should be considered.

The design action effects (moments, internal forces, etc.) in retaining structure elements shall be calculated on the basis of design earth pressures and design ground resistances and shall then be multiplied by a model factor γ_{sd}. The design resistance of the retaining structure element shall be calculated from the relevant material-related Eurocode.

(5)P For each ultimate limit state, it shall be demonstrated that the required strengths can be mobilised in the ground and the structure with compatible deformations.

8.6.7 Failure by pull-out of anchors

(1)P It shall be demonstrated that equilibrium can be achieved without pull-out failure of ground anchors.

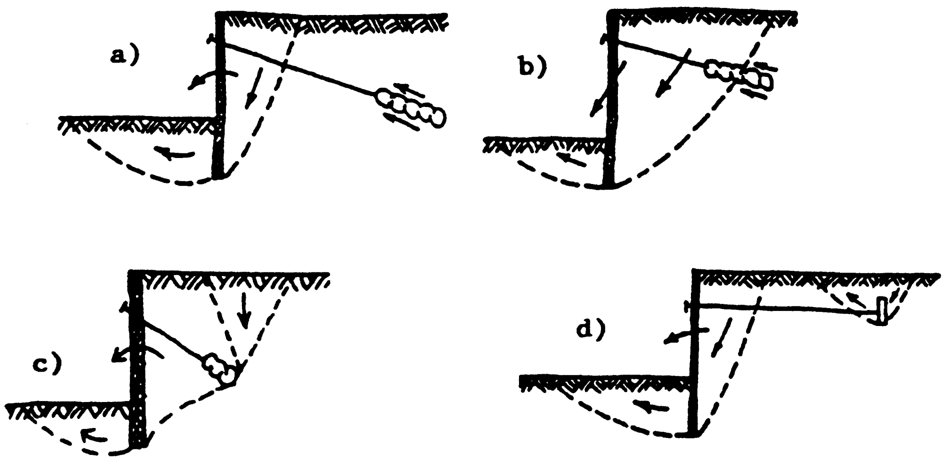

Figure 8.6 — Examples of limit modes for failure by pull-out of anchors

(2) Grouted anchors should be designed in accordance with section 8.8. As a minimum, limit modes of the types illustrated in Figure 8.6[a), b), c)] should be considered.

For "dead men" anchors, the failure mode illustrated in Figure 8.6 d) should be considered. Calculation of the pull-out capacity of the anchor should be based on passive resistance with no allowance for wall friction δ.

Where anchors are closely spaced, forming parallel or overlapping sets, interaction between anchors and the possible failure of the complete group should be considered.

8.7 Serviceability limit state

8.7.1 General

(1)P The design of retaining structures shall be checked at the serviceability limit state using the appropriate design situations as specified in **8.3**.

8.7.2 Displacements

(1)P Limiting values for the allowable displacements for walls and the ground adjacent to them shall be established in accordance with **2.4.5**, taking into account the tolerance of supported structures and services to displacement.

(2)P 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. It shall be verified that the estimated displacements do not exceed the limiting values.

(3)P 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 If the estimated displacements exceed 50 % of the limiting values, 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 the wall retains more than 6 m of soil of low plasticity or 3 m of soils of high plasticity;

— where the wall is supported by soft clay within its height or beneath its base;

— where comparable experience is not well established.

(5)P Displacement calculations shall take account of the stiffness of the ground and structural elements and the sequence of construction.

(6) 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.

8.7.3 Vibrations

(1)P The provisions of **6.6.2** also apply to retaining structures.

8.7.4 Structural serviceability limit states

(1)P Design earth pressures for the serviceability limit state check of the structural elements shall be derived using characteristic values of all soil parameters.

(2) The assessment of design earth pressures should take into account the initial stress, stiffness and strength of the ground and the stiffness of the structural elements.

The design earth pressures should be derived taking account of the allowable deformation of the structure at its serviceability limit state. These will not necessarily be limiting active and passive values.

8.8 Anchorages

8.8.1 General

(1)P This section is concerned with any type of anchorage used to support a retaining structure by transmitting a tensile force to a load-bearing formation of soil or rock.

(2) Such anchorages include

— installations consisting of an anchor head, a free anchor length and a fixed anchor length, established by grouting;

— installations consisting of an anchor head and a fixed anchor length, but without a free anchor length (soil nails);

— installations consisting of an anchor head, a free anchor length and a reinforced concrete or steel anchor;

— installations consisting of a screw anchor and an anchoring cap.

Ground anchorages may be employed as temporary or as permanent elements of a retaining structure.

8.8.2 Anchorage design

(1)P Anchorage design shall take into account all circumstances during the foreseeable design life of the anchorage. The corrosion and creep of permanent anchorages shall be considered.

Preferably anchorage systems for which successful long-term experience has been documented with respect to performance and durability should be used.

(2)P The ground investigation prior to the design and execution of anchorage shall include those ground formations outside the actual site if the tensile forces are transferred there.

(3)P Anchorages which are going to be in use for more than two years shall be designed as permanent anchors.

(4)P To check an anchorage against the ultimate limit states, three failure mechanisms shall be analysed'

— the failure of the tendon or anchor head in terms of their material strength or the failure of bonding at internal interfaces;

— the failure of the anchorage at the tendon-grout or grout-ground interface; the design pull-out resistance shall exceed the anchorage design load;

— the overall stability failure of the structure including the anchorages according to the principles given in **8.6**.

(5) The pull-out resistance for a given design situation depends on the geometry of the anchorage, but the transfer of stresses to the surrounding ground is influenced by the execution technique.

This particularly holds for grouted anchorages where the procedure and to a lesser extent the chosen drilling technique and method of flushing is important.

(6)P Steel tendons and bars used for anchors shall be designed according to the principles of ENV 1993-1 Eurocode 3 Design of steel structures.

(7) The minimum free anchor length should be about 5 m.

8.8.3 Construction considerations

(1)P The connection between the tendon and the wall shall be able to adjust to the movements occurring in service.

(2)P Permanent anchorages shall be protected against corrosion over the full length of the tendon and the anchor head. The environmental conditions during the lifetime of the anchor shall be taken into account.

(3) The following criteria are considered to be indicative levels in excess of which special precautions are needed against water aggressiveness to concrete or hardened cement grout:

— pH values	less than 5,5;
— carbon acid CO_2	more than 40 mg/l;
— ammonium NH_4	more than 30 mg/l;
— magnesium	more than 1 000 —mg/l;
— sulphate SO_4	more than 200 mg/l;
— hardness	less than 30 mg CaO/1.

Besides the protection against corrosion, a mechanical protection is usually required for permanent anchors to avoid damage of the corrosion protection during transportation, installation and stressing.

8.8.4 Anchorage testing

(1)P The load carrying resistance of an anchorage shall be evaluated from test results and from local experience. The following load tests may be carried out on-site on anchors:

— assessment tests;

— acceptance tests.

(2)P Assessment tests shall be carried out in advance of the main contract or on selected working anchorages during the course of construction with the aim of assessing the suitability of the anchorage system to provide the necessary anchorage resistance under the soil conditions in question. They also provide criteria for the acceptance tests.

(3)P Acceptance tests shall be carried out to demonstrate that each of the anchorages installed has the capacity to carry the load assumed in the design.

(4)P The method used for the installation of anchorages submitted to onsite assessment tests shall be fully documented in accordance with **8.8.7**.

(5)P Between the time of installation of an anchorage and the beginning of a load test, adequate time shall be allowed to ensure that the required quality of the bond at the tendon-grout (or, where relevant, grout-encapsulation) and grout-ground interface is achieved.

(6)P It shall be checked that all equipment and measuring apparatus used for anchorage testing is sensitive, accurate and in perfect working order.

8.8.5 Assessment tests

(1)P At least one assessment test shall be carried out for each distinct condition for ground and construction unless comparable experience exists.

(2) On large anchorage projects the number of assessment tests per ground condition should be at least 1 % for temporary anchorages where failure will have few serious consequences and at least 2 % in the case of permanent anchorage or temporary anchorages where the consequences of failure are serious.

(3)P The test duration shall be sufficient to ensure that prestress or creep fluctuations stabilize within tolerable limits.

(4)P When deriving the ultimate characteristic anchorage resistance R_{ak} from values R_{am} measured in one or more assessment tests, an allowance shall be made for the variability of the ground and the variability of the effect of pile installation. As a minimum, both conditions (a) and (b) of Table 8.1 shall be satisfied using the equation:

$$R_{ak} = R_{am} / \xi \qquad (8.4)$$

Table 8.1 — Conversion factors ξ to derive R_{ak}

Number of assessment tests	1	2	> 2
(a) ξ on mean R_{am}	[1.5]	[1.35]	[1.3]
(b) ξ on lowest R_{am}	[1.5]	[1.25]	[1.1]

The anchorage resistance, R_a, obtained from the assessment tests shall be equal to the lowest of the loads corresponding to the first two failure mechanisms referred in **8.8.2** (4)P and the creep limit load.

(5) The systematic and random components of the variations in the ground should be distinguished in the interpretation of the assessment tests.

Account of the systematic components of ground variability, can be made considering different zones of homogeneous conditions or a trend of ground conditions with position on the site. Then the data about the installation of the anchorages should be checked, and any deviations from the normal execution conditions should be accounted for. Such variations should be covered in part by a correct selection of the assessment tests.

(6)P The design resistance, R_a, shall be derived from:

$$R_a = R_{ak} / \gamma_m \qquad (8.4)$$

where:

γ_m = [1.25] for temporary anchorages and;

γ_m = [1.5] for permanent anchorages.

The design resistance is then compared with the largest ultimate limit state design load which is going to be carried by the anchorage.

(7)P The assessment test procedure, 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 anchorage resistance, the creep limit load, and the apparent free tendon length.

8.8.6 Acceptance tests

(1)P All grouted anchorages shall be subjected to an acceptance test before they become operational and prior to the lock-off procedure.

(2)P Acceptance testing shall be performed in compliance with standard procedures and acceptance criteria which are derived from the results of the assessment tests with the aim of proving the ability of each anchorage to support the maximum limit state load.

(3)P The test procedure shall provide confirmation of the apparent free tendon length and that the relaxation of anchor force after lock-off will be acceptably small.

(4) The acceptance test loading may be used to pre-load the anchorage in order to minimise future relaxation of anchor force.

8.8.7 Supervision of construction and monitoring

(1)P An anchorage installation plan shall be available on site containing the technical specification related to the anchorage system to be used.

(2) An anchorage installation plan may contain the following information, as appropriate:

— the anchorage type with designation if standardized by European Technical Approval;

— number of anchorages;
— the location and orientation of each anchorage and tolerances in position;
— anchorage length;
— date and time of installation of each anchorage;
— for grouted anchorage; material, pressure, grouted volume, grouting length, grouting time;
— required anchorage load carrying capacity;
— installation of the chosen corrosion protection;
— installation technique (drilling, placing, bonding and stressing);
— known obstructions;
— any other constraints on anchoring activities.

(3)P The installation of all anchorages shall be monitored and records shall be made at the site and as the anchorages are installed. A signed record shall be kept for each anchorage.

(4)P If inspection reveals uncertainties with respect to the quality of installed anchorages, additional investigations shall be carried out to determine the as-built conditions of the anchorages.

(5)P Records shall be kept after completion of the works. As-built record plans shall be compiled after completion of the anchorages and kept with the construction documents. Certificates for all materials and their relevant properties shall also be held.

Section 9. Embankments and slopes

9.1 Scope

(1)P The provisions of this chapter apply to embankments and slopes but not to dykes and dams.

(2)P Placement and compaction of fill are considered in section 5 and retaining structures which provide support for slopes are considered in section 8.

9.2 Limit states

(1)P In order to fulfil the fundamental requirements for embankments and slopes of stability, limited deformation, durability and limitation of damage to nearby structures or services, the following limit states shall be considered:

— loss of overall stability or bearing resistance;

— failure due to internal erosion;

— failure due to surface erosion or scour;

— failure due to hydraulic uplift;

— deformations (including those due to creep) of the embankment or slope and their foundations which cause structural damage in adjacent structures, roads or services;

— rockfalls;

— deformations of the embankment or slope, including those due to creep, which cause loss of serviceability;

— surface erosion.

9.3 Actions and design situations

(1)P In selecting the actions for calculation of limit states, the list in **2.4.2** (4)P shall be considered.

(2)P The effects of the following processes shall be considered:

— construction processes, such as excavation in front of the slope or the placing of an embankment, the effect of vibrations caused by rock blasting, piling, etc.;

— the effect of structures which are expected to be placed on or near the embankment or slope after its completion;

— the effect of the new slope on existing work;

— the effect of any previous or continuing movement of existing slopes;

— the effects of overtopping, waves and rain on the slopes and crest of embankments (erosion);

— effects of the temperature on the embankment slopes (shrinkage);

— animal activities causing clogging of drains or digging of holes in the ground.

(3)P The design free water level in front of the slope and the design groundwater level, or their combination, shall be chosen from the available hydrological data to give the most unfavourable conditions that could occur in the design situation being considered. The possibility of failure of drains, filters or seals shall be considered.

(4) For embankments along water the most unfavourable hydraulic conditions are normally steady seepage for the highest possible groundwater level and rapid drawdown of the free water level.

(5)P In deriving design distributions of pore water pressure, account shall be taken of the possible range of anisotropy and variability of the soil.

9.4 Design and construction considerations

(1)P Embankments and slopes shall be designed and constructed taking into account experience of slopes and cuttings in similar ground.

(2) Embankments built on soft cohesive soils are normally raised in increments of height. The thickness of these layers and the speed of construction should be determined during design in order to prevent loss of stability of the slopes or bearing resistance of the foundation during construction. Consolidation time can be calculated only approximately. The rate of consolidation of the soft soil layers should therefore be checked during construction by measuring settlements. Measurement of pore pressures may also be required, the next layer of fill being placed when excess pore pressures have fallen below safe values, which are to be stated in the design report. The results of the settlements measurements are to be used as a check on this procedure. If vertical drains are installed to accelerate the consolidation, and hence the construction, special care should be taken with respect to the location of the pore pressure measuring devices. They should be located in the centre of the grid of vertical drains. The observational method described in **2.7** should be implemented.

(3)P Slope surfaces exposed to serious erosion shall be protected.

(4) Slopes may be sealed or planted, or protected artificially. For slopes with berms, a drainage system within the bern may be needed. Trees or scrubs should generally not be planted on river and lake embankments.

9.5 Ultimate limit state design

9.5.1 Loss of overall stability

(1)P In analysing the stability of an embankment or ground slope (soil or rock masses) all possible failure modes shall be considered.

(2) The mass of soil or rock bounded by the failure surface is normally treated as a rigid body or as several rigid bodies moving simultaneously. Alternatively, stability may be checked by finding a statically admissible stress 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.

Where ground or embankment material is relatively homogeneous and isotropic in strength, it will usually be sufficient to assume circular failure surfaces.

For slopes in layered soils with considerable variations of shear strength, special attention should be paid to the layers with smaller shear strength. This may require analysis of non-circular failure surfaces.

In jointed materials, including hard rock and some soils, the shape of the failure surface depends on the discontinuities and may also pass through intact material. This may require analysis of three dimensional wedges.

(3)P The equilibrium of the body bounded by any possible failure surface shall be verified when the actions and the shear strength parameters of the ground are assigned their design values in accordance with **2.4.2** and **2.4.3**.

(4) In soils and soft rocks which do not exhibit marked strength anisotropy, the method of slices may be used. The method shall verify the overall moment and vertical stability of the sliding mass. If horizontal equilibrium is not checked, interslice forces should be assumed to be horizontal.

A conservative analysis may be performed using earth pressures in embankments calculated according to section 8 and bearing resistance of the foundation from section 6.

(5)P In computations of overall stability for slopes and embankments, Case A of **2.4** may generally be omitted.

(6) Unless there is abnormal uncertainty about soil density, it is not necessary to distinguish between favourable and unfavourable gravity loads in slope stability calculations.

9.5.2 Deformations

(1)P The design shall show that the deformation of the embankment or slope under the design actions will not cause severe damage to structures, transportation networks or services sited on or near the embankment or slope.

(2)P Deformation within the embankment shall be considered together with that of the ground beneath it.

(3) The settlement of an embankment on a compressible soil foundation may be calculated using the principles of **6.6.1**. Special attention should be paid to the settlement-time relationship which includes both consolidation and secondary settlement. Attention should also be paid to the possibility of the occurrence of differential settlements. Since the analytical and numerical methods available at present do not usually provide reliable predictions of the-pre-failure deformation of a slope, the occurrence of ultimate limit states may be avoided by either:

— limiting the mobilized shear strength, or;

— observing the movements and taking action to control them if this becomes necessary.

9.5.3 Superficial erosion, internal erosion and hydraulic uplift

(1)P If steady or temporary seepage of water is possible, the design shall ensure that failure will not be caused by superficial erosion, internal erosion or hydraulic uplift.

(2) The measures most commonly used to ensure that superficial or internal erosion and hydraulic uplift do not occur are:

— seepage control;

— protective filters;

— avoidance of the use of dispersive clays without the protection of adequate filters;

— slope revetments;

— inverted filters;

— relief wells;

— reduction of hydraulic gradient.

In addition, observations of the phreatic surface and of the rate of seepage can be made to check that the slope is performing as intended.

9.5.4 Rockslides

(1)P The risk of slides in rockmasses shall be considered. The rockslides can be planar or rotational depending on the rockmass structure.

(2) Prevention of rockslides can be obtained by providing stable inclination, anchoring, bolting, surface and internal drainage etc.

9.5.5 Rockfalls

(1)P In rock, the risk of rockfalls caused by toppling, wedge failure or spalling shall be considered.

(2) Prevention of rockfalls may be achieved by such techniques as anchoring, bolting or shotcreting. Alternatively rockfalls may be allowed to occur and the damage limited by trapping the falling rocks.

9.5.6 Creep

(1)P The risk of displacements of ground slopes due to creep shall be considered.

(2) Prediction of creep movements is normally difficult and the better prevention is to avoid the utilization of susceptible areas.

9.6 Serviceability limit state design

(1)P The design shall show that the deformation of the embankment or slope under the design actions will not cause loss of serviceability in structures, roads or services sited on or near the embankment or slope.

(2) Application rule **9.5.2** (3) for calculating the settlement of an embankment on a compressible soil layer is also applicable here. Trial embankments may be useful for predicting the behaviour of embankments where serviceability limit states must be prevented.

Compression of the fill due to its own weight or foundation loads will be small provided the fill is well compacted and the foundation loads are light. The possibility of deformations due to changes in the groundwater conditions should be considered. Special attention should be given to possible long term consolidation settlements due to changes in the water content of the fill or the ground beneath the fill.

9.7 Monitoring

(1)P Embankments and slopes shall be monitored using appropriate equipment if either:

— it is not possible to prove by calculation or by prescriptive measures that all of the limit states given in **9.2** will not occur or;

— the assumptions made in the calculations are not based on adequate and reliable data.

(2) Monitoring should follow the principles of section 4.

Monitoring should be used where knowledge is required of:

— groundwater levels or pore pressures in and beneath an embankment or slope so that an effective stress analyses can be carried out, or checked;

— lateral and vertical movements of a moving soil and rock mass in order to predict further deformations;

— the-depth and shape of the sliding 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.

The construction of embankments on soft soil with low permeability shall be monitored and controlled by means of pore pressure measurements in the soft layers and settlement measurements of the fill.

Monitoring should normally be employed for embankments placed in Geotechnical Category 3.

Annex A (informative) Checklist for construction supervision and performance monitoring

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.

A.1 Construction supervision

A.1.1 *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 pressure variations resulting from excavation or loading.

4) Safety of workmen with the due consideration of geotechnical limit states.

A.1.2 *Water flow and pore pressures*

1) Adequacy of system to ensure control of pore water pressures in all aquifers where excess pressures 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.

A.2 Performance monitoring

1) Settlement at established time intervals of buildings and other structures including those due to effects of vibrations, metastable soils.

Settlement observations shall be referred to a stable benchmark.

2) Lateral displacement, distortions especially those related to fills and stockpiles; soil supported structures, such as buildings or large tanks; deep excavations 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 measurement 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: temperature monitoring; ground freezing; frost heave, displacement; effects of subsequent thawing.

7) Watertightness.

Annex B (informative) A sample analytical method for bearing resistance calculation

B.1 General

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.

The following symbols are used in addition to those in **1.6** and **1.7**:

δ	the design base friction angle as discussed in **6.5.3**;
q	the design total overburden pressure at the level of the foundation base;
q'	the design effective overburden pressure at the level of the foundation base;
γ'	the design effective unit weight of the soil below the foundation level, reduced to $\gamma' = \gamma - \gamma_w (1 + l)$ in the case of an upward hydraulic gradient l;
B'	the design effective foundation width;
L'	the design effective foundation length;
$A' = B' L'$	the design effective foundation area, defined as the foundation base or, in the case of an eccentric load, the reduced area of the foundation whose centroid is the point through which the load resultant acts;
s, i	the design values of the dimensionless factors for the shape of the foundation and the inclination of the load, respectively; the subscripts c, q and γ indicate the influences due to cohesion, the surcharge and the weight of the soil; these coefficients are only valid when the shear parameters are independent of direction.

B.2 Undrained conditions

The design bearing resistance is calculated from:

$$R / A' = (2 + \pi)\, c_u\, s_c\, i_c + q \qquad \text{(B.1)}$$

with the design values of dimensionless factors for:

— 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:

$i_c = 0{,}5\,(1 + \sqrt{1 - H / A'\, c_u})$

B.3 Drained conditions

The design bearing resistance is calculated from:

$$R / A' = c' \cdot N_c \cdot s_c \cdot i_c + q' \cdot N_q \cdot s_q \cdot i_q + 0.5 \cdot \gamma' \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma \qquad \text{(B.2)}$$

with the design values of dimensionless factors for:

— the bearing resistance:

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

$N_c = (N_q - 1) \cot \phi'$

$N_\gamma = 2\,(N_q - 1) \tan \phi'$ when $\delta \geq \phi' /2$ (rough base) the shape of foundation:

$s_q = 1 + (B'/L') \sin\phi'$	for a rectangular shape;
$s_q = 1 + \sin\phi'$	for a square or circular shape;
$s_\gamma = 1 - 0.3\,(B'/L')$	for a rectangular shape;
$s_\gamma = 0{,}7$	for a square or circular shape;
$s_c = (s_q \cdot N_q - 1)/(N_q - 1)$	for rectangular, square or circular shape.

— the inclination of the load, caused by a horizontal load H parallel to L'

$i_q = i_\gamma = 1 - H / (V + A'\, c' \cot \phi')$

$i_c = (i_q \cdot N_q - 1) / (N_q - 1)$

— the inclination of the load, caused by a horizontal load H parallel to B':

$i_q = (1 - 0.7\, H /(V + A' \cdot c' \cot \phi))^3$

$i_\gamma = (1 - H / (V + A' \cdot c' \cot \phi'))^3$

$i_c = (i_q \cdot N_q - 1) / (N_q - 1)$

The additional influences of embedment depth, inclination of the base of the foundation and of the ground surface should also be considered.

Annex C (informative) A sample semi-empirical method for bearing resistance estimation

To estimate the design bearing resistance of a foundation on soil semiempirically, in-situ tests such as the pressuremeter test may be used. When using the pressuremeter the design bearing resistance of a foundation subjected to a vertical load is related to the limit pressure of the soil by the linear function:

$$R/A' = q + k\, p_1^* \tag{C.1}$$

where the following symbols are used:

- A' the design effective foundation area as in Annex B;
- q the design total overburden pressure at the level of the foundation base;
- k the bearing resistance factor with numerical values in the range of 0,8 to 3 depending on the type of soil, the embedment depth and the shape of the foundation;
- p_1^* the design net equivalent limit pressure; the net limit pressure p_1^* 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_o at the level of the test; p_o may be determined from an estimate of the at rest earth pressure coefficient K_o and from the values of the effective overburden pressure q' and the pore water pressure u as $p_o = K_o\, q' + u$.

Annex D (informative) Sample methods for settlement evaluation

D.1 Stress-strain Method

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.

D.2 Adjusted Elasticity Method

The total settlement of a foundation on cohesive or non-cohesive soil may be evaluated using elasticity theory and an equation of the form:

$$s = p \cdot B \cdot f / E_m \tag{D.1}$$

where the following symbols are used:

- p the serviceability limit state bearing pressure linearly distributed on the base of the foundation, which for normally consolidated cohesive soils should be reduced by the weight of the excavated soil above the base; buoyancy effects should also be taken into account;
- E_m the design drained Young's modulus of the deforming stratum for drained conditions. If no useful settlement results measured at neighbouring similar structures in similar conditions are available to evaluate E_m, it may be estimated from the results of laboratory or in situ tests;
- f a coefficient whose value 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;
- B the width of the foundation.

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.

D.3 Settlements without drainage

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.

D.4 Settlements caused by consolidation

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.

D.5 Time-settlement behaviour

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 insitu tests in accordance with **3.3.10**.

Annex E (informative)
A sample method for deriving presumed bearing resistance for spread foundations on rock

For weak and broken rocks with tight joints, including chalk with porosity less than 35 %, presumed bearing resistance may be derived from Figure E.1. This is based on the grouping given in Table E.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 E.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

For chalk with porosity greater than 35 %, values of presumed bearing resistance may be derived from Table E.2.

Table E.2 — Classification and presumed bearing resistance for high porosity chalk

Grade	Brief description	Presumed bearing resistance kPa
V	Structureless remoulded chalk containing lumps of intact chalk. Dry chalk above the water table	125 to 250
IV	Rubbly partly-weathered chalk with bedding and jointing. Joints 10 mm to 60 mm apart, open to 20 mm, and often infilled with soft remoulded chalk and fragments	250 to 500
III	Rubbly to blocky unweathered chalk. Joints 60 mm to 200 mm apart, open to 3 mm, and sometimes infilled with fragments	500 to 750
II	Blocky medium-hard (weak) chalk. Joints more than 200 mm apart and closed	750 to 1 000
I	As for grade II, but hard (moderately weak) and brittle	1 000 to 1 500

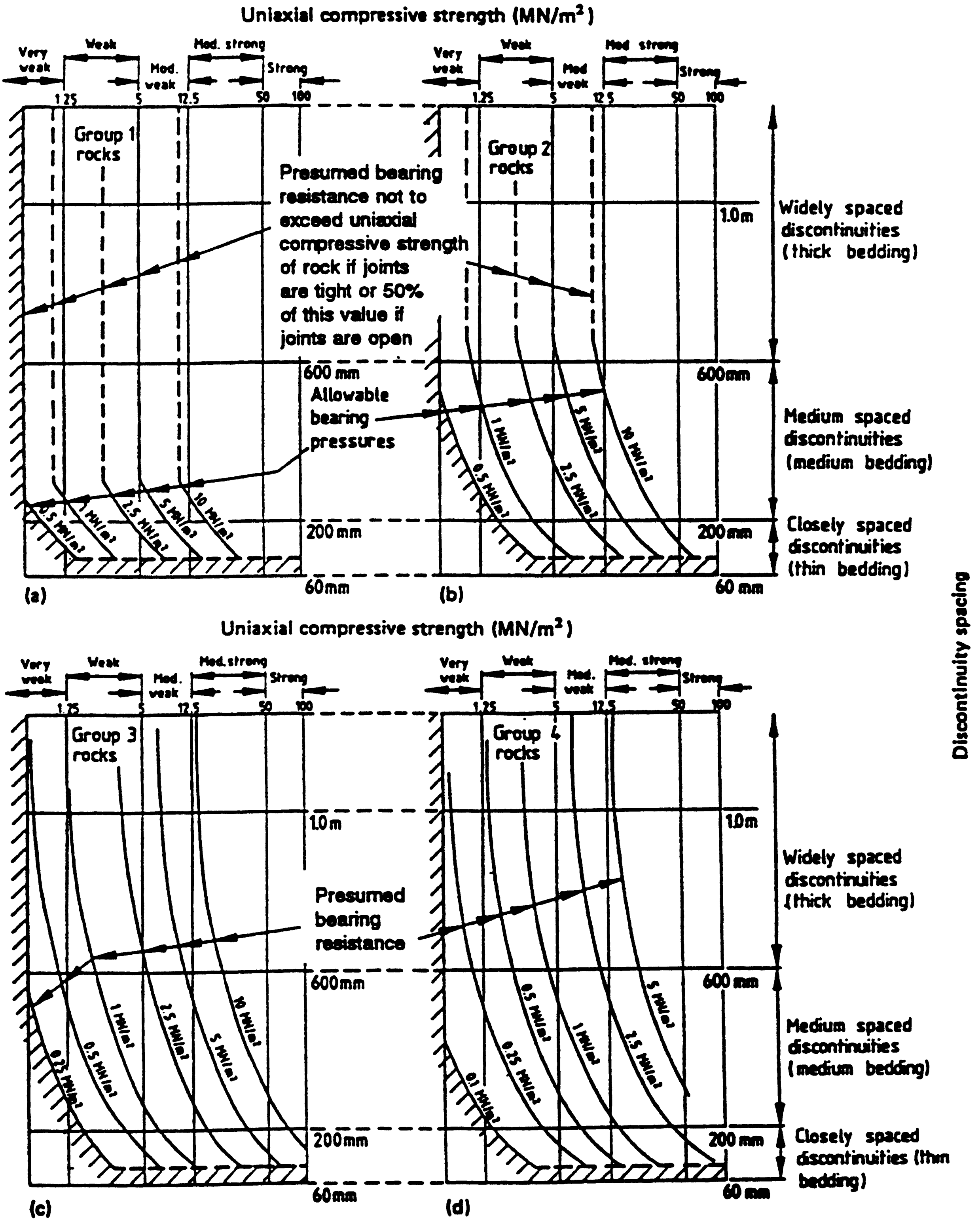

Figure E.1 — Presumed bearing resistance for square pad foundations bearing on rock (for settlements not exceeding 0,5 % of foundation width). For types of rock in each of four groups, see Table E.1. Presumed bearing resistance in hatched areas to be assessed after inspection and/or making tests on rock

Annex F (informative)
A sample calculation model for the tensile resistance of individual or grouped piles

The following calculation model, shown in Figure F.1, may be used to check the tensile resistance of individual or grouped piles. The following symbols are used in addition to those in **1.6** and **1.7**:

F_t	is the tensile load on each pile;
$F_t\,(z)$	is the tension in pile at depth z;
$q_s\,(z)$	is the design shaft resistance at depth z;
u(z)	is the design pore water pressure at depth z;
p	is the pile perimeter;
s_{eq}	is the pile spacing assuming a regular array of piles or equivalent spacing for piles placed in an non-regular array.

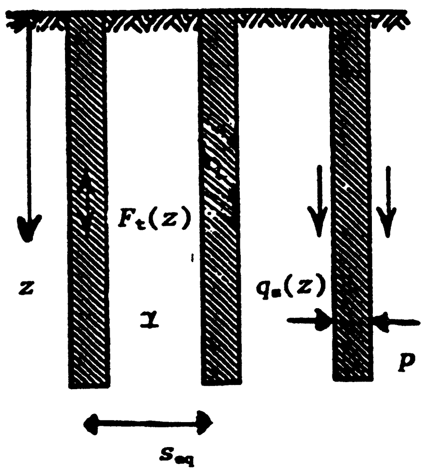

Figure F.1 — Model to check tensile resistance of individual or grouped piles

The design is satisfactory if a distribution of tension, $F_t(z)$ can be found which satisfies the following requirements:

— At the top of the pile (z = 0): $F_t(0) = F_t$

— At the base of the pile (z = L): $F_t(L) = 0$

— For grouped piles, at all depths z, $F_t(z)$ is limited by the weight of soil above depth z:

$$F_t(z) \le F_t - \left[\int_o^z \Upsilon \, dz - u(z)\right] s_{eq}^2 \tag{F.1}$$

— An allowance for shear forces on the perimeter of the group may be included.

— At all depths z, the gradient of $F_t(z)$ is limited by the shaft resistance:

$$\left|\frac{dF_t(z)}{dz}\right| = -\frac{dF_t(z)}{dz} \le p\, q_s(z) \tag{F.2}$$

— In homogeneous, ground, the resistant soil block always extends down to the depth of the pile base.

— When calculating $q_s(z)$ allowance should be made for its dependence on the effective vertical stresses in the ground between the piles. These stresses are influenced in an unfavourable way by the pile tensile load F_t.

— The value of $q_s(z)$ may be smaller for tensile piles than for compression piles and this effect should to be considered.

Annex G (informative)
Sample procedures to determine limit values of earth pressure

Three earth pressure coefficients are defined, $K\gamma$ for the ground weight as defined by the unit weights γ, K_q for vertical surface loading q and K_c for ground cohesion c, all depending on the angle of shearing resistance of the ground.

At any point at distance z down the face of the wall (or vertical depth $z \cos\theta$) from the ground surface, the total pressure components are then σ (normal) and τ (tangential), with τ positive when the pressure from the ground on the wall is directed toward the top:

For drained states and non-saturated soils:

$$\sigma = \sigma' = K\gamma \cdot \int_o^z \gamma \mathrm{d}z + K_q \cdot q' + K_c \cdot c' \tag{G.1}$$

$$\tau = \sigma' \tan\delta + a \tag{G.2}$$

in which:

σ' is the effective stress normal to the wall at depth z;

δ is the angle of shearing resistance between ground and wall;

a' is the effective wall adhesion.

For drained states in saturated soils an approximate formulation is the following:

$$\sigma = \sigma' + u_z \tag{G.3}$$

$$\sigma' = K_\gamma \left(\int_o^z \gamma dz - \frac{u_z - u_o}{\cos\theta}\right) + K_q q' + K_c c' \tag{G.4}$$

$$\tau = \sigma' \tan\delta + a' \tag{G.5}$$

in which:

q' is the effective surcharge pressure;

u_z is the pore pressure on the failure surface at depth $z \cdot \cos\theta$ below the top of the wall;

u_o is the pore pressure at $z = 0$;

σ' is the effective stress normal to the wall at depth z;

δ is the angle of shearing resistance between ground and wall;

a' is the effective wall adhesion.

For undrained states:

$$\sigma = K\gamma_u \int_o^z \gamma \mathrm{d}z + K_{qu} q + K_{cu} c_u \tag{G.6}$$

$$\tau = a_u \tag{G.7}$$

in which:

$K\gamma_u$ = $K_{qu} = 1$, when the wall is vertical and the ground surface is horizontal

q = total surcharge pressure (including water pressure)

a_u = undrained wall adhesion

In layered soils, the coefficients K may normally be determined by the friction angle at depth z only, independent of the values at other depths.

In active state, tensile active stresses should never be considered as actions on retaining structures.

Explicit formulas for the earth pressure coefficients are not available for the general case. Two sample procedures to determine earth pressure coefficients are given in the following.

Diagrams for vertical walls

For vertical walls, the values may be taken from Figure G.1, Figure G.2, Figure G.3 and Figure G.4.

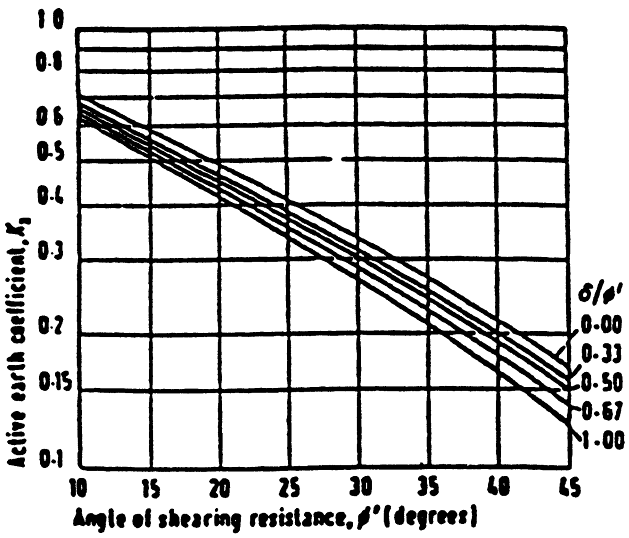

Figure G.1 — Coefficients of active earth pressure (horizontal component) for horizontal retained surface

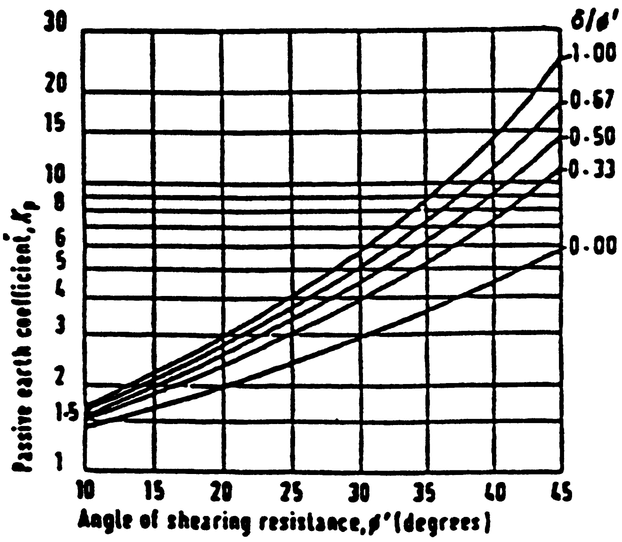

Figure G.2 — Coefficients of passive earth pressure (horizontal component) for horizontal retained surface

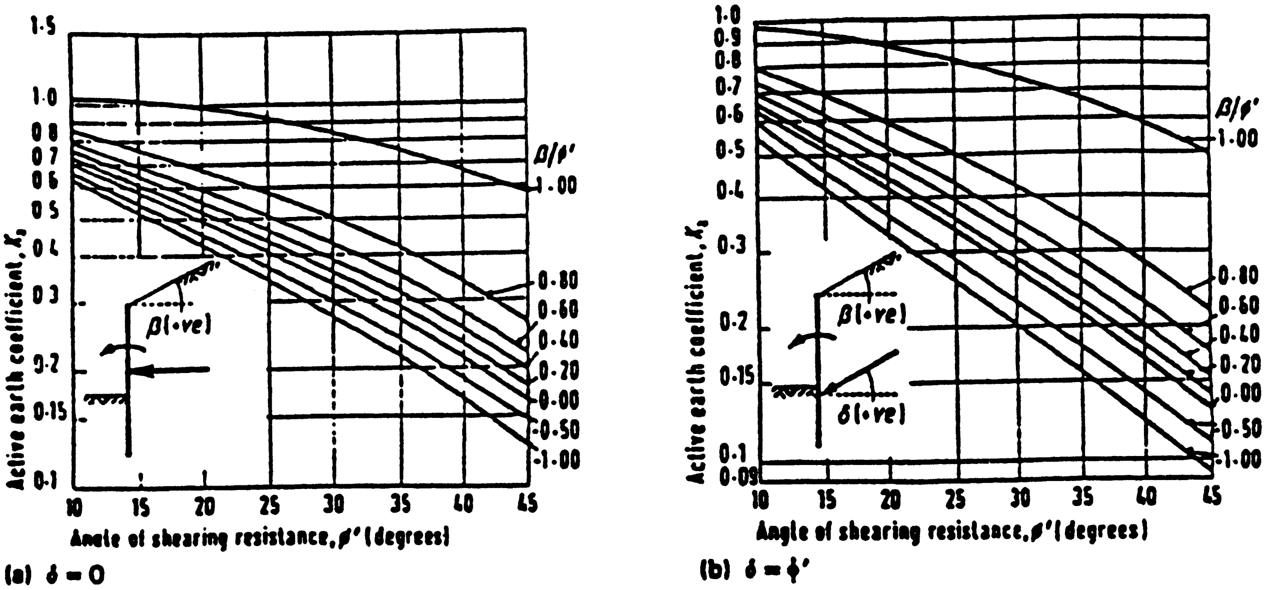

Figure G.3 — Coefficients of active earth pressure (horizontal component) for general case on inclined backfill with wall friction

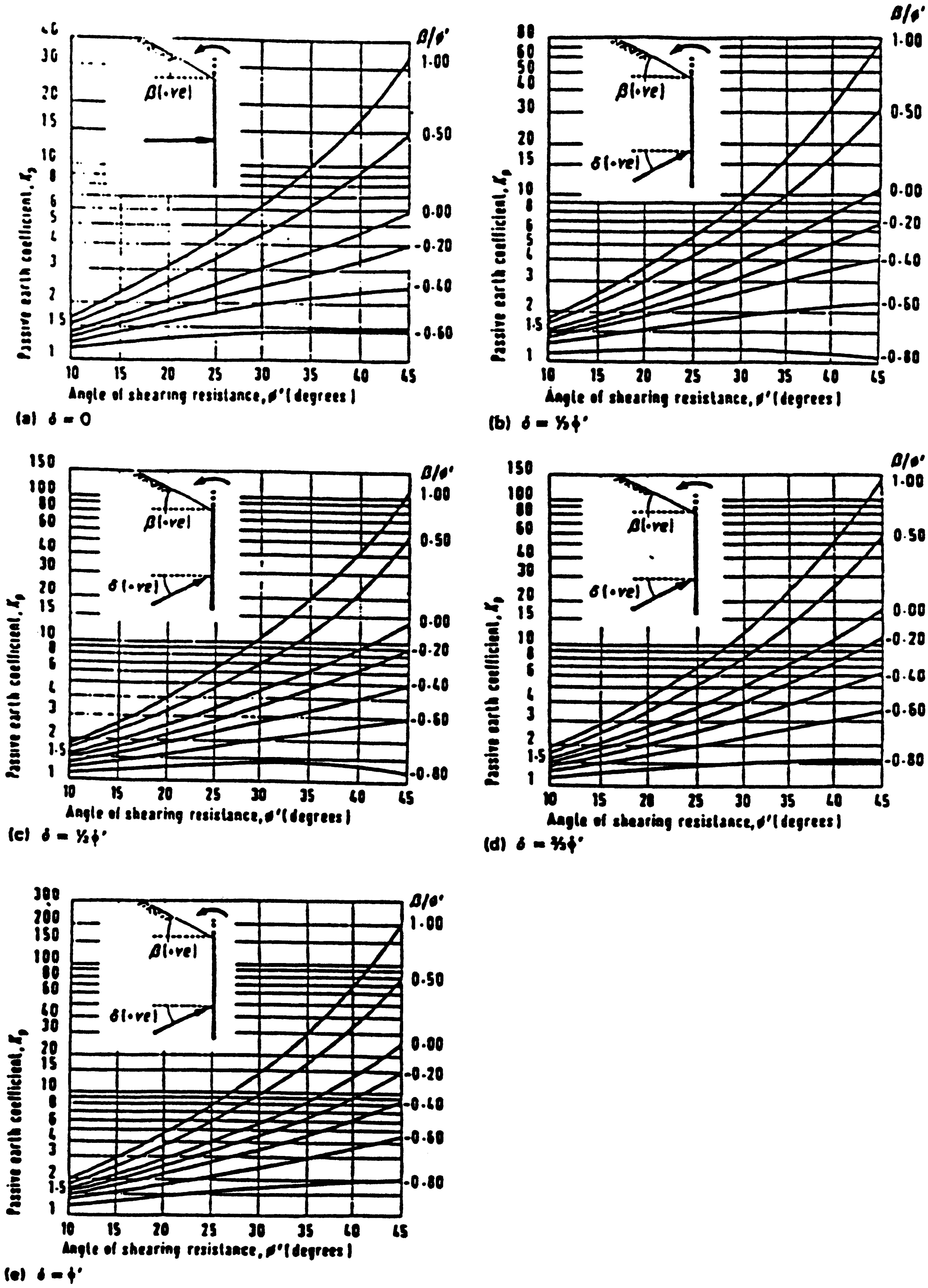

Figure G.4 — Coefficients of passive earth pressure (horizontal component) for general case of inclined backfill with wall friction

Numerical procedure

The following procedure, which includes certain approximations on the safe side, may be used in all cases.

The procedure is stated for passive pressures in which case the strength parameters (represented in the following by ϕ, c, δ, a) are inserted as positive values.

The following symbols are used in addition to those in **1.6** and **1.7**, cf Figure G.5:

- m_t is the angle from the soil surface direction, pointing away from the wall, to the tangent direction of the intersecting slip line that bounds the moving soil mass, pointing out from the soil surface;
- m_w is the angle from wall normal to the tangent direction at the wall of the exterior slip line, positive when the tangent points upwards behind the wall;
- β is the angle from the horizontal to the soil surface direction, positive when the soil surface rises away from the wall;
- θ is the angle between the vertical and the wall direction, positive when the wall is overhanging;
- v is the tangent rotation along the exterior slip line, positive when the soil mass above this slip line is of a convex shape;
- q is a general uniform surcharge pressure, per area unit of the actual surface;
- p is a vertical uniform surcharge pressure, per area unit in a horizontal projection.

The interface parameters δ and a must be chosen so that:

$a/c = \tan\delta/\tan\phi$

The boundary condition at the soil surface involves β_0 which is the angle of incidence of an equivalent surface load. With this concept the angle is defined from the vectorial sum of two terms, see Figure G.5:

— the actual distributed surface loading, q per unit of surface area, uniform but not necessarily vertical, and;

— $c \cot\phi$ acting as normal load.

The angle β_0 is positive when the tangential component of q points toward the wall while the normal component is directed toward the soil. If $c = 0$ while the surface load is vertical or zero, and for active pressures generally, $\beta_0 = \beta$.

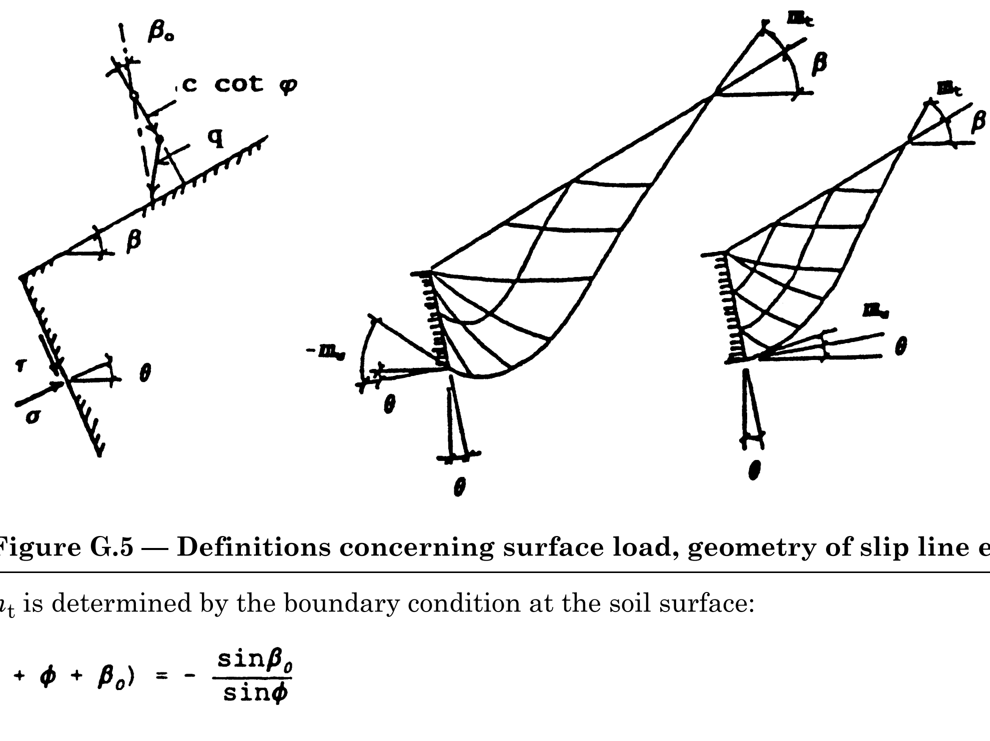

Figure G.5 — Definitions concerning surface load, geometry of slip line etc

The angle m_t is determined by the boundary condition at the soil surface:

$$\cos(2m_t + \phi + \beta_0) = -\frac{\sin\beta_0}{\sin\phi} \tag{G.8}$$

The boundary condition at the wall determines m_w by:

$$\cos(2m_w + \phi + \delta) = \frac{\sin\delta}{\sin\phi} \quad (G.9)$$

The angle m_w is negative for passive pressures ($\phi > 0$) if the ratio $\sin\delta/\sin\phi$ is sufficiently large, as assumed for Figure G.5.

The total tangent rotation along the exterior slip line of the moving soil mass, see Figure G.5, is determined by the angle v to be computed by the expression:

$$v = m_t + \beta - m_w - \theta \quad (G.10)$$

The coefficient K_n for normal loading on the surface (i.e. the normal earth pressure on the wall from a unit pressure normal to the surface) is then determined by the following expression in which v is to be inserted in radians:

$$K_n = \frac{1 + \sin\phi\,\sin(2m_w + \phi)}{1 - \sin\phi\,\sin(2m_t + \phi)}\exp(2v\tan\phi) \quad (G.11)$$

The coefficient for a vertical loading on the surface force per unit of horizontal area projection, is:

$$K_q = K_n \cos^2\beta \quad (G.12)$$

and the coefficient for the cohesion term is:

$$K_c = (K_n - 1)\cot\phi \quad (G.13)$$

For the soil weight an approximate expression is:

$$K\gamma = K_n \cos\beta \cos(\beta - \phi) \quad (G.14)$$

This expression is on the safe side. While the error is unimportant for active pressures it may be considerable for passive pressures with positive values of β.

For $\phi = 0$ the following limit values are found:

$\cos 2m_t = -\frac{p}{c}\sin\beta\cos\beta$; $\cos 2m_w = \frac{a}{c}$;

$K_q = \cos^2\beta$; $K_c = 2v + \sin 2m_t + \sin 2m_w$;

(with v in radians) while for $K\gamma$, a better approximation when $\phi = 0$ is:

$$K_\gamma = \cos\phi + \frac{\sin\beta\cos m_w}{\sin m_t}; \quad (G.15)$$

For active pressures the same algorithm is used, with the following changes:

The strength parameters ϕ, c, δ and a are inserted as negative values. The angle of incidence of the equivalent surface load $\beta_o = \beta$, mainly because of the approximations used for $K\gamma$.

Both for passive and active pressures, the procedure assumes the angle of convexity to be positive:

$v \geq 0$

If this condition is not (even approximately) fulfilled, e.g. for a smooth wall and a sufficiently sloping soil surface when β and ϕ have opposite signs, it may be necessary to consider using other methods. This may also be the case when irregular surface loads are considered.

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