BRITISH STANDARD

**BS EN
1997-2:2007**

*Incorporating
corrigendum
June 2010*

Eurocode 7 — Geotechnical design —

Part 2: Ground investigation and testing

ICS 91.010.30; 93.020

BSI

British Standards

National foreword

This British Standard is the UK implementation of EN 1997-2:2007, incorporating corrigendum June 2010. It supersedes DD ENV 1997-2:2000 and DD ENV 1997-3:2000 which are withdrawn. It partially supersedes BS 5930:1999 and BS 1377-9:1990.

The start and finish of text introduced or altered by corrigendum is indicated in the text by tags. Text altered by CEN corrigendum June 2010 is indicated in the text by ⟨AC₁⟩ ⟨AC₁⟩.

The structural Eurocodes are divided into packages by grouping Eurocodes for each of the main materials: concrete, steel, composite concrete and steel, timber, masonry and aluminium; this is to enable a common date of withdrawal (DOW) for all the relevant parts that are needed for a particular design. The conflicting national standards will be withdrawn at the end of the coexistence period, after all the EN Eurocodes of a package are available.

Following publication of the EN, there is a period allowed for national calibration during which the National Annex is issued, followed by a coexistence period of a maximum three years. During the coexistence period Member States are encouraged to adapt their national provisions. At the end of this coexistence period, the conflicting parts of national standard(s) will be withdrawn.

In the UK, the primary corresponding national standard is:

— BS 5930:1999, *Code of practice for site investigations*

and based on this transition period, this standard will be withdrawn/revised on a date to be announced, but at the latest by March 2010.

For in-situ testing, the primary test method references in BS EN 1997-2:2007 are to new CEN/ISO standards and this will require the withdrawal of some sections of BS 1377-9:1990, Methods of test for soils for civil engineering purposes — In-situ tests, as well as parts of BS 5930:1999 as they are published.

For laboratory testing, many of the primary test method references in BS EN 1997-2:2007 are new CEN/ISO technical specifications. These will not be adopted in the UK and the standards below will continue to be the preferred standards.

— BS 1377-1:1990, *Methods of test for soils for civil engineering purposes – General requirements and sample preparation*

— BS 1377-2:1990, *Methods of test for soils for civil engineering purposes – Classification tests*

Amendments/corrigenda issued since publication

Date	Comments
31 October 2010	Implementation of CEN corrigendum June 2010

— BS 1377-3:1990, *Methods of test for soils for civil engineering purposes – Chemical and electro-chemical tests*

— BS 1377-4:1990, *Methods of test for soils for civil engineering purposes – Compaction-related tests*

— BS 1377-5:1990, *Methods of test for soils for civil engineering purposes – Compressibility, permeability and durability tests*

— BS 1377-6:1990, *Methods of test for soils for civil engineering purposes – Consolidation and permeability tests in hydraulic cells and with pore pressure measurement*

— BS 1377-7:1990, *Methods of test for soils for civil engineering purposes – Shear strength tests (total stress)*

— BS 1377-8:1990, *Methods of test for soils for civil engineering purposes – Shear strength tests (effective stress)*

Further guidance will be given in the National Annex to EN 1997-2.

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

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

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

To enable EN 1997-2 to be used in the UK, the NDPs will be published in a National Annex, which will be made available by BSI in due course, after public consultation has taken place.

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

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

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EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

EN 1997-2

March 2007

ICS 91.060.01; 91.120.20

Supersedes ENV 1997-2:1999, ENV 1997-3:1999
Incorporating corrigendum June 2010

English Version

Eurocode 7 - Geotechnical design - Part 2: Ground investigation and testing

Eurocode 7 - Calcul géotechnique - Partie 2:
Reconnaissance des terrains et essais

Eurocode 7 - Entwurf, Berechnung und Bemessung in der
Geotechnik - Teil 2: Erkundung und Untersuchung des
Baugrunds

This European Standard was approved by CEN on 12 June 2006.

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

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

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

EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

Management Centre: rue de Stassart, 36   B-1050 Brussels

Ref. No. EN 1997-2:2007: E

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Foreword

This document (EN 1997-2: 2007) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held by BSI.

This European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by September 2007, and conflicting national standards shall be withdrawn at the latest by March 2010.

This document supersedes ENV 1997-2:1999 and ENV 1997-3:1999.

CEN/TC 250 is responsible for all Structural Eurocodes.

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

Background of the Eurocode programme

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

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

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

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement[1] between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (e.g. the Council Directive 89/106/EEC on construction products - CPD - and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

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

EN 1990	Eurocode :	Basis of Structural Design
EN 1991	Eurocode 1:	Actions on structures

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

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

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

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes:
— as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire;
— as a basis for specifying contracts for construction works and related engineering services;
— as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs).

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents[2] referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards[3]. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

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

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

3 According to Art. 12 of the CPD the interpretative documents shall :
a)   give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;
b)   indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc. ;
c)   serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.
The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

National Standards implementing Eurocodes

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

The National annex may only contain information on those parameters, which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e.:
 – values and/or classes where alternatives are given in the Eurocode;
 – values to be used where a symbol only is given in the Eurocode;
 – country specific data (geographical, climatic), *e.g.* snow map;
 – the procedure to be used where alternative procedures are given in the Eurocode.
It may also contain:
 – decisions on the application of informative annexes;
 – references to non-contradictory complementary information to assist the user to apply the Eurocode.

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

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works[4]. Furthermore, all the information accompanying the CE Marking of the construction products, which refer to Eurocodes, should clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to Eurocode 7

EN 1997-2 gives guidance for the planning and interpretation of geotechnical laboratory and field tests that are used for the support of geotechnical design of buildings and civil engineering works.

EN 1997-2 is intended for clients, designers, geotechnical laboratories, field testing laboratories and public authorities.

EN 1997-2 is intended to be used with EN 1997-1.

When using EN 1997-2, particular regard should be paid to the underlying assumptions and conditions given in 1.3.

The six sections of EN 1997-2 are complemented by 24 informative annexes.

National annex for EN 1997-2

The national standard implementing EN 1997-2 should have a national annex containing all information concerning the application of EN 1997-2 in the relevant country.

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

Section 1   General

1.1 Scope

1.1.1 Scope of Eurocode 7

(1) EN 1997 is intended to be used in conjunction with EN 1990:2002, which establishes the principles and requirements for safety and serviceability, describes the basis of design and verification and gives guidelines for related aspects of structural reliability.

(2) EN 1997 is intended to be applied to the geotechnical aspects of the design of buildings and civil engineering works. It is subdivided into various separate parts (see 1.1.2).

(3) EN 1997 is concerned with the requirements for strength, stability, serviceability and durability of structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

(4) Numerical values of actions on buildings and civil engineering works to be taken into account in design are provided in EN 1991 for the various types of construction. Actions imposed by the ground, such as earth pressures, shall be calculated according to the rules of EN 1997.

(5) Separate European Standards are intended to be used to treat matters of execution and workmanship. They are denoted in the relevant sections.

(6) In EN 1997 execution is covered to the extent that is necessary to conform to the assumptions of the design rules.

(7) EN 1997 does not cover the special requirements of seismic design. EN 1998 provides additional rules for geotechnical seismic design, which complete or adapt the rules of this standard.

1.1.2 Scope of EN 1997-2

(1) EN 1997-2 is intended to be used in conjunction with EN 1997-1 and provides rules supplementary to EN 1997-1 related to:
− planning and reporting of ground investigations;
− general requirements for a number of commonly used laboratory and field tests;
− interpretation and evaluation of test results;
− derivation of values of geotechnical parameters and coefficients.

In addition, examples of the application of field test results to design are given.

NOTE   Establishment of characteristic values is covered in EN 1997-1.

(2) This document gives no specific provisions for environmental ground investigations.

(3) Only commonly used geotechnical laboratory and field tests are covered in this standard. These were selected on the basis of their importance in geotechnical practice, availability in

commercial geotechnical laboratories and existence of an accepted testing procedure in Europe. The laboratory tests on soils are mainly applicable to saturated soils.

NOTE   It is expected that updates of the present standard will gradually include laboratory and field tests covering additional aspects of soil and rock behaviour.

(4) The provisions of this standard apply primarily to projects of geotechnical category 2, as defined in 2.1 of EN 1997-1:2004. The ground investigation requirements for category 1 projects are normally limited as the verifications often will be based on local experience. For geotechnical category 3 projects, the amount of investigations required will normally be at least the same as indicated for geotechnical category 2 projects in the following sections. Additional investigations and more advanced tests, related to the circumstances that place a project in geotechnical category 3, may be necessary.

(5) The derivation of parameter values is dedicated primarily to the design of pile and spread foundations based on field testing, as detailed in Annexes D, E, F and G of EN 1997-1:2004.

1.2 Normative references

(1) The following normative documents contain provisions which, through reference in this text, constitute provisions of this European Standard. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. However, parties to agreements based on this European Standard are encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references, the last edition of the normative document referred to applies.

EN 1990:2002	*Eurocode — Basis of structural design*
EN 1997-1:2004	*Eurocode 7 — Geotechnical design — Part 1: General rules*
EN ISO 14688-1	*Geotechnical investigation and testing — Identification and classification of soil — Part 1: Identification and description*
EN ISO 14688-2	*Geotechnical investigation and testing — Identification and classification of soil — Part 2: Classification principles*
EN ISO 14689-1	*Geotechnical investigation and testing — Identification and classification of rock - Part 1: Identification and description*
EN ISO 22475-1[5]	*Geotechnical investigation and testing — Sampling by drilling and excavation and groundwater measurements — Part 1: Technical principles of execution*
ⒶⒸ₁⟩ EN ISO 22476-1[5]	*Geotechnical investigation and testing — Field testing — Part 1: Electrical cone and piezocone penetration tests* ⟨ⒶⒸ₁
EN ISO 22476-2	*Geotechnical investigation and testing — Field testing — Part 2: Dynamic probing*
EN ISO 22476-3	*Geotechnical investigation and testing — Field testing — Part 3: Standard penetration test*
EN ISO 22476-4[5]	*Geotechnical investigation and testing — Field testing — Part 4: Ménard pressuremeter test*
EN ISO 22476-5[5]	*Geotechnical investigation and testing — Field testing — Part 5: Flexible dilatometer test*
EN ISO 22476-6[6]	*Geotechnical investigation and testing — Field testing — Part 6: Self boring pressuremeter test*
ⒶⒸ₁⟩ EN ISO 22476-7[6]	*Geotechnical investigation and testing – Field testing – Part 7: Borehole jack test* ⟨ⒶⒸ₁

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[5] to be published
ⒶⒸ₁⟩ [6] to be published ⟨ⒶⒸ₁

EN ISO 22476-8[6]   *Geotechnical investigation and testing — Field testing — Part 8: Full displacement pressuremeter test*

EN ISO 22476-9[6]   *Geotechnical investigation and testing — Field testing — Part 9: Field vane test*

[AC1] EN ISO 22476-12   *Geotechnical investigation and testing — Field testing — Part 12: Mechanical CPT* [AC1]

EN ISO 22476-13[6]   *Geotechnical investigation and testing — Field testing — Part 13: Plate loading test*

NOTE   The Bibliography presents a number of CEN ISO Technical Specifications (CEN ISO/TS), giving information on procedures, equipment, evaluation and presentation for some field and laboratory tests. These technical specifications may become European/ISO standards in due time. The National Standards Body may decide to keep its national standard in force during the lifetime of a CEN ISO/TS. National Annexes to EN 1997-2 may give information regarding national practise involved.

1.3 Assumptions

(1) Reference is made to - EN 1990:2002, 1.3 and EN 1997-1:2004, 1.3.

(2) The provisions of this standard are based on the assumptions given below:
− data required for design are collected, recorded and interpreted by appropriately qualified personnel;
− structures are designed by appropriately qualified and experienced personnel;
− adequate continuity and communication exist between the personnel involved in data-collection, design and construction;

1.4 Distinction between Principles and Application Rules

(1) Depending on the character of the individual clauses, distinction is made in EN 1997-2 between Principles and Application Rules.

(2) The Principles comprise:
− general statements and definitions for which there is no alternative;
− requirements and analytical models for which no alternative is permitted unless specifically stated.

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

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

(5) It is permissible to use alternatives to the Application Rules given in this standard, provided it is shown that the alternative rules accord with the relevant Principles and are at least equivalent with regard to the structural safety, serviceability and durability, which would be expected when using the Eurocodes.

NOTE   If an alternative design rule is submitted for an application rule, the resulting design cannot be claimed to be wholly in accordance with EN 1997-2, although the design will remain in accordance with the Principles of EN 1997-1. When EN 1997-2 is used in respect of a property listed in an Annex Z of a product standard or an ETAG, the use of an alternative design rule may not be acceptable for CE marking.

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(6) In EN 1997-2, the Application Rules are identified by a number in brackets e.g. as in this clause.

1.5 Definitions

1.5.1 Terms common to all Eurocodes

(1)P The terms used in common for all Eurocodes are defined in EN 1990.

1.5.2 Terms common to Eurocode 7

(1)P The terms specific to EN 1997 are defined in 1.5.2 of EN 1997-1:2004.

1.5.3 Specific definitions used in EN 1997-2

1.5.3.1
derived value
value of a geotechnical parameters obtained from test results by theory, correlation or empiricism (see 1.6)

1.5.3.2
disturbed sample
sample where the soil structure, water content and/or constituents have been changed during sampling

1.5.3.3
measured value
value that is measured in a test

1.5.3.4
natural specimen
specimen made from the available (disturbed, undisturbed, remoulded) sample

1.5.3.5
quality class
classification by which the quality of a soil sample is assessed in the laboratory

NOTE   For laboratory testing purposes, soil samples are classified in five quality classes (see 3.4.1).

1.5.3.6
remoulded sample
sample of which the soil or rock structure is fully disturbed

1.5.3.7
remoulded specimen
fully disturbed specimen, at natural water content

1.5.3.8
re-compacted specimen
specimen forced into a mould with a rammer or under desired static stress state

1.5.3.9
reconstituted specimen
specimen prepared in the laboratory; for fine soils, it is prepared as a slurry (at or above the liquid limit) and then consolidated (sedimented); for coarse soils, it is either poured or pluviated in dry (dried) or wet conditions and compacted, or consolidated

1.5.3.10
re-consolidated specimen
specimen compressed in a mould or cell under static pressure while allowing drainage to take place

1.5.3.11
sample
portion of soil or rock recovered from the ground by sampling techniques

1.5.3.12
specimen
part of a soil or rock sample used for a laboratory test

1.5.3.13
strength index test
test of a nature that yields an indication of the shear strength, without necessarily giving a representative value

NOTE   The results of such a test are subject to uncertainty.

1.5.3.14
swelling
expansion due to reduction of effective stress resulting from either reduction of total stress or absorption of (in general) water at constant total stress

NOTE   Swelling includes the reverse of both compression and consolidation.

1.5.3.15
undisturbed sample
sample where no change in the soil characteristics of practical significance has occurred

1.6 Test results and derived values

(1) Test results and derived values form the basis for the selection of characteristic values of ground properties to be used for the design of geotechnical structures, in accordance with 2.4.3 of EN 1997-1:2004.

NOTE 1   The process of geotechnical design consists of a few successive phases (see Figure 1.1), the first of which covers the site investigation and testing, whereas the next one is devoted to the determination of characteristic values, and the last phase covers the design verification calculations. Rules for the first phase are given in the

present standard. The determination of characteristic values and the design of the structures are covered by EN 1997-1.

Type of test
F= field L= laboratory

Correlations

Test results and derived values

EN 1997-2

EN 1997 1
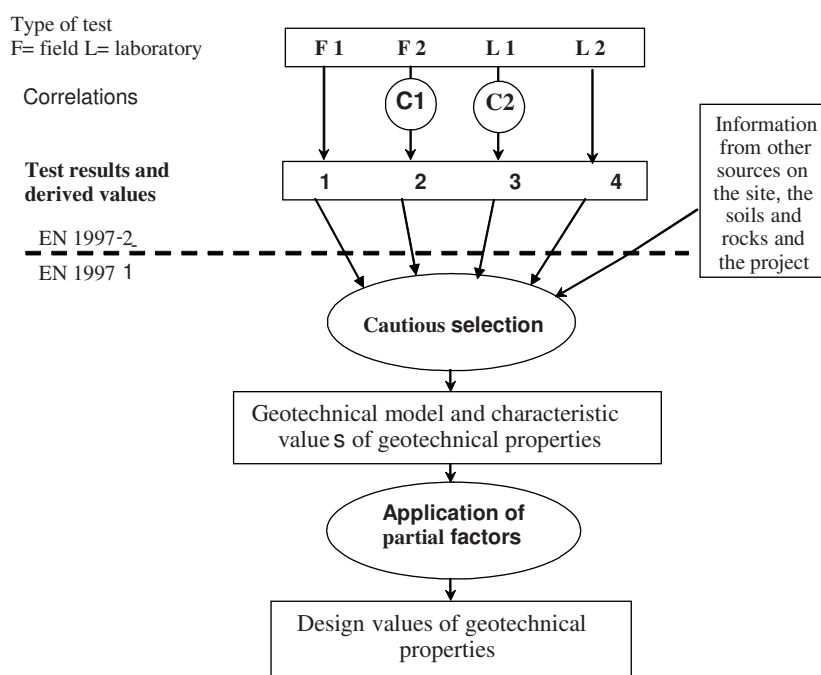
Figure 1.1 — General framework for the selection of derived values of geotechnical properties

[AC1] (2) Test results can be experimental curves or values of geotechnical parameters. In Annex A, a list of test results is given to serve as a reference to test standards[7]. [AC1]

(3) Derived values of geotechnical parameters and/or coefficients, are obtained from test results by theory, correlation or empiricism.

NOTE 2   The examples of correlations used to determine derived values given in the annexes to Section 4 of this standard are obtained from the literature. These correlations may correlate the value of a geotechnical parameter or coefficient with a test result, such as the q_c-value of a CPT. They may also connect a geotechnical parameter to a test result by means of theoretical considerations (for example, when deriving a value of the angle of shearing resistance φ' from pressuremeter test results or from the index of plasticity).

NOTE 3   In certain cases, the derivation of geotechnical parameters by means of a correlation is not made before the determination of the characteristic value, but after the test results have been corrected or transformed into conservative estimates.

1.7 The link between EN 1997-1 and EN 1997-2

(1) Figure 1.2 presents the general architecture of the CEN standards related to geotechnical engineering problems and those directly linked to EN 1997. The design part is covered by EN 1997-1. The present standard gives rules for ground investigations and obtaining geotechnical parameters or coefficients values to be used for determining the characteristic values (as specified in EN 1997-1). It gives also informative examples of calculation methods for spread and deep foundations. The implementation of EN 1997 needs information based on other standards, in particular those related to ground investigations and to the execution of geotechnical works.

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7 Geotechnical test standards, yielding these results are prepared by CEN/TC341.

EN 1997-1

Design rules
- General framework for geotechnical design
- Definition of ground parameters
- Characteristic and design values
- General rules for site investigation
- Rules for the design of main types of geotechnical structures
- Some assumptions on execution procedures

EN 1997-2

Geotechnical investigation and testing
- Detailed rules for site investigations
- General test specifications
- Derivation of ground properties and geotechnical model of the site
- Examples of calculation methods based on field and laboratory tests

Test standards (CEN/TC 341)

Standards for
- Drilling and sampling methods and groundwater measurements
- Laboratory and field tests on soils and rocks
- Tests on structures or parts of structures
- Identification and classification of soils and rocks

Execution of geotechnical works (CEN/TC 288)

Execution standards
- specific design rules (informative annexes)
- specific test procedures

Figure 1.2 — General architecture of the CEN standards linked with EN 1997

1.8 Symbols and units

(1) For the purpose of EN 1997-2 the following symbols apply.

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

Latin letters

C_c compression index

c' cohesion intercept in terms of effective stress

c_{fv} undrained shear strength from the field vane test

c_u undrained shear strength

C_s swelling index

c_v coefficient of consolidation

C_α coefficient of secondary compression

D_n particle size such that n % of the particles by weight are smaller than that size e.g. D_{10}, D_{15}, D_{30}, D_{60} and D_{85}

E	Young's modulus of elasticity
E'	drained (long term) Young's modulus of elasticity
E_{FDT}	flexible dilatometer modulus
E_M	Ménard pressuremeter modulus
E_{meas}	measured energy during calibration
E_{oed}	oedometer modulus
E_{PLT}	modulus from plate loading test
E_r	energy ratio (= E_{meas} / E_{theor})
E_{theor}	theoretical energy
E_u	undrained Young's modulus of elasticity
E_0	initial Young's modulus of elasticity
E_{50}	Young's modulus of elasticity corresponding to 50 % of the maximum shear strength
I_A	activity index
I_C	consistency index
I_D	density index
I_{DMT}	material index from the flat dilatometer test
K_{DMT}	horizontal stress index from the flat dilatometer test
$\boxed{\text{AC}_1}$ k	coefficient of permeability $\boxed{\text{AC}_1}$
I_L	liquidity index
I_P	plasticity index
k_s	coefficient of sub-grade reaction
m_v	coefficient of compressibility
N	number of blows per 300 mm penetration from the SPT
N_k	cone factor for CPT, (see equation (4.1))
N_{kt}	cone factor for CPTU, (see equation (4.2))
N_{10L}	number of blows per 10 cm penetration from the DPL
N_{10M}	number of blows per 10 cm penetration from the DPM
N_{10H}	number of blows per 10 cm penetration from the DPH
N_{10SA}	number of blows per 10 cm penetration from the DPSH-A
N_{10SB}	number of blows per 10 cm penetration from the DPSH-B
N_{20SA}	number of blows per 20 cm penetration from the DPSH-A
N_{20SB}	number of blows per 20 cm penetration from the DPSH-B
N_{60}	number of blows from the SPT corrected to energy losses
$(N_1)_{60}$	number of blows from the SPT corrected to energy losses and normalized for effective vertical overburden stress

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p_{LM}	Ménard limit pressure
q_c	cone penetration resistance
q_t	cone penetration resistance corrected for pore water pressure effects
q_u	unconfined compressive strength
w_{opt}	optimum water content

Greek letters

AC₁⟩ α	correlation factor for E_{oed} and q_c, (see Equation (4.3)) ⟨AC₁
φ	angle of shearing resistance
φ'	angle of shearing resistance in terms of effective stress
μ	correction factor to derive c_u from c_{fv}, (see Equation (4.4))
$\rho_{d;max}$	maximum dry density
σ_C	unconfined compression strength of rock
AC₁⟩ σ'_p	effective pre-consolidation pressure or effective vertical yield stress in situ ⟨AC₁
σ_T	tensile strength of rock
AC₁⟩ σ_{v0}	initial vertical total stress ⟨AC₁
AC₁⟩ σ'_{v0}	initial vertical effective stress ⟨AC₁
ν	Poisson's ratio

Abbreviations

AC₁⟩ CPT	electrical cone penetration test ⟨AC₁
AC₁⟩ CPTM	mechanical cone penetration test ⟨AC₁
CPTU	cone penetration test with pore water pressure measurement
DMT	flat dilatometer test
DP	dynamic probing
DPL	dynamic probing light
DPM	dynamic probing medium
DPH	dynamic probing heavy
DPSH-A	dynamic probing superheavy, type A
DPSH-B	dynamic probing superheavy, type B
FDP	full displacement pressuremeter
FDT	flexible dilatometer test
FVT	field vane test
MPM	Ménard pressuremeter

PBP   pre-bored pressuremeter

PLT   plate loading test

PMT   pressuremeter test

RDT   rock dilatometer test

SBP   self-boring pressuremeter

SDT   soil dilatometer test

SPT   standard penetration test

WST   weight sounding test

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

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

Section 2   Planning of ground investigations

2.1 Objectives

2.1.1 General

(1)P Geotechnical investigations shall be planned in such a way as to ensure that relevant geotechnical information and data are available at the various stages of the project. Geotechnical information shall be adequate to manage identified and anticipated project risks. For intermediate and final building stages, information and data shall be provided to cover risks of accidents, delays and damage.

(2) The aims of a geotechnical investigation are to establish the soil, rock and groundwater conditions, to determine the properties of the soil and rock, and to gather additional relevant knowledge about the site.

(3)P Careful collection, recording and interpretation of geotechnical information shall be made. This information shall include ground conditions, geology, geomorphology, seismicity and hydrology, as relevant. Indications of the variability of the ground shall be taken into account.

(4) Ground conditions which may influence the choice of geotechnical category should be determined as early as possible in the investigation.

NOTE   As a result of the geotechnical investigations, it may be necessary to change the geotechnical category of the project (see 1.1.2 (4)).

(5) Geotechnical investigations should consist of ground investigations, and other investigations for the site, such as:
− the appraisal of existing constructions, e.g. buildings, bridges, tunnels, embankments and slopes;
− the history of development on and around the site.

(6) Before designing the investigation programme, the available information and documents should be evaluated in a desk study.

(7) Examples of information and documents that can be used are:
− topographical maps;
− old city maps describing the previous use of the site;
− geological maps and descriptions;
− engineering geological maps;
− hydrogeological maps and descriptions;
− geotechnical maps;
− aerial photos and previous photo interpretations;
− aero-geophysical investigations;
− previous investigations at the site and in the surroundings;
− previous experiences from the area;
− local climatic conditions.

(8) Ground investigations should consist of field investigations, laboratory testing, additional desk studies and controlling and monitoring, where appropriate.

(9)P Before the investigation programme has been drawn up the site shall be visually examined and the findings recorded and cross-checked against the information gathered by desk studies.

(10) The ground investigation programme should be reviewed as the results become available so that the initial assumptions can be checked. In particular:
− the number of investigation points should be extended if it is deemed necessary to obtain an accurate insight into the complexity and the variability of the ground at the site;
− the parameters obtained should be checked to see that they fit into a consistent behavioural pattern for soil or rock. If necessary additional testing should be specified;
− any limitations in the data, revealed according to EN 1997-1:2004, 3.4.3 (1) should be considered.

(11) Special attention should be paid to sites that have been previously used, where disturbance of the natural ground conditions may have taken place.

(12)P An appropriate quality assurance system shall be in place in the laboratory, in the field and in the engineering office, and quality control shall be exercised competently in all phases of the investigations and their evaluation.

2.1.2 Ground

(1)P Ground investigations shall provide a description of ground conditions relevant to the proposed works and establish a basis for the assessment of the geotechnical parameters relevant for all construction stages.

(2) The information obtained should enable assessment of the following aspects, if possible:
− the suitability of the site with respect to the proposed construction and the level of acceptable risks;
− the deformation of the ground caused by the structure or resulting from construction works, its spatial distribution and behaviour over time;
− the safety with respect to limit states (e.g. subsidence, ground heave, uplift, slippage of soil and rock masses, buckling of piles, etc.);
− the loads transmitted to the structure from the ground (e.g. lateral pressures on piles) and the extent to which they depend on its design and construction;
− the foundation methods (e.g. ground improvement, whether it's possible to excavate, driveability of piles, drainage);
− the sequence of foundation works;
− the effects of the structure and its use on the surroundings;
− any additional structural measures required (e.g. support of excavation, anchorage, sleeving of bored piles, removal of obstructions);
− the effects of construction work on the surroundings;
− the type and extent of ground contamination on, and in the vicinity of, the site;
− the effectiveness of measures taken to contain or remedy contamination.

2.1.3 Construction materials

(1)P Geotechnical investigations of soil and rock for use as construction materials shall provide a description of the materials to be used and shall establish their relevant parameters.

(2) The information obtained should enable an assessment of the following aspects:
– the suitability for the intended use;
– the extent of deposits;
– whether it is possible to extract and process the materials, and whether and how unsuitable material can be separated and disposed of;
– the prospective methods to improve soil and rock;
– the workability of soil and rock during construction and possible changes in their properties during transport, placement and further treatment;
– the effects of construction traffic and heavy loads on the ground;
– the prospective methods of dewatering and/or excavation, effects of precipitation, resistance to weathering, and susceptibility to shrinkage, swelling and disintegration.

2.1.4 Groundwater

(1)P Groundwater investigations shall provide all relevant information on groundwater needed for geotechnical design and construction.

(2) Groundwater investigations should provide, when appropriate, information on:
– the depth, thickness, extent and permeability of water-bearing strata in the ground, and joint systems in the rock;
– the elevation of the groundwater surface or piezometric surface of aquifers and their variation over time and actual groundwater levels including possible extreme levels and their periods of recurrence;
– the pore water pressure distribution;
– the chemical composition and temperature of groundwater.

(3) The information obtained should be sufficient to assess the following aspects, where relevant:
– the scope for and nature of groundwater-lowering work;
– possible harmful effects of the groundwater on excavations or on slopes (e.g. risk of hydraulic failure, excessive seepage pressure or erosion);
– any measures necessary to protect the structure (e.g. waterproofing, drainage and measures against aggressive water);
– the effects of groundwater lowering, desiccation, impounding etc. on the surroundings;
– the capacity of the ground to absorb water injected during construction work;
– whether it is possible to use local groundwater, given its chemical constitution, for construction purposes.

2.2 Sequence of ground investigations

(1)P The composition and the extent of the ground investigations shall be based on the anticipated type and design of the construction, e.g. type of foundation, improvement method or retaining structure, location and depth of the construction;

(2)P The results of the desk studies and the site inspection shall be considered when selecting the investigation methods and locating the various investigation points. Investigations shall be targeted at points representing the variation in ground conditions for soil, rock and groundwater.

(3) Ground investigations should normally be performed in phases depending on the questions raised during planning, design and construction of the actual project. The following phases are treated separately in Section 2:
− preliminary investigations for positioning and preliminary design of the structure (see 2.3);
− design investigations (see 2.4);
− controlling and monitoring (see 2.5).

NOTE   The provisions in this document are based on the premise that the results from investigations recommended in one phase are available before the next phase is started.

(4) In cases where all investigations are performed at the same time, 2.3 and 2.4 should be considered simultaneously.

NOTE   The different stages of ground investigations, including laboratory and field work and the process of evaluating soil and rock parameters, can follow the schemes in B.1 and B.2.

2.3 Preliminary investigations

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

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

2.4 Design investigations

2.4.1 Field investigations

2.4.1.1 General

(1)P In cases where the preliminary investigations do not provide the necessary information to assess the aspects mentioned in 2.3, complementary investigations shall be performed during the design investigation phase.

(2) If relevant, field investigations in the design phase should comprise:
− drilling and/or excavations (test pits including shafts and headings) for sampling;
− groundwater measurements;
− field tests.

(3) Examples of the various types of field investigations are:
− field testing (e.g. CPT, SPT, dynamic probing, WST, pressuremeter tests, dilatometer tests, plate load tests, field vane tests and permeability tests);
− soil and rock sampling for description of the soil or rock and laboratory tests;
− groundwater measurements to determine the groundwater table or the pore pressure profile and their fluctuations;
− geophysical investigations (e.g. seismic profiling, ground penetrating radar, resistivity measurements and down hole logging);
− large scale tests, for example to determine the bearing capacity or the behaviour directly on prototype elements, such as anchors.

(4) To develop strategies for planning field investigations, Table 2.1 can be used as a guide to the applicability of the field investigations covered in Sections 3 and 4.

NOTE   See also B.2.

(5)P Where ground contamination or soil gas is expected, information shall be gathered from the relevant sources. This information shall be taken into account when planning the ground investigation.

Table 2.1 — Simplified overview of the applicability of field investigation methods[a] covered by Sections 3 and 4

Field investigation methods [a]	Possibly obtainable results																		
	Sampling						Field tests											Groundwater measurements	
	Soil			Rock			CPT & CPTU	Pressuremeter [c]	RDT	Flexible Dilatometer	SPT [d]	DPL/DPM	DPH/DPSH	WST	FVT	DMT	PLT	Open system	Closed system
	Category A	Category B	Category C	Category A	Category B	Category C													
Basic information																			
Type of soil	C1 F1	C1 F1	C2 F2	—	—	—	C2 F2	C3 F3	—	C3 F3	C2 F1	C3 F3	C3 F3	—	—	C2 F2	—	—	—
Type of rock	—	—	—	R1	R1	R2	R3 [e]	R3	R2		—	—	—	—	—	—	—	—	—
Extension of layers [b]	C1 F1	C1F1	C3 F3	R1	R1	R2	C1 F1	R3 C3 F3	R3	C3 F3	C2 F2	C1 F2	C1 F2	F2	—	C2 F1	—	—	—
Groundwater level	—	—	—	—	—	—	C2	—	—	—	—	—	—	—	—	—	—	R2 C1 F2	R1 C1 F1
Pore water pressure	—	—	—	—	—	—	C2 F2	F3	—	—	—	—	—	—	—	—	—	R2 C1 F2	R1 C1 F1
Geotechnical properties																			
Particle size	C1 F1	C1 F1		R1	R1	R2	—	—	—	—	C2 F1	—	—	—	—	—	—	—	—
Water content	C1 F1	C2 F1	C3 F3	R1	R1	—	—	—	—	—	C2 F2	—	—	—	—	—	—	—	—
Atterberg limits	F1	F1	—	—	—	—	—	—	—	—	F2	—	—	—	—	—	—	—	—
Density	C2 F1	C3 F3	—	R1	R1	—	C2 F2	—	—	—	C2F2	C2	C2	—	—	C2 F2	—	—	—
Shear strength	C2 F1	—	—	R1	—	—	C2 F1	C1 F1	—	—	C2 F3	C2 F3	C2 F3	C2	F1	C2 F1	R2 C1 F1	—	—
Compressibility	C2 F1	—	—	R1	—	—	C1 F2	C1 F1	R1	F1	C2 F2	C2 F2	C2 F2	C2	—	C2 F1	C1F1	—	—
Permeability	C2 F1	—	—	R1	—	—	C3 F2	F3	—	—	—	—	—	—	—	—	—	C2 F3	C2 F2
Chemical tests	C1 F1	C1 F1	—	R1	R1	—	—	—	—	—	C2 F2	—	—	—	—	—	—	—	—

[a] see sections 3 and 4 for nomenclature
[b] in horizontal and vertical direction
[c] will depend on pressuremeter type
[d] assuming sample is retained
[e] soft rock only

R1	High for rock	R2	Medium for rock	R3	Low for rock
C1	High for coarse soil [*]	C2	Medium for coarse soil	C3	Low for coarse soil
F1	High for fine soil [*]	F2	Medium for fine soil	F3	Low for fine soil
—	not applicable				

[*] main soil groups "coarse" and "fine" according to ISO 14688-1

Applicability:

NOTE   Depending on the ground conditions (such as soil type, groundwater conditions) and the planned design, the selection of investigation methods will vary and may deviate from this table.

(6)P If ground contamination or soil gas is detected in the course of ground investigations, this shall be reported to the client and the responsible authorities.

2.4.1.2 Field investigation programme

(1)P The field investigation programme shall contain:
− a plan with the locations of the investigation points including the types of investigation;
− the depth of the investigations;
− the types of sample (category, etc.) to be taken including specifications for the number and depth at which they are to be taken;
− specifications on the groundwater measurement;
− the types of equipment to be used;
− the standards to be applied.

2.4.1.3 Locations and depths of the investigation points

(1)P The locations of investigation points and the depths of the investigations shall be selected on the basis of the preliminary investigations as a function of the geological conditions, the dimensions of the structure and the engineering problems involved.

(2) When selecting the locations of investigation points, the following should be observed:
− the investigation points should be arranged in such a pattern that the stratification can be assessed across the site;
− the investigation points for a building or structure should be placed at critical points relative to the shape, structural behaviour and expected load distribution (e.g. at the corners of the foundation area);
− for linear structures, investigation points should be arranged at adequate offsets to the centre line, depending on the overall width of the structure, such as an embankment footprint or a cutting;
− for structures on or near slopes and steps in the terrain (including excavations), investigation points should also be arranged outside the project area, these being located so that the stability of the slope or cut can be assessed. Where anchorages are installed, due consideration should be given to the likely stresses in their load transfer zone;
− the investigation points should be arranged so that they do not present a hazard to the structure, the construction work, or the surroundings (e.g. as a result of the changes they may cause to the ground and groundwater conditions);
− the area considered in the design investigations should extend into the neighbouring area to a distance where no harmful influence on the neighbouring area is expected.
− for groundwater measuring points, the possibility of using the equipment installed during the ground investigation for continued monitoring during and after the construction period should be considered.

(3) Where ground conditions are relatively uniform or the ground is known to have sufficient strength and stiffness properties, wider spacing or fewer investigation points may be applied. In either case, this choice should be justified by local experience.

(4)P In cases where more than one type of investigation is planned at a certain location (e.g. CPT and piston sampling), the investigation points shall be separated by an appropriate distance.

(5) In the case of a combination of, for example, CPTs and boreholes, the CPTs should be carried out prior to the boreholes. The minimum spacing should then be such that the borehole does not or is considered unlikely to encounter the CPT hole. If the drilling is conducted first, the CPT should be carried out at a horizontal separation of at least 2 m.

(6)P The depth of investigations shall be extended to all strata that will affect the project or are affected by the construction. For dams, weirs and excavations below groundwater level, and where dewatering work is involved, the depth of investigation shall also be selected as a function of the hydrogeological conditions. Slopes and steps in the terrain shall be explored to depths below any potential slip surface.

NOTE   For the spacing of investigation points and investigation depths, the values given in B.3 can be used as guidance.

2.4.1.4 Sampling

(1)P The sampling categories (see 3.4.1 and 3.5.1), and the number of samples to be taken shall be based on:
−   the aim of the ground investigation;
−   the geology of the site;
−   the complexity of the geotechnical structure.

(2)P For identification and classification of the ground, at least one borehole or trial pit with sampling shall be available. Samples shall be obtained from every separate ground layer influencing the behaviour of the structure.

(3) Sampling may be replaced by field tests if there is enough local experience to correlate the field tests with the ground conditions to ensure unambiguous interpretation of the results.

(4) Further details on sampling are given in Section 3.
2.4.1.5 Groundwater

(1)P Groundwater measurements shall be planned and carried out in accordance with 3.6.

2.4.2 Laboratory tests

2.4.2.1 General

(1) Prior to setting up a test programme, the expected stratigraphy at the site should be established and the strata relevant for design selected to enable the specification of the type and number of tests in each stratum. Stratum identification should be a function of the geotechnical problem, its complexity, the local geology and the required parameters for design.

2.4.2.2 Visual inspection and preliminary ground profile

(1) Samples and trial pits should be inspected visually and compared with field logs of the drillings so that the preliminary ground profile can be established. For soil samples, the visual inspection should be supported by simple manual tests to identify the soil and to give a first impression of its consistency and mechanical behaviour.

(2) If distinct and significant differences in the properties between different portions of one stratum are found, the preliminary soil profile should be further subdivided.

(3) Where practicable, the quality of the sample should be assessed before laboratory tests are performed. Quality classes for soil samples are defined in Table 3.1.

2.4.2.3 Test programme

(1)P The type of construction, the type of ground and stratigraphy and the geotechnical parameters needed for design calculations shall be taken into account when setting up the laboratory test programme.

(2) The laboratory test programme depends in part on whether comparable experience exists. The extent and quality of comparable experience for the specific soil or rock should be established. The results of field observations on neighbouring structures, when available, should also be used.

(3)P The tests shall be run on specimens representative of the relevant strata. Classification tests shall be used to check whether the samples and test specimens are representative.

NOTE   This can be checked in an iterative way. In a first step, classification tests and strength index tests are performed on as many samples as possible to determine the variability of the index properties of a stratum. In a second step, an assessment of how representative the samples used for the strength and compressibility tests are of the stratum can be checked by comparing the results of the classification and strength index tests for the samples with all results from the classification and strength index tests for the stratum.

(4) The need for more advanced testing or additional site investigation as a function of the geotechnical aspects of the project, soil type, soil variability and computation model should be considered.

2.4.2.4 Number of tests

(1)P The necessary number of specimens to be tested shall be established depending on the homogeneity of the ground, the quality and amount of comparable experience with the ground and the geotechnical category of the problem.

(2) To allow for difficult soil, damaged specimens and other factors, additional test specimens should be made available, whenever possible.

(3) Depending on the test type, a minimum number of specimens should be investigated.

NOTE   A recommended minimum number for some test types can be taken from the tables in Annexes L to W (except Annexes O and T). The annexes can also be used to check whether the extent of the testing was sufficient.

(4) The minimum number of tests may be reduced if the geotechnical design does not need to be optimised and uses conservative values of the soil parameters, or if comparable experience or combination with field information applies.

2.4.2.5 Classification tests

(1) Soil and rock classification tests should be performed to determine the composition and index properties of each stratum. The samples for the classification tests should be selected in such a way that the tests are approximately equally distributed over the complete area and the full depth of the strata relevant for design. Thus the results should give the range of index properties of the relevant layers.

(2) The results of the classification tests should be used to check if the extent of the investigations was sufficient or if a second investigation stage is needed.

(3) Suitable routine classification tests for ground samples with various degrees of disturbance are presented in Table 2.2. The routine tests are generally performed in all phases of the ground investigation (see 2.2 (3)).

2.4.2.6 Tests on samples

(1)P Samples for testing shall be selected so as to cover the range of index properties of each relevant stratum.

(2) For a fill or a stratum of sand or gravel, reconstituted specimens may be tested. Reconstituted specimens should have approximately the same composition, density and water content as in-situ material.

Table 2.2 — Soil classification tests

Parameter	Type of soil							
	Clayey soil			**Silty soil**			**Sandy, gravelly soil**	
	Type of specimen			Type of specimen			Type of specimen	
	Undis-turbed	Dis-turbed	Re-moul-ded	Undis-turbed	Dis-turbed	Re-moul-ded	Dis-turbed	Re-moul-ded
Geological description and soil classification	X	X	X	X	X	X	X	X
Water content	X	(X)	(X)	X	(X)	(X)	(X)	(X)
Bulk density	X	(X)	—	X	(X)	—	—	—
Minimum and maximum densities	—	—	—	(X)	(X)	(X)	X	X
Atterberg (consistency) limits	X	X	X	X	X	X	—	—
Particle size distribution	X	X	X	X	X	X	X	X
Undrained shear strength	X	—	—	(X)	—	—	—	—
Permeability	X	—	—	X	(X)	(X)	(X)	(X)
Sensitivity	X	—	—	—	—	—	—	—

X = normal to determine
(X) = possible to determine, not necessarily representative
— = not applicable
NOTE   For some types of soil, further tests may be considered, for example the determination of organic content, particle density and activity.

(3) Laboratory tests to determine parameters for geotechnical calculations are given in Table 2.3.

(4) Suitable routine laboratory tests for rock samples giving the necessary basis for the description of the rock material are as follows:

−   the geological classification;
−   the density or bulk mass density (ρ) determination;
−   the water content (w) determination;
−   the porosity (n) determination;
−   the uniaxial compression strength (σ_C) determination;
−   the Young's modulus of elasticity (E) and Poisson's ratio (ν) determination;
−   the point load strength index test ($I_{s,50}$).

(5) The classification of rock core samples will normally comprise a geological description, the core recovery, the Rock Quality Designation (RQD), the degrees of induration, fracture log, weathering and fissuring. In addition to the routine tests mentioned in 2.4.2.6 (4) for rocks, other tests may be selected for different purposes, e.g. density of grains determination, wave velocity determination, Brazilian tests, shear strength of rock and joints determination, slake durability tests, swelling tests and abrasion tests.

(6) The properties of the rock mass including the layering and fissuring or discontinuities may be investigated indirectly by compression and shear strength tests along joints. In weak rocks, complementary tests in the field or large-scale laboratory tests on block samples may be made.

Table 2.3 — Laboratory tests for the determination of geotechnical parameters

Geotechnical parameter	Type of soil					
	Gravel	Sand	Silt	NC clay	OC clay	Peat organic clay
Oedometer modulus (E_{oed}); compression index (C_c); [one-dimensional compressibility]	(OED) (TX)	(OED) (TX)	OED (TX)	OED (TX)	OED (TX)	OED (TX)
Young's modulus (E); Shear modulus (G)	TX	TX	TX	TX	TX	TX
Drained (effective) shear strength (c'), (φ')	TX SB	TX SB	TX SB	TX SB	TX SB	TX SB
Residual shear strength (c'_R), (φ_R)	RS (SB)	RS (SB)	RS (SB)	RS (SB)	RS (SB)	RS (SB)
Undrained shear strength (c_u)	—	—	TX DSS SIT	TX DSS (SB) SIT	TX DSS (SB) SIT	TX DSS (SB) SIT
Bulk density (ρ)	BDD	BDD	BDD	BDD	BDD	BDD
Coefficient of consolidation (c_v)			OED TX	OED TX	OED TX	OED TX
Permeability (k)	TXCH PSA	TXCH PSA	PTC TXCH (PTF)	TXCH (PTF) (OED)	TXCH (PTF) (OED)	TXCH (PTF) (OED)

— = not applicable
( ) = partially applicable only; for details, see Section 5.

Abbreviations of laboratory tests:
BDD   Bulk density determination
DSS   Direct simple shear test
OED   Oedometer test
PTF   Permeability test in the falling head permeameter
PTC   Permeability test in the constant head permeameter
RS   Ring shear (Ring shear box test)
SB   Translational shear box test
SIT   Strength index tests (normally performed in the preliminary phase only)
PSA   Particle size analysis
TX   Triaxial test
TXCH   Permeability constant head test in the triaxial cell (or flexible wall permeameter)

2.5 Controlling and monitoring

(1)P A number of checks and additional tests shall be made during the construction and execution of the project, when relevant, in order to check that the ground conditions agree with

those determined in the design investigations and that the properties of the delivered construction materials and the construction works correspond to those presumed or specified.

NOTE   See also EN 1997-1:2004, Section 4.

(2)P The following control measures shall be applied:
−   check of ground profile when excavating;
−   inspection of the bottom of the excavation.

(3) The following general control measures may be applied:
−   measurements of groundwater level or pore pressures and their fluctuations;
−   measurements of the behaviour of neighbouring constructions, services or civil engineering works;
−   measurements of the behaviour of the actual construction.

NOTE   Controlling and monitoring is of prime importance when the observational method is used (see EN 1997-1:2004, 2.7).

(4)P The results of the control measures shall be compiled, reported and checked against the design requirements. Decisions shall be taken based on these findings.

Section 3 Soil and rock sampling and groundwater measurements

3.1 General

(1)P Sampling of soils and rocks by drilling and excavations and groundwater measurements shall be conducted so comprehensively that the necessary geotechnical design data are obtained.

3.2 Sampling by drilling

(1)P The drilling equipment shall be selected according to:

- the sampling categories required, as defined in 3.4.1 and 3.5.1;
- the depth to be reached and the required diameter of the sample;
- the functions required from the drilling rig, e.g. recording of the drilling parameters, automatic or manual adjustment.

(2)P The requirements of EN ISO 22475-1 shall be followed.

3.3 Sampling by excavation

(1)P If samples are recovered from trial pits, headings or shafts, the requirements of EN ISO 22475-1 shall be followed.

3.4 Soil sampling

3.4.1 Categories of sampling methods and laboratory quality classes of samples

(1)P Samples shall contain all the mineral constituents of the strata from which they have been taken. They shall not be contaminated by any material from other strata or from additives used during the sampling procedure.

(2)P Three sampling method categories shall be considered (EN ISO 22475-1), depending on the desired sample quality as follows (for sample quality see Table 3.1):

- category A sampling methods: samples of quality class 1 to 5 can be obtained;
- category B sampling methods: samples of quality class 3 to 5 can be obtained;
- category C sampling methods: only samples of quality class 5 can be obtained.

(3) Samples of quality classes 1 or 2 can only be obtained by using category A sampling methods. The intention is to obtain samples of quality classes 1 or 2, in which no or only slight disturbance of the soil structure has occurred during the sampling procedure or in the handling of the samples. The water content and the void ratio of the soil correspond to those in-situ. No change in constituents or in chemical composition of the soil has occurred. Certain unforeseen circumstances such as variations in geological strata may lead to lower sample quality classes being obtained.

(4) Using category B sampling methods will preclude achieving samples of quality classes better than 3. The intention is to obtain samples that contain all the constituents of the in-situ soil in their original proportions and for the soil to retain its natural water content. The general arrangement of the different soil layers or components can be identified. The structure of the soil has been

disturbed. Certain unforeseen circumstances such as variation in geological strata may lead to lower sample quality classes being obtained.

(5) By using category C sampling methods, samples of quality classes better than 5 cannot be obtained. The soil structure in the sample has been totally changed. The general arrangement of the different soil layers or components has been modified so that the in-situ layers cannot be identified accurately. The water content of the sample needs not represent the natural water content of the soil layer sampled.

(6)P Soil samples for laboratory tests are divided in five quality classes with respect to the soil properties that are assumed to remain unchanged during sampling and handling, transport and storage. The classes are described in Table 3.1, together with the sampling category to be used.

Table 3.1 — Quality classes of soil samples for laboratory testing and sampling categories to be used

Soil properties / quality class	1	2	3	4	5
Unchanged soil properties					
particle size	*	*	*	*	
water content	*	*	*		
density, density index, permeability	*	*			
compressibility, shear strength	*				
Properties that can be determined					
sequence of layers	*	*	*	*	*
boundaries of strata – broad	*	*	*	*	
boundaries of strata – fine	*	*			
Atterberg limits, particle density, organic content	*	*	*	*	
water content	*	*	*		
density, density index, porosity, permeability	*	*			
compressibility, shear strength	*				
Sampling category according to EN ISO 22475-1	A				
			B		
					C

3.4.2 Soil identification

(1)P Soil identification, based on the examination of the samples recovered, shall conform to EN ISO 14688-1.

3.4.3 Planning of soil sampling

(1)P The quality class and number of samples to be recovered shall be based on the aims of the soil investigations, the geology of the site, and the complexity of the geotechnical structure and of the construction to be designed.

(2) Two different strategies may be followed for sampling by drilling.

− Drilling aimed at recovering the complete soil column, with samples obtained by the drilling tools down the borehole and by special samplers at selected depths at the borehole bottom.

− Drilling to recover samples only at specific predetermined elevations, e.g. by separately conducted penetration tests.

(3)P The sampling categories shall be selected considering the desired laboratory quality classes, as detailed in Table 3.1, the expected soil types, and groundwater conditions.

(4)P The requirements of EN ISO 22475-1 shall be followed, for the selection of the drilling or excavation methods and sampling equipment adequate to the soil sampling category prescribed.

NOTE   The recovery of completely undisturbed samples is practically impossible due to, among other factors, the mechanical disturbance caused by the sampling operations and to the unavoidable stress release when recovering the sample. The effect of these factors on the degree of disturbance depends on the sampling category used and the types of soil being sampled. The type of soil being sampled has a decisive influence on the degree of disturbance of samples obtained by the same sampling methods. Thus very sensitive soils are prone to disturbance, while less sensitive soils, such as most stiff clays, may require less restrictive methods of sampling for obtaining fairly undisturbed samples. On the other hand, each problem requires a different degree of accuracy for the soil parameters to be used. As a consequence, when preparing a sampling programme, the factors mentioned above should be considered in order to decide the degree of disturbance that can be accepted and therefore the sampling methods to be required.

(5) For a given project, specific sampling equipment and methods may be required within the sampling categories defined in 3.4.1. For instance, this is the case when the deformation moduli (stiffness) at small strains have to be determined in undisturbed samples.

(6)P The dimensions of the samples to be recovered shall be in accordance with the type of soil and the type and number of tests to be performed.

NOTE   See Section 5 and Annexes L, M, N, P, Q, R and S.

(7) Samples should be taken at any change of stratum and at a specified spacing, usually not larger than 3 m. In inhomogeneous soil, or if a detailed definition of the ground conditions is required, continuous sampling by drilling should be carried out or samples recovered at very short intervals.

3.4.4 Handling, transport and storing of samples

(1)P Handling, transport and storing of samples shall be carried out in accordance with EN ISO 22475-1.

NOTE   For handling and storing in the laboratory, see Section 5.

3.5 Rock sampling

3.5.1 Categories of sampling methods

(1)P Samples shall contain all the mineral constituents of the strata from which they have been taken. They shall not be contaminated by any material from other strata or from additives used during the sampling procedure.

(2)P The discontinuities and corresponding infilling materials existing in the rock mass often control the strength and deformation characteristics of the material as a whole. Therefore, they shall be defined as closely as possible during the sampling operations, if such properties have to be determined.

(3)P Three sampling method categories shall be considered (see EN ISO 22475-1), depending on the quality of sample:

- − category A sampling methods;
- − category B sampling methods;
- − category C sampling methods.

(4) By using category A sampling methods, the intention is to obtain samples in which no or only slight disturbance of the rock structure has occurred during the sampling procedure or in handling of the samples. The strength and deformation properties, water content, density, porosity and the permeability of the rock sample correspond to the in-situ values. No change in constituents or in chemical composition of the rock mass has occurred. Certain unforeseen circumstances such as variations of the geological strata may lead to lower sample quality being obtained.

(5) By using category B sampling methods, the intention is to obtain samples that contain all the constituents of the in-situ rock mass in their original proportions and with the rock pieces retaining their strength and deformation properties, water content, density and porosity. By using category B sampling methods, the general arrangement of discontinuities in the rock mass can be identified. The structure of the rock mass has been disturbed and thereby the strength and deformation properties, water content, density, porosity and permeability for the rock mass itself. Certain unforeseen circumstances such as variations of the geological strata can lead to lower sample quality being obtained.

(6) Category C sampling methods lead to the structure of the rock mass and its discontinuities being totally changed. The rock material may have been crushed. Some changes in constituents or in chemical composition of the rock material can occur. The rock type and its matrix, texture and fabric can be identified.

3.5.2 Rock identification

(1)P Visual rock identification shall be based on examination of the rock masses and samples including all observations of decomposition and discontinuities. The identification shall conform to EN ISO 14689-1.

(2)P Weathering classification shall be related to the geological processes and shall cover the grades between fresh rock and rock decomposed into soil. The classification shall conform to 4.2.4 and 4.3.4 of EN ISO 14689-1:2003.

(3)P Discontinuities such as bedding planes, joints, fissures, cleavages and faults shall be quantified with respect to pattern, spacing and inclination using unambiguous terms. The quantification shall conform to 4.3.3 of EN ISO 14689-1:2003.

(4)P Rock quality designation (RQD), total core recovery (TCR), and solid core recovery (SCR), as defined by EN ISO 22475-1, shall be determined.

3.5.3 Planning of rock sampling

(1)P The characteristics and number of samples to be recovered shall be based on the aim of the site investigations, the geology of the area and the complexity of the geotechnical structure and of the construction to be designed.

(2)P The category of the sampling methods to be prescribed shall be selected according to the rock characteristics to be preserved, as detailed in 3.5.1, and the expected rock and groundwater conditions.

(3)P The requirements of EN ISO 22475-1 shall be followed for the selection of the drilling or excavation methods and sampling equipment.

(4) For a given project, specific sampling equipment and methods may be required within the rock sampling categories defined in 3.5.1.

3.5.4 Handling, transport and storing of samples

(1)P After sampling and visual inspection has taken place, the obtained cores shall be preserved, handled and stored according to EN ISO 22475-1.

3.6 Groundwater measurements in soils and rocks

3.6.1 General

(1)P Groundwater measurements shall conform to 2.1.4.

(2)P The determination of the groundwater table or pore water pressures in soils and rocks shall be made by installing open or closed groundwater measuring systems into the ground.

NOTE   Subclause 3.6 applies to the measurement of positive pore water pressures relative to the atmospheric pressure. Measurements of negative pore water pressures are not considered.

3.6.2 Planning and execution of the measurements

(1)P When relevant, groundwater measurements and sampling shall be conducted in accordance with EN ISO 22475-1.

(2)P The type of equipment to be used for groundwater measurements shall be selected according to the type and permeability of ground, the purpose of the measurements, the required observation time, the expected groundwater fluctuations and the response time of the equipment and ground.

(3) There are two main methods for measuring the groundwater pressure: open systems and closed systems. In open systems the piezometric groundwater head is measured by an observation well, usually provided with an open pipe. In closed systems the groundwater pressure at the selected point is directly measured by a pressure transducer.

(4) Open systems are best suited for soils and rock with a relatively high permeability (aquifers and aquitards), e.g. sand, gravel or highly fissured rock. With soils and rocks of low permeability they may lead to erroneous interpretations, due to the time lag for filling and emptying the pressure pipe. The use of filter tips connected to a small diameter hose in open systems, decreases the time lag.

(5) Closed systems can be used in all types of soil or rock. They should be used in very low permeability soils and rocks (aquicludes), e.g. clay or low fissured rock. Closed systems are also recommended when dealing with high artesian water pressure.

(6)P When very short- term variations or fast pore water fluctuations are to be monitored, continuous recording shall be used by means of transducers and data loggers, with any types of soils and rocks.

(7)P In cases where open water is situated within or close to the investigation area, the water level shall be considered in the interpretation of the groundwater measurements. The water level in wells, the occurrence of springs and artesian water shall also be noted.

(8)P The number, location and depth of the measuring stations shall be chosen considering the purpose of the measurements, the topography, the stratigraphy and the soil conditions, especially the permeability of the ground or identified aquifers.

(9)P For monitoring projects e.g. groundwater lowering, excavations, fillings and tunnels, the location shall be chosen with respect to the expected changes to be monitored.

(10) For reference purposes, measurement of the natural fluctuations in groundwater should be made, if possible, outside the area affected by the actual project.

(11)P In order to obtain measurements reflecting the pore pressure at the intended point in a soil or rock layer, provisions shall be made, according to EN ISO 22475-1, to ensure that the measuring point is adequately sealed off with regard to other layers or aquifers.

(12)P The number and frequency of readings and the length of the measuring period for a given project shall be planned considering the purpose of the measurements and the stabilisation period.

(13) The criteria adopted should be adjusted after an initial period of time, according to the actual variations of the readings being observed.

(14)P If it is intended to assess groundwater fluctuations, measurements shall be taken at intervals smaller than the natural fluctuations to be characterised and over a long period of time.

(15) During the drilling process, the observation of the water level at the end of the day and the start of the following day (before the drilling is resumed) is a good indication of the groundwater conditions and should be recorded. Any sudden inflow or loss of water during drilling should also be recorded, since it can provide additional useful information.

(16) During the first phases of site investigations, some of the boreholes may be equipped with open perforated pipes protected with filters. The water level readings obtained during the following days yield a preliminary indication of groundwater conditions, but are subject to the limitations mentioned in 3.6.2 (4). The dangers associated with the connection of different aquifers should be taken into account, as well as any relevant environmental regulations.

3.6.3 Evaluation of results of groundwater measurements

(1)P The evaluation of groundwater measurements shall take into account the geological and geotechnical conditions of the site, the accuracy of individual measurements, the fluctuations of pore water pressures with time, the duration of the observation period, the season of measurements and the climatic conditions during and prior to that period.

(2)P The evaluated results of groundwater measurements shall comprise the observed maximum and minimum elevations of the water table, or pore pressures and the corresponding measuring period.

(3)P If applicable, upper and lower bounds for both extreme and normal circumstances shall be derived from the measured values, by adding or subtracting the expected fluctuations or a reduced part of them, to the respective extreme or normal circumstances. The frequent lack of reliable data for extended periods of time of this type of measurements will necessitate the derived values being a cautious estimate based on the limited available information.

(4) The need for making further measurements or installing additional measuring stations should be assessed during the field investigations and in the ground investigation report.

NOTE   In Annex C, an example is presented of a statistical method to evaluate the groundwater conditions, when long term measurements in a reference pipe in the region are available and a short measuring period on the actual site has been carried out.

Section 4   Field tests in soil and rock

4.1 General

(1)P When field tests are conducted, they shall be linked to sampling by excavating and drilling, in order to collect information on the ground stratification and to obtain geotechnical parameters or direct input for design methods (see also EN 1997-1:2004, 3.3).

(2)P Field tests shall be planned considering the following general points (see also Section 2):
−   geology/stratification of the ground;
−   type of structure, the possible foundation and the anticipated work during the construction;
−   type of geotechnical parameter required;
−   design method to be adopted.

(3) The tests or combinations thereof should be selected from the following types, contained in the Parts of EN ISO 22476 and covered in this Section:
−   cone penetration test;
−   pressuremeter and dilatometer tests;
−   standard penetration test;
−   dynamic probing;
−   weight sounding test;
−   field vane test;
−   flat dilatometer test;
−   plate loading test.

Table 2.1 gives a broad overview for the applicability of the tests in different ground conditions.

(4) Other complementary internationally recognised investigation methods, for instance geophysical methods, may be used.

4.2 General requirements

4.2.1 Planning a specific test programme

(1)P In addition to the recommendations given in 2.3 and the requirements given in 2.4 and 4.1 (2), the following information shall be established:
−   ground profile to be expected;
−   desired total depth of investigation;
−   elevation of ground surface and if applicable groundwater level.

(2)P When designing the ground investigation programme, the selection of the type of field tests and of the test equipment shall aim at obtaining the best technical and economical solution for the intended purpose.

NOTE   see also Table 2.1 and B.2.

4.2.2 Execution

AC₁ (1)P For the tests covered in this Section, equipment and procedures shall correspond to the requirements in EN ISO 22476-1 to EN ISO 22476-9, EN ISO 22476-12 and EN ISO 22476-13. AC₁

NOTE   Further information on a procedure, presentation and evaluation of the weight sounding test and the flat dilatometer test can be found in CEN ISO/TS 22476-10 and CEN ISO/TS 22476-11 respectively.

(2)P If the results obtained during the ongoing investigation do not correspond to the initial information (see Section 2) about the test site and/or the aim of the investigation, additional measures shall be considered such as:
− additional tests;
− changing to different test methods.

(3) If the desired investigation depth is not reached, the client should be informed immediately.

4.2.3 Evaluation

(1)P In evaluating the field test results, especially in the context of deriving geotechnical parameters/coefficients from the results, any additional information about the ground conditions shall be considered.

(2)P Results from any sampling by drilling and excavations according to Section 3 shall be available and shall be used in evaluating the test results.

(3)P In evaluating the test results, the possible geotechnical and equipment influences on the measured parameters shall be considered. When a soil or rock formation exhibits anisotropy, attention shall be paid to the axis of loading with respect to the anisotropy.

(4)P If correlations are used to derive geotechnical parameters/coefficients, their suitability shall be considered for each particular project.

(5)P When using Annexes D to K, it shall be ensured that the ground conditions of the site under investigation (soil type, uniformity coefficient, consistency index etc.) are compatible with the boundary conditions given for the correlation. Local experience shall be used for confirmation, if available.

NOTE 1   Annexes D to K give examples of correlations for the establishment of derived values and for the application of test values to design methods.

NOTE 2   X.3 contains examples of correlations for the establishment of derived values from test results and also the use of test results directly in design.

ⒶⒸ₁ 4.3 Cone penetration and piezocone penetration tests (CPT, CPTU, CPTM) ⒶⒸ₁

4.3.1 Objectives

(1) The objective of the cone penetration test (CPT) is to determine the resistance of soil and soft rock to the penetration of a cone and the local friction on a sleeve.

(2)P The CPT consists of pushing a cone penetrometer vertically into the soil using a series of push rods. The cone penetrometer shall be pushed into the soil at a constant rate of penetration. The cone penetrometer comprises the cone and if appropriate a cylindrical shaft or friction sleeve. The penetration resistance of the cone (q_c) as well as, if appropriate, the local friction on the friction sleeve shall be measured.

(3)P For electrical CPTs, all measurements shall be made by sensors contained in the cone penetrometer.

(4) For mechanical CPTs, the measurements are generally made remotely.

(5) The piezocone penetration test, CPTU, is an electrical CPT, which includes additional instrumentation to measure the pore water pressure during penetration at the level of the base of the cone.

(6) The CPTU results should be used mainly for the determination of a soil profile together with results from sampling by drilling and excavations according to Section 3 or in comparison with other field tests.

(7) The results may also be used for the determination of geotechnical parameters such as the strength and deformation properties of soil and soft rock provided penetration is possible, and for direct input to design methods, generally in coarse and fine soil but also possibly in other deposits.

(8) The results may also be used to determine the length of piles and their compressive or tensile resistance or the dimensions of shallow foundations.

4.3.2 Specific requirements

ⒶⒸ₁ (1)P The tests shall be carried out and reported in accordance with a method that conforms to the requirements given in EN ISO 22476-1 for the electrical CPT and CPTU, or EN ISO 22476-12 for the CPTM. ⒶⒸ₁

(2)P When planning the test programme for a project, the following items shall be decided in addition to the requirements given in 4.2.1:
− type of required cone penetration test according to EN ISO 22476-1 or EN ISO 22476-12;
− depth and duration of pore pressure dissipation tests, if applicable.

(3)P Any deviation from the requirements given in EN ISO 22476-1 or
EN ISO 22476-12 shall be justified and reported. In particular, any influence on the results shall be commented upon.

4.3.3 Evaluation of test results

(1)P In addition to the requirements given in 4.2, the field and test reports according to EN ISO 22476-1 or EN ISO 22476-12 shall be used for evaluation purposes.

(2)P Possible geotechnical influences on the penetration resistance shall be considered in evaluating the test results, e.g. in clays, the cone penetration resistance corrected for pore water pressure effects, (q_t), should be used in evaluation.

4.3.4 Use of test results and derived values

4.3.4.1 Bearing resistance and settlement of spread foundations

(1)P If the bearing resistance or the settlement of a spread foundation is derived from CPT results, either a semi-empirical or an analytical design method shall be used.

NOTE   See, for example, EN 1997-1:2004, Annex D or F.

(2)P If a semi-empirical method is used, all the features of the method shall be taken into account.

NOTE   If, for instance, the semi-empirical method to determine the settlement of spread foundations from CPT results is used (see D.3), only Young's modulus of elasticity derived from q_c is applied in this particular method as shown in the example.

(3) If the sample analytical method for bearing resistance of Annex D in EN 1997-1:2004 is used, the undrained shear strength of fine soil, (c_u) may be determined for a CPT from:

$$c_u = \frac{q_c - \sigma_{v0}}{N_k} \tag{4.1}$$

Or, in the case of a CPTU, from:

$$c_u = \frac{q_t - \sigma_{v0}}{N_{kt}} \tag{4.2}$$

where

q_c	is the cone penetration resistance
q_t	is the cone penetration resistance corrected for pore water pressure effects;
N_k and N_{kt}	are coefficients estimated from local experience or reliable correlations
σ_{vo}	is the initial total vertical overburden stress at the depth under consideration;

(4) If the sample analytical method for bearing resistance calculation of Annex D of EN 1997-1:2004 is used, the angle of shearing resistance (φ') may be determined from the cone resistance (q_c), on the basis of local experience, taking into account depth effects, when relevant.

NOTE 1   An example of ranges of values to estimate φ' from q_c for quartz and feldspar sands is given in D.1, for estimating the bearing resistance of spread foundations when depth effects do not need to be taken into account.

NOTE 2   In addition, an example for a correlation between φ' and q_c for poorly-graded sands is given in D.2. The correlation given in D.2 should be considered as giving a conservative estimate.

(5) More elaborate methods may also be used for determining φ' from q_c, taking into account the effective vertical stress, the compressibility, and the over-consolidation ratio.

(6) If an adjusted elasticity method is used for calculating settlements of spread foundations from CPT results, the correlation between cone resistance (q_c) and the drained (long term) Young's modulus of elasticity (E') depends on the nature of the method: the semi-empirical elasticity method, or the theoretical elastic method.

NOTE   An adjusted elasticity method is given in EN 1997-1:2004, Annex F.

(7) Semi-empirical methods may be used for calculating settlements in coarse soil.

NOTE   An example is given in D.3.

(8) When a theoretical elastic method is used, the drained (long term) Young's modulus of elasticity (E') may be determined from cone resistance (q_c), on the basis of local experience.

NOTE   An example of sample values for quartz and feldspar sands is given in D.1 to estimate a value of E' from q_c.

(9) Correlations between the oedometer modulus (E_{oed}) and the cone resistance (q_c) may also be used when calculating settlements of spread foundations. The following relationship between E_{oed} and q_c is often adopted:

$$E_{oed} = \alpha \times q_c \qquad\qquad (4.3)$$

where

α is a correlation factor depending on local experience.

NOTE   An example of a correlation is given in D.4.

(10) When a theoretical elastic method is used to calculate the settlements of spread foundations, a stress dependant oedometer modulus (E_{oed}), based on q_c, may be used.

NOTE 1   For examples of theoretical elastic methods, see EN 1997-1:2004, Annex F.

NOTE 2   Examples of correlations between q_c and E_{oed} are given in D.5. The correlations given in D.5 should be considered as conservative estimates.

4.3.4.2 Pile bearing resistance

(1)P If the ultimate compressive or tensile resistance of piles according to EN 1997-1:2004, 7.6.2.3 or 7.6.3.3 is derived from CPT results, calculation rules based on locally established correlations between the results of static load tests and CPT results shall be used.

NOTE 1   An example for such correlations for coarse soil is shown in D.6.

NOTE 2   An example is given for the assessment of the compressive resistance of a single pile on the basis of q_c-values from a CPT in D.7.

4.4 Pressuremeter tests (PMT)

4.4.1 Objectives

(1) The objective of the pressuremeter test is to measure in-situ the deformation of soil and soft rock caused by the expansion of a cylindrical flexible membrane under pressure.

(2)P The test consists of inserting a probe containing a cylindrical flexible membrane into the ground either into a pre-formed borehole, or by self-boring or by full displacement pushing. Once at a predetermined depth the membrane is expanded under pressure and readings of pressure and expansion are recorded until a maximum expansion for the particular device is reached.

NOTE   Expansion is measured from radial displacement, or calculated from volume change of the cylindrical membrane.

(3) The test should be used to derive strength and/or deformation parameters of the ground or specific pressuremeter parameters.

(4) The results may be used to derive stress-strain curves in fine soil and soft rock.

4.4.2 Specific requirements

(1)P When planning a test programme for a project the type of pressuremeter to be used shall be specified.

(2)P There are four different types of apparatus generally available, for which the corresponding standards shall be used:
- pre-bored pressuremeters (PBP), e.g. the flexible dilatometer test (FDT), according to EN-ISO 22476-5;
- the Ménard pressuremeter (MPM), a specific form of the PBP, according to EN ISO 22476-4;
- the self-boring pressuremeter (SBP), according to EN ISO 22476-6;
- the full-displacement pressuremeter (FDP), according to EN ISO 22476-8.

NOTE   The PBP and the MPM are lowered into a test hole created specifically for the pressuremeter test. The SBP is drilled into the ground using an integral cutting head at its lower end such that the probe replaces the material it removes thereby creating its own test hole. The FDP is usually pushed into the ground with an integral cone at its lower end, thereby creating its own test hole. The MPM may in some instances be pushed or driven into the ground. PBP, SBP and FDP probes may take a number of forms, in accordance with the type of installation and measuring systems.

(3) Two different basic test procedures may be used:
- a procedure to obtain a pressuremeter modulus, (E_M), and limit pressure, (p_{LM}), that may be used in design procedures formulated for the Ménard pressuremeter; and
- a procedure to obtain other stiffness and strength parameters.

(4)P The tests shall be carried out and reported in accordance with a test method that conforms to the requirements for the particular instrument type to be used,
(see 4.4.2 (2)P).

(5)P Any deviations from the requirements given in the corresponding standard shall be justified and in particular their influence on the results shall be commented upon.

4.4.3 Evaluation of test results

(1)P If necessary, the applied pressure shall be corrected for membrane stiffness to obtain the true pressure applied to the cylindrical ground contact surface around the probe.

(2)P If a radial displacement type pressuremeter is used, the displacement readings shall be converted to cavity strain and, if testing weak rock, corrected for membrane compression and thinning.

(3)P If a volume displacement type pressuremeter is used (e.g. MPM), the volume reading shall be corrected for system expansion.

(4)P In addition to the requirements given in 4.2, the field and test reports according to EN ISO 22476-4 EN ISO 22476-5 EN ISO 22476- 6 and EN ISO 22476-8 for the specific test type shall be used for the basis of any further evaluation.

(5) In addition to the plots required by the individual equipment test standard, the list of additional plots in Table 4.1 should be considered.

Table 4.1 — A list of additional plots

Probe	Ground type	Abscissa	Ordinate
Radial displacement type			
Self-bored, pushed in	All	Cavity strain for each arm	Applied pressure
Pre-bored	All	Cavity strain for each pair of arms	Applied pressure
Self bored	All	initial cavity strain for each arm	Applied pressure
All	All	Cavity strain for unload-reload cycle for each arm	Applied pressure
All	Clay	Logarithm of cavity strain for each arm	Applied pressure
All	Sands	Natural logarithm of current cavity strain for each arm	Natural logarithm of effective applied pressure
Volume displacement type (except MPM) [a]			
Pre-bored	All	Volume change	Applied pressure
Pre-bored	All	Rate of change of volume	Applied pressure
[a] For MPM tests, the pressure is plotted as abscissa and the volume change as ordinate.			

4.4.4 Use of test results and derived values

4.4.4.1 General criteria

(1)P If an indirect or analytical design method is used, the geotechnical parameters of shear strength and shear modulus shall be derived from the pressuremeter curve using methods relevant for the particular test and equipment type.

(2)P If a direct or semi-empirical design method is used, all the features of the method shall be taken into account.

NOTE   Direct foundation design procedures use directly the measurements from field tests, instead of conventional soil properties.

(3)P If, for instance, the semi-empirical method to determine the settlement of spread foundations from MPM results is used, only the modulus (E_M) determined from Ménard pressuremeter results shall be applied in this particular method.

NOTE   Examples of calculations of settlements can be found in E.2.

4.4.4.2 Bearing resistance of spread foundations

(1)P If a semi-empirical method is used, then all aspects relating to the method need to be followed, in particular the specification for the pressuremeter type used in establishing the method. EN ISO 22476-4 shall be followed.

NOTE 1   The semi-empirical method according to Ménard is given in EN 1997-1:2004, Annex E.

NOTE 2   An example of the calculation of the bearing resistance is given in E.1.

(2) If an analytical method is used, the strength of the soil may be determined using empirical and theoretical methods but only on the basis of local experience.

NOTE   Examples of analytical methods are given in EN 1997-1:2004, Annex D.

(3) The angle of shearing resistance (φ') may be determined from an SBP test in coarse soil by theoretical methods and from FDP and PBP tests using empirical correlations but only on the basis of local experience.

4.4.4.3 Settlement of spread foundations

(1) The settlement of spread foundations may be determined from MPM tests using a semi-empirical method.

NOTE   An example of the calculation is given in E.2.

(2) If an analytical method is used, the stiffness of the soil may be determined using theoretical models to interpret the pressuremeter test but only on the basis of local experience.

NOTE   Examples of analytical methods are given in EN 1997-1:2004, Annex F.

4.4.4.4 Pile bearing resistance

(1) The ultimate compressive resistance of piles may be derived directly from stress controlled tests.

NOTE   An example of the calculation of the ultimate compressive resistance is given in E.3.

(2) When the ultimate compressive or tensile resistance of a pile is derived indirectly from pressuremeter test results, an analytical method may be applied to derive values of base and shaft resistance but only on the basis on local experience.

4.5 Flexible dilatometer test (FDT)

4.5.1 Objectives

(1) The objective of the flexible dilatometer test is to measure in-situ the deformability of rock (rock dilatometer test, RDT), and soil (soil dilatometer test, SDT) from measurements of the radial expansion of a borehole section under a known uniform radial pressure applied by means of a cylindrical dilatometer probe.

(2)P The test consists of inserting a cylindrical probe, having an outer expandable flexible membrane, into a borehole, and measuring, at selected time intervals or in a semi-continuous manner, the radial displacement of the borehole while inflating the probe under known radial pressure.

(3) The RDT should be used mainly in soft and hard rock formations while the SDT should be used primarily in soft to stiff soil to obtain profiles of deformability variations with depth.

(4) The results of cylindrical dilatometer tests may be used to determine the deformation and creep properties in-situ when testing intact rock.

(5) In fragile or clayey rock, and in fractured or closely jointed formations, where core recovery is poor or inadequate for the purpose of obtaining representative samples for laboratory testing, the cylindrical dilatometer test may be used for rapid index logging of boreholes and for comparisons of relative deformability of different rock strata.

4.5.2 Specific requirements

(1)P When planning a test programme for a project, the specific requirements of the device to be used shall be specified.

(2)P The tests shall be carried out and reported in accordance with a test method that conforms to EN ISO 22476-5.

(3)P Any deviations from the requirements given in EN ISO 22476-5 shall be justified and in particular their influence on the results shall be commented upon.

4.5.3 Evaluation of test results

(1)P In addition to the requirements given in 4.2, the field and test reports according to EN ISO 22476-5 for the specific test type shall be used for evaluation purposes.

(2) The interpretation of flexible dilatometer tests requires that the Poisson's ratio of the soil or rock should either be known or assumed.

4.5.4 Use of test results and derived values

(1) The results of dilatometer tests may be used to check the serviceability limit state of spread foundations on soil or rock through a deformation analysis.

(2) When performing a deformation analysis, the Young's modulus of elasticity (E) may be taken equal to the dilatometer modulus (E_{FDT}) on the assumption that the soil or rock is linearly elastic and isotropic.

(3)P When an indirect or analytical design method is used, the geotechnical parameters of shear modulus shall be derived from the dilatometer curve using methods relevant for that particular test type.

4.6 Standard penetration test (SPT)

4.6.1 Objectives

(1) The objectives of the standard penetration test are the determination of the resistance of soil at the base of a borehole to the dynamic penetration of a split barrel sampler (or solid cone) and the obtaining of disturbed samples for identification purposes.

(2)P The sampler shall be driven into the soil by dropping a hammer of 63,5 kg mass onto an anvil or drive head from a height of 760 mm. The number of blows (N) necessary to achieve a penetration of the sampler of 300 mm (after its penetration under gravity and below a seating drive) is the penetration resistance.

(3) The test should be used mainly for the determination of the strength and deformation properties of coarse soil.

(4) Valuable additional data may also be obtained in other types of soil.

4.6.2 Specific requirements

(1)P The tests shall be carried out and reported in accordance with EN ISO 22476-3.

(2)P Any deviation from the requirements given in EN ISO 22476-3 shall be justified and in particular its influence on the results of the test shall be commented upon.

4.6.3 Evaluation of test results

(1)P In addition to the requirements given in 4.2, the field and test reports according to EN ISO 22476-3 shall be used for evaluation purposes.

(2)P Existing design methods of foundations based on the SPT are of empirical nature. Equipment-related operating methods have been adapted to obtain more reliable results. Therefore, the application of appropriate correction factors for interpreting the results shall be considered (see EN ISO 22476-3).

(3)P The energy ratio (E_r) has to be known for the equipment if the results are to be used for the quantitative evaluation of foundations or for the comparison of the results. E_r is defined as the ratio of the actual energy E_{meas} (measured energy during calibration) delivered by the drive-weight assembly into the drive rod below the anvil, to the theoretical energy (E_{theor}) as calculated for the drive-weight assembly. The measured number of blows (N) shall be corrected accordingly (see EN ISO 22476-3).

(4) In sands, the energy losses due to rod length and the effect of effective overburden pressure should be taken into account accordingly (see EN ISO 22476-3:2005, A.2 and A.4).

(5) Other corrections should be considered, such as taking into account the use of liners (see EN ISO 22476-3:2005, A.3) or the use of a solid cone.

4.6.4 Use of test results and derived values

4.6.4.1 General criteria

(1) When dealing with sands, a wide empirical experience in the use of this test is available, such as for the quantitative evaluation of the density index, the bearing resistance and the settlement of foundations, even though the results should be considered as only a rough approximation. Most of the existing methods are still based on uncorrected or partly corrected values.

(2) There is no general agreement on the use of the SPT results in clayey soil. In principle, it should be restricted to a qualitative evaluation of the soil profile or to a qualitative estimate of the strength properties of the soil.

(3) The SPT results may sometimes be used in a quantitative way in clayey soil under well-known local conditions, when directly correlated to other appropriate tests.

4.6.4.2 Bearing resistance of spread foundations in sands

(1) If an analytical method for the calculation of bearing resistance is used, the effective angle of shearing resistance (φ) may be derived from SPT results.

NOTE   For examples of analytical methods for the calculation of bearing resistance, see EN 1997-1:2004, Annex D.

(2) The value of φ' may be derived empirically from:
−   direct correlations with SPT results;
−   correlations with density index, where the density index is derived from SPT results.

NOTE 1   See for example F.1 and F.2.

NOTE 2   The relationships given in F.1 can be used to determine the density index I_D from either N_{60} or $(N_1)_{60}$, (see EN ISO 22476-3).

(3) The resistance of sand to deformation is often increased the longer the geological period of consolidation. This "ageing" effect is reflected in higher blow counts and should be taken into account.

(4) Over-consolidation should be taken into account because it increases the blow counts, for the same values of I_D and σ'_{v0}.

NOTE 1   In F.1, some sample correlations are shown by the means of which the effect of both ageing and over-consolidation can be taken into account.

NOTE 2   When correcting for over-consolidation and ageing effects, the resultant derived ϕ values, using the density index, from the correlations in F.2 can be conservative.

4.6.4.3 Settlement of spread foundations in sand

(1) If a purely elastic design method is used, the drained Young's modulus of elasticity (E') may be derived from the N-values through empirical correlations.

(2) Alternatively, the density index may be derived based on the N_{60}-value. Then an appropriate correlation may be used to obtain E' through the density index.

(3) The direct design methods are based on comparisons of the N-values and results of plate loading tests or records of measured settlements of foundations. Allowable bearing resistance for a maximum settlement of 25 mm or the settlement corresponding to a given applied pressure can be obtained through the corresponding procedures with reference to the width of the footing, its embedment in the ground and groundwater table position.

NOTE   The sample method for the calculation of the settlements caused by spread foundations in sand, as given in F.3, can be used.

4.6.4.4 Pile bearing resistance in sand

(1)P If the ultimate compressive or tensile resistance of piles is derived from SPT results according to EN 1997-1:2004, 7.6.2.3 or 7.6.3.3, calculation rules based on locally established correlations between the results of static load test and SPT results shall be used.

4.7 Dynamic probing tests (DP)

4.7.1 Objectives

(1) The objective of the dynamic probing tests is to determine the resistance of soil and soft rock in-situ to the dynamic penetration of a cone.

(2)P A hammer of a given mass and falling height shall be used to drive the cone. The penetration resistance is defined as the number of blows required to drive the penetrometer over a defined distance. A continuous record shall be provided with respect to depth. No samples are recovered.

(3) The test results should be used particularly for the determination of a soil profile together with results from sampling by drilling and excavations according to Section 3 or as a relative comparison of other in-situ tests.

(4) The results may also be used for the determination of the strength and deformation properties of soil, generally of the coarse type but also possibly in fine soil, through appropriate correlations.

(5) The results can also be used to determine the depth to very dense ground layers indicating for instance the length of end bearing piles.

4.7.2 Specific requirements

(1)P For planning the specific test programme for a project, in addition to the requirements given in 4.2.1, the type of required DP test according to EN ISO 22476-2, shall be decided upon.

(2)P The tests shall be carried out and reported in accordance with EN ISO 22476-2.

NOTE   Five procedures are available according to EN ISO 22476-2, covering a wide range of specific work per blow: DPL, DPM, DPH, DPSH-A and DPSH-B as follows.
− Dynamic probing light (DPL): test representing the lower end of the mass range of dynamic penetrometers. Blow count: N_{10L}.
− Dynamic probing medium (DPM): test representing the medium mass range of dynamic penetrometers. Blow count: N_{10M}.
− Dynamic probing heavy (DPH): test representing the medium to very heavy mass range of dynamic penetrometers. Blow count: N_{10H}.
− Dynamic probing super heavy (DPSH-A and DPSH-B): tests representing the upper end of the mass range of dynamic penetrometers closely related to the dimensions of the SPT. Blow count: N_{10SA}, or N_{20SA}, N_{10SB} or N_{20SB}.

(3)P Any deviation from the requirements given in EN ISO 22476-2 shall be justified and in particular its influence on the results of the test shall be commented upon.

NOTE   Deviations exist with respect to:
− falling height and hammer mass;
− dimensions of the cone: e.g. an area of 10 cm^2 for the DPM cone, instead of 15 cm^2 as specified in EN ISO 22476-2:2005, clause 4.

(4) In locations with special difficulties of accessibility, lighter equipment and procedures other than those specified in EN ISO 22476-2 may be used.

4.7.3 Evaluation of test results

(1)P In addition to the requirements given in 4.2, the field and test reports according to EN ISO 22476-2 shall be used for evaluation purposes.

(2)P Possible geotechnical and equipment influences on the penetration resistance according to EN ISO 22476-2:2005, 5.4 shall be considered in evaluating the test results.

4.7.4 Use of test results and derived values

(1) For coarse soil, it is possible to obtain correlations with some geotechnical parameters and field tests. The correlations may be used in a quantitative evaluation for foundation design, provided the friction along the rods is negligible, or duly corrected.

(2) For fine soil, the quantitative use of the results should be employed only under well-known local conditions and supported by specific correlations. The skin friction during the test is a factor of special concern with this type of soil and should be duly taken into account.

(3) Several correlations have been established among the different dynamic probing tests and between them and other tests or geotechnical parameters. In some cases the friction along the rods has been eliminated or corrected, but the actual energy transmitted to the probe has not been measured. Therefore they cannot be considered valid in general.

NOTE 1   Examples of such correlations are included in Annex G.

NOTE 2   The correlations given in Annex G should be considered as conservative estimates.

(4) If an analytical method for bearing resistance of spread foundations is used, the angle of shearing resistance (φ') of coarse soil may be determined from the number of blows and the corresponding density index (I_D) with correlations.

NOTE 1   An example of an analytical method is given in EN 1997-1:2004, D.4.

NOTE 2   Such correlations for the determination of φ' are given as examples in G.1 and G.2.

(5) If a theoretical elastic method is applied to calculate the settlements of spread foundations, the oedometer modulus (E_{oed}) derived from the number of blows may be used.

NOTE 1   Examples of theoretical elastic methods are given in EN 1997-1:2004, Annex F.

NOTE 2   Corresponding examples of correlations for the determination of the oedometer modulus are given in G.3.

(6) If well-established correlations between ultimate compressive resistance from static pile load tests (see EN 1997-1:2004, 7.6.2.3) and cone penetration resistance (q_c) in coarse soil are used for the design, q_c may be estimated from N_{10} or N_{20} values using established relationships.

NOTE 1   Examples for DPH correlations are given in G.4.

NOTE 2   An example of correlations between the results of different dynamic probing tests is given in G.5.

4.8 Weight sounding test (WST)

4.8.1 Objectives

(1) The objective of the weight sounding test is the determination of the resistance of soil in situ to the static and/or rotational penetration of a screw-shaped point.

(2)P The weight sounding test shall be made as a static sounding in soft soil if the penetration resistance is less than 1 kN. If the resistance exceeds 1 kN, the penetrometer shall be rotated, manually or mechanically, and the number of half-turns for a given depth of penetration recorded. A continuous record is provided with respect to depth but no samples are recovered.

NOTE   Further information on a procedure, presentation and evaluation for the WST can be found in CEN ISO/TS 22476-10, (see X.3.5).

(3) The weight sounding test should primarily be used to give a continuous soil profile and an indication of the layer sequence. The penetrability in even stiff clays and dense sands is good.

(4) The weight sounding test may also be used to estimate the density index of coarse soil.

(5) The results can also be used to determine the depth to very dense ground layers indicating the length of end bearing piles.

4.8.2 Specific requirements

(1) The tests should be carried out and reported in accordance with a recognised method.

(2)P Any deviation from the requirements in the method referred to in (1) shall be justified and in particular its influence on the results of the test shall be commented upon.

NOTE   Further information on a procedure, presentation and evaluation for the weight sounding test can be found in CEN ISO/TS 22476-10.

4.8.3 Evaluation of test results

(1)P The requirements given in 4.2 shall be followed for the evaluation of the test results.

(2) In addition, the field and test reports, according to the method referred to in 4.8.2 (1), should be used for evaluation purposes.

NOTE   Further information on a procedure, presentation and evaluation for weight sounding test can be found in CEN ISO/TS 22476-10.

⟨AC₁⟩ (3) The following influences can affect the evaluation of the results. ⟨AC₁⟩
- The variations of the resistance with depth can depend on the variations in the soil layer sequence.
- In very soft to firm clays, the resistance is often less than 1 kN or approximately constant and less than 10 half-turns per 0,2 m of penetration.
- As the sensitivity of the clay also influences the penetration resistance, the strength of the clay cannot be determined directly from the penetration resistance without a calibration for each site.
- In very loose to loose sediments of silt and sand, rather low and constant penetration resistances are obtained.
- In medium dense to dense silts and fine sands, higher (10 to 30 half-turns per 0,2 m of penetration) resistances are obtained, which remain approximately constant with depth.
- In sand and gravel sediments, the variation in penetration resistances increases with the grain size.
- In silty sands and coarse gravel, a high penetration resistance does not always correspond to higher density or strength and deformation properties.

4.8.4 Use of test results and derived values

(1)P When the bearing resistance or the settlement of a spread foundation is derived from weight sounding test results, an analytical design method shall be used.

(2) If an analytical method for bearing resistance is used, the angle of shearing resistance φ' may be determined from correlations with weight sounding resistance.

NOTE   Examples of analytical methods are given in EN 1997-1:2004, Annex D.

(3) Such correlations should be based upon comparable experience, relevant to the design situation.

NOTE   Annex H presents an example correlation, derived for quartz and feldspar sands in a European region.

(4) If an adjusted elasticity method is used for calculating settlements of spread foundations from weight sounding results, the drained (long term) Young's modulus of elasticity (E') may be determined from weight sounding resistance on the basis of local experience. In the case of

quartz and feldspar sands, for example, the angle of shearing resistance (ϕ) may be estimated from the weight sounding resistance.

NOTE 1   Such an adjusted elasticity method is given in EN 1997-1:2004, Annex F.

[AC1] NOTE 2 Examples for correlations to estimate the drained (long term) Young's modulus of elasticity (E') and the angle of shearing resistance (ϕ) of quartz and feldspar sands are given in Annex H. [AC1]

(5) In coarse soil, the weight sounding resistance may also be used in direct estimation of the bearing capacity of spread foundation and piles.

(6) In fine soil, the weight sounding resistance may be used to estimate the undrained shear strength of soil, based on local experience, considering the sensitivity of the soil and water conditions in the borehole.

4.9 Field vane test (FVT)

4.9.1 Objectives

(1) The objectives of the field vane test are the measurement of the resistance to rotation in-situ of a vane installed in soft fine soil for the determination of the undrained shear strength and the sensitivity.

(2)P The field vane test shall be carried out with a rectangular vane, consisting of four plates fixed at 90° angles to each other, pushed into the soil to the desired depth and rotated.

(3) The field vane test may also be used for the determination of the undrained shear strength in stiff clays, silts and glacial clays. The reliability of test results varies depending on the type of soil.

(4) After extensive rotation of the vane, whereby the soil along the failure surface becomes thoroughly remoulded, the remoulded shear strength value can be measured and the soil's sensitivity can be calculated.

4.9.2 Specific requirements

(1) The tests should be carried out and reported in accordance with requirements given in EN ISO 22476-9.

(2)P Any deviation from the requirements given in EN ISO 22476-9 shall be justified, and in particular its influence on the results of the test shall be commented upon.

4.9.3 Evaluation of test results

(1)P In addition to the requirements given in 4.2, the field and the test reports according to EN ISO 22476-9 shall be used for evaluation purposes.

(2) The results of other field test, e.g. from CPT, SPT, WST or DP if conducted, should be available and considered.

4.9.4 Use of test results and derived values

(1)P If the bearing resistance of a spread foundation, the ultimate compressive or tensile resistance of piles or stability of slopes are derived based on vane test results, an analytical design method shall be used.

(2)P In order to obtain derived values for the undrained shear strength from field vane test results, the test result (c_{fv}) shall be corrected based on:

$$c_u = \mu \times c_{fv}.$$

(4.4)

The correction factor μ shall be determined based on local experience.

(3) Existing correction factors are usually related to the liquid limit plasticity index, the effective vertical stress or the degree of consolidation.

NOTE   Annex I gives examples of such correction factors.

4.10 Flat dilatometer test (DMT)

4.10.1 Objectives

(1) The objectives of the flat dilatometer test are the determination of the in-situ strength and deformation properties of soil by expanding a thin circular steel membrane mounted flush on one face of a blade-shaped steel probe inserted vertically into the ground.

(2)P The test consists of measuring the pressures when the membrane is flush with the blade and just begins to move and when the displacement in the centre of the membrane reaches 1,10 mm into the soil. The test shall be performed at selected depths or in a semi-continuous manner.

(3) The results of DMT tests may be used to obtain information on soil stratigraphy, in-situ state of stress, deformation properties and shear strength.

(4) The DMT test should primarily be used in clays, silts and sands where particles are small compared to the size of the membrane.

NOTE   Further information on a procedure, presentation and evaluation of the DMT can be found in CEN ISO/TS 22476-11, (see X.3.7).

4.10.2 Specific requirements

(1) The tests should be carried out and reported in accordance with a recognised method.

(2)P Any deviations from the requirements given in the method referred to in (1) shall be justified and in particular their influence on the results shall be commented upon.

NOTE   Further information on a procedure, presentation and evaluation of the DMT can be found in CEN ISO/TS 22476-11

4.10.3 Evaluation of test results

(1)P The requirements given in 4.2, shall be followed for the evaluation of the test results.

(2) In addition the field and test reports, according to the method referred to in 4.10.2 (1), should be used for evaluation purposes.

NOTE   Further information on a procedure, presentation and evaluation of the DMT can be found in CEN ISO/TS 22476-11

4.10.4 Use of test results and derived values

4.10.4.1 Bearing resistance and settlement of spread foundations

(1)P If the bearing resistance of spread foundations is derived from DMT results, an analytical design method shall be used.

(2) If an analytical method is used, the derived value of the undrained shear strength (c_u) of non-cemented clays, for which the DMT test results show material index $I_{DMT} < 0,8$, may be determined using the following relationship:

$$c_u = 0,22 \; \sigma'_{v0} \times (0,5 \; K_{DMT})^{1,25} \tag{4.5}$$

where
K_{DMT} is the horizontal stress index or any other well documented relationship based on local experience.

NOTE   Examples of analytical methods are given in EN 1997-1:2004, Annex D.

(3) If applying an adjusted elasticity method, the one-dimensional settlement of spread foundations may be calculated using values of the one-dimensional tangent modulus (E_{oed}) determined from results of DMT tests. In fine soil, such procedures should be applied only when the sum of the effective overburden pressure and the stress increase induced by the foundation load is less than the pre-consolidation pressure.

NOTE 1   Such an adjusted elasticity method is given in EN 1997-1:2004, Annex F.

NOTE 2   An example of such a determination of settlements is shown in Annex J.

4.10.4.2 Pile bearing resistance

(1)P If the ultimate compressive or tensile resistance of piles is derived from DMT results, an analytical calculation method shall be applied to derive the values of base and shaft resistance.

4.11 Plate loading test (PLT)

4.11.1 Objectives

(1) The objective of the plate loading test is the determination of the vertical deformation and strength properties of soil and rock masses in-situ by recording the load and the corresponding settlement when a rigid plate modelling a foundation is loading the ground.

(2)P The plate loading test shall be carried out on a thoroughly levelled and undisturbed surface either at ground level or on the bottom of an excavation at a certain depth or the bottom of a large diameter borehole, an exploration shaft or gallery.

(3) The test is applied in all soils, fills and rock but normally should not be used for very soft fine soil.

4.11.2 Specific requirements

(1)P The test shall be carried out and reported in accordance with EN ISO 22476-13.

(2)P Any deviation from the requirements given in EN ISO 22476-13 shall be justified and in particular its influence on the results shall be commented upon.

NOTE   Deviations exist, for example with respect to plate size and test procedure (incremental loading, constant rate of deformation).

4.11.3 Evaluation of test results

(1)P In addition to the requirements given in 4.2, the field and the test reports according to EN ISO 22476-13 shall be used for evaluation purposes.

4.11.4 Use of test results and derived values

(1) The results of a PLT may be used to predict the behaviour of spread foundations.

(2) For deriving geotechnical parameters of a homogeneous layer (for use in indirect design methods), the layer should have a thickness beneath the plate of at least two times the width or diameter of the plate.

(3) Results of a PLT may only be used for direct design methods if:
−   the size of the plate has been chosen considering the width of the planned spread foundation (in which case the observations are transformed directly);
−   a homogeneous layer up to two times the width of the planned spread foundation exists (in which case the results of smaller sized plates – not considering the planned foundation width – are used to transform the results on an empirical basis to the actual foundation size).

(4) If an analytical design method for bearing resistance is used, the undrained shear strength (c_u) may be derived from a PLT conducted at a constant rate of penetration, sufficiently fast to practically preclude any drainage.

NOTE 1   Examples of analytical design methods for bearing resistance are given in EN 1997-1:2004, Annex D.

NOTE 2   An example of a relationship used for deriving the value of c_u is given in K.1.

(5) If an adjusted elasticity method for settlement evaluation is used, the Young's modulus of elasticity (E) may be derived from the plate settlement modulus (E_{PLT}), based on established experience.

NOTE 1   Such an adjusted elasticity method for settlement evaluation is given in EN 1997-1:2004, Annex F.

NOTE 2   The determination of E_{PLT} is shown in K.2.

(6) The coefficient of sub-grade reaction (k_s) for evaluating deformations may be derived from results of an incremental loading test.

NOTE   An example of the calculation of k_s is given in K.3.

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(7) For direct design, the results of PLT may be transferred directly to the foundation problem without using any geotechnical parameters.

(8) Settlements of footings in sand may be derived from PLT results.

NOTE   An example is given in K.4.

Section 5   Laboratory tests on soil and rock

5.1 General

(1)P The laboratory test program shall be established in conjunction with the other parts of the ground investigation program (see Section 2 for more details).

(2) Whenever possible, the information obtained from field tests and soundings should be used for selecting the test samples (see 2.4.1.3).

5.2 General requirements for laboratory tests

5.2.1.General requirements

(1) The requirements given in this section should be considered a minimum.

(2) Additional specifications, additional presentation requirements or additional interpretation, as appropriate for the ground conditions or geotechnical aspects of interest, may be required.

(3)P Details of the tests required to determine the parameters needed for design shall be specified.

5.2.2 Procedures, equipment and presentation

(1)P Tests shall be carried out and reported according to existing EN and EN ISO documents.

NOTE   CEN ISO/TS documents are available for a number of laboratory tests. Some EN ISO documents are under preparation..

(2) Provided the requirements of this standard are met, alternative test methods and procedures may be selected.

(3)P Checks shall be made that the laboratory equipment used is adequate, fit for its purpose, calibrated and within the calibration requirements.

(4) The reliability of the equipment and procedures should be checked by comparing the test results with data obtained on comparable soil or rock types.

(5)P The test methods and procedures used shall be reported together with the test results. Any deviations from a standard test procedure shall be reported and justified.

(6) If appropriate, the results of laboratory soil classification tests should be presented together with the soil profile on a plot summarizing the soil description and all classification results.

(7) If possible and required, the location of the other laboratory tests (such as oedometer and triaxial tests) should be indicated on the same plot.

5.2.3 Evaluation of test results

(1) Requirements for evaluation of laboratory test results are given in 6.3.

(2) Results of individual tests should be compared with other test results to check that no contradiction exists between the available data.

(3) The test results should be checked with values found in the literature, correlations with index properties and comparable experience.

5.3 Preparation of soil specimens for testing

5.3.1 Objective

(1) The objective of the preparation of soil for laboratory tests is to provide test specimens that are as representative as possible of the soil from which the samples are taken.

(2) For the purposes of preparation, five types of soil specimens may be distinguished: disturbed, undisturbed, re-compacted, remoulded and reconstituted specimens.

5.3.2 Requirements

5.3.2.1 Quantity of soil

(1)P The soil specimen used for testing shall be sufficiently large to take account of:
− the largest size of particles present in significant quantity;
− the natural features such as structure and fabric (e.g. discontinuities).

NOTE   Minimum masses of disturbed soil for classification tests and tests on re-compacted specimens and masses of soil required for preparation of undisturbed specimens for strength and compressibility tests are given in Annex L.

5.3.2.2 Handling and processing

(1)P The requirements of EN-ISO 22475-1 shall be observed.

(2)P All samples shall be clearly and unambiguously labelled.

(3)P Soil samples shall be protected at all times against damage, deterioration and excessive changes in temperature. Special care shall be taken with undisturbed samples to prevent distortion and loss of water during the preparation of test specimens. The material used for the sampling containers shall not react with the contained soil.

(4)P Soil shall not be allowed to dry before testing if the test results can be affected by a loss of moisture.

(5) Undisturbed samples should be prepared under conditions of controlled humidity. If preparation is interrupted, the specimen should be protected from changes in water content.

(6)P If disaggregating processes are applied, the breaking down of individual particles shall be avoided. If special treatment of bonded and cemented soil is required, this shall be specified.

(7)P Subdivision methods shall ensure that representative portions are obtained, avoiding segregation of large particles.

5.4 Preparation of rock specimens for testing

5.4.1 Objective

(1) The objective of preparing specimens for testing rock is to provide specimens that are as representative as possible of a rock formation.

NOTE   Annexes T to W and X.2 provide more detail on the preparation of rock specimens for testing and some guidelines.

5.4.2 Requirements

(1)P It shall be specified how a rock specimen is prepared. If these specifications cannot be met, the specimen shall be prepared as near to the specifications as possible and it shall be reported how the specimen has been prepared.

(2)P All instruments and assemblies for determining straightness, flatness and perpendicularity of end surfaces shall be controlled on a registered regular time basis with tolerances satisfying at least the requirements of the specific rock tests.

(3)P The following shall be specified:
− the storage conditions for rock samples (short term and/or long term storage);
− the moisture condition of the test specimens at the time of the test;
− the method for preparing rock core specimens;
− the method for determining dimension and shape tolerances.

⟨AC₁⟩ (4)P Any change in water content should be avoided. If a change in the natural water content should occur, its effect shall be counteracted as part of the preparation for testing, if relevant. ⟨AC₁⟩

(5) The cause and effect of any change in water content should be reported.

(6)P The need for re-coring to a specified dimension shall be defined with reference to the laboratory coring method, coolant applied and the need for re-saturation of the test specimens.

(7) Together with the data and results for the particular test, the following should be recorded and reported:
− the source of test specimen, including depth/level and orientation in space;
− the dates of specimen preparation and testing;
− comments on the representativeness of the specimen(s) tested;
− all dimension and shape measurements, including conformity to requirements;
− the water content of the sample/specimen (as received, during preparation, saturated);
− the conditions for drying (air- or oven-drying, pressurised or partial vacuum).

(8) The following information on the samples should be given for the interpretation of the test results:
− a physical description of the specimen including rock type (such as sandstone, limestone, granite, etc.), location and orientation of inherent rock structural features and any discontinuities, and inclusions or non-homogeneities;
− a sketch of the test specimen or a colour photograph for other than monotonous homogeneous rock types;
− a Core Recovery and Rock Quality Designation, where possible;

– data to substantiate the tolerance checks on deviations of the right cylindrical form of the test specimen, from the flatness of the end bearing surfaces and perpendicularity of the end surfaces with respect to the axis of the core.

5.5 Tests for classification, identification and description of soil

5.5.1 General

(1)P Classification, identification and description of soil shall be made in accordance with EN ISO 14688-1 and EN ISO14688-2.

NOTE   Annex M provides more details on individual classification tests and their interpretation, and a guideline for the minimum number of samples and tests in one stratum.

5.5.2 Requirements for all classification tests

(1) For all classification tests, special caution should be exercised when choosing the temperature for oven-drying, as too high a temperature can have detrimental effects on the value measured.

5.5.3 Water content determination

5.5.3.1 Objective and requirements

(1) The objective of the test is to determine the water content of a soil material. The water content is defined as the ratio of the mass of free water to the mass of dry soil.

(2) Soil specimens for measuring the water content should be at least of Quality Class 3, according to 3.4.

(3) If a sample contains more than one soil type, water contents should be determined on the specimens representing the different soil types.

NOTE   Further information on a procedure, presentation and evaluation for the determination of water content can be found in CEN ISO/TS 17892-1, (see X.4.1.2).

5.5.3.2 Evaluation of test results

(1)P When evaluating the results, the presence of significant amounts of gypsum, highly organic soil, materials in which the pore water contains dissolved solids and soil with closed pores filled with water, shall be accounted for, if relevant.

(2) The extent to which the water content measured in the laboratory on the soil "as received" is representative of the "in situ" value should be checked. The effects of the sampling method, transport and handling, specimen preparation method and laboratory environment, should be taken into account in this assessment.

(3) For the soil referred to in (1)P, a drying temperature of approximately 50 °C may be more appropriate than the usually prescribed (105 ± 5) °C, but the results obtained should be considered with caution.

5.5.4 Bulk density determination

5.5.4.1 Objective and requirements

(1) The test is used to determine the bulk (total) mass density of a soil, including any liquid or gas contained.

(2) The test specimens should be at least of Quality Class 2, according to 3.4.

(3)P The test method to be used shall be specified.

NOTE   Further information on a procedure, presentation and evaluation of bulk density determination can be found in CEN ISO/TS 17892-2, (see X.4.1.3).

5.5.4.2 Evaluation and use of test results

(1) The evaluation of the test results should consider the possible sample disturbance.

(2) Except in the case of special sampling methods, the laboratory determination of the density of a coarse soil is generally only approximate.

(3) The bulk density can be used in establishing design values of actions derived from soil and in processing results of other laboratory tests.

(4) The bulk density can also be used in evaluating other soil characteristics. For example, in conjunction with the water content, in computing the density of dry soil.

5.5.5 Particle density determination

5.5.5.1 Objective and requirements

(1) The aim of the test is to determine by a conventional method the density of solid soil particles.

(2)P The choice of the test method to be used shall take the soil type into account.

NOTE   Further information on a procedure, presentation and evaluation of determination of particle density can be found in CEN ISO/TS 17892-3, (see X.4.1.4).

5.5.5.2 Evaluation of test results

(1)P If for a particular stratum, the measured values of the particle density are not within the normally expected range of (2 500 to 2 800) kg/m^3, the mineralogy of the soil, its organic matter and its geological origin shall be checked.

5.5.6 Particle size analysis

5.5.6.1 Objective and requirements

(1) The objective of the test is to determine the mass percentage of individual particle size ranges found in the soil.

NOTE   Further information on a procedure, presentation and evaluation of particle size analysis can be found in CEN ISO/TS 17892-4, (see X.4.1.5).

(2)P Two methods shall be used for particle size analysis, according to the size of the particles:
−   the sieve method for particles > 63 μm (or closest sieve available);
−   the sedimentation method using a hydrometer, or pipette, for particles ≤ 63 μm (or closest sieve available).

(3) Equivalent methods may be used, provided that they are calibrated against the two methods mentioned in (2)P.

(4)P Prior to sedimentation, the specimens of fine soil shall not be dried.

(5) Procedures for the removal of organics, salts and carbonates prior to sieving and sedimentation or for corrections to account for the presence of carbonates, salts and organic material should be used, if appropriate.

NOTE   Carbonates and organic matter can have a cementing or coagulating effect and influence the particle size distribution.

(6) It should be taken into account that for some soils, for example chalky soil, treatment for carbonate removal is unsuitable.

5.5.6.2 Evaluation and use of test results

(1)P The report shall mention the following:
−   the drying method used;
−   whether organics, salts and carbonates have been removed and by which method;
−   the carbonate and/or organic content, if relevant;
−   whether the mass fractions are reported with respect to the total mass (including carbonate and organic matter).

(2) The particle size such that *n* % of the particles by weight are smaller than that size can be denoted D_n. The particle sizes D_{10}, D_{30} and D_{60}, can be used to determine the coefficient of uniformity and coefficient of curvature.

(3) The particle sizes D_{15} and D_{85} can be used in filter criteria for soil.

5.5.7 Consistency limits determination

5.5.7.1 Objective and requirements

(1) The consistency limits (Atterberg limits) comprise the liquid limit, plastic limit and shrinkage limit. Only the determination of the liquid limit and the plastic limit are covered.

(2) The consistency limits are used to characterise the behaviour of clays and silty soil when the water content is changing. Classification of clays and silty soil is mainly based on the consistency limits.

(3)P The testing method to be used for determining the liquid limit (fall cone or Casagrande apparatus) shall be specified.

(4) In general for the liquid limit, the fall cone method should be preferred to the Casagrande method. The fall cone method gives more reliable results particularly for low plasticity soil.

(5) The specimens should at least be of Quality Class 4, according to 3.4, if the test results are supposed to characterise the soil in situ.

NOTE   Further information on a procedure, presentation and evaluation of the determination of consistency limits can be found in CEN ISO/TS 17892-12, (see X.4.1.6).

5.5.7.2 Evaluation and use of results

(1) Different geotechnical properties, for example compressibility or optimum water content, can be derived from correlations with the liquid or plastic limits.

(2) The value of the plasticity index I_P can be computed from liquid and plastic limits. I_P can be used in soil classification and in correlations with some geotechnical soil properties, for example with soil strength.

(3) The value of the consistency index I_C (or the liquidity index I_L) can be computed from liquid and plasticity limits and from the current water content of the soil. It can be used to represent soil consistency and in correlations with some geotechnical properties.

(4) The activity index I_A can be computed from I_P and the percentage of clay particles. (I_A) can be used in soil classification and in correlations with different geotechnical soil properties, for example with soil strength.

5.5.8 Determination of the density index of granular soil

5.5.8.1 Objective and requirements

(1) The density index relates the void ratio of a soil sample to reference values determined by standard laboratory procedures. It gives an indication of the state of compaction of a free draining granular soil.

(2)P The following shall be specified or checked:
−   the quantity and quality of samples;
−   the type of testing procedure to be applied;
−   the method of preparation of each test specimen.

(3) The tested soil should contain less than 10 % of fines (particles passing through the 0,063 mm sieve) and less than 10 % of gravel (particles retained on the 2 mm sieve).

(4)P Density index test results shall be reported together with the available particle size analysis results, natural water content, particle density and percentage of oversize fraction (the latter if applicable). Any deviation with respect to (3) shall be reported.

NOTE   Further information on a procedure, presentation and evaluation the determination of the density index can be found in X.4.1.7.

5.5.8.2 Evaluation and use of test results

(1) When evaluating density indexes, it should be taken into account that the maximum and minimum densities obtained in the laboratory do not necessarily represent limiting densities. It is also generally recognised that these tests give densities with a high degree of variability.

(2) The density index can be used to characterise the shear strength and the compressibility of coarse soil.

5.5.9 Soil dispersibility determination

5.5.9.1 Objective

(1) The objective of the test is to identify the dispersive characteristics of clayey soil. Standard tests for classifying soil for engineering purposes do not identify the dispersive characteristics of a soil. Tests for dispersibility are carried out on clayey soil, primarily in connection with earth embankments, mineral sealings and other geotechnical structures in contact with water.

(2) Four test types are considered, (see M.7):
− the pinhole test, which models the action of water flowing along a crack;
− the double hydrometer test, which compares the dispersion of clay particles in plain water without mechanical stirring with that obtained using a dispersant solution and mechanical stirring;
− the crumb test, which shows the behaviour of crumbs of soil placed in a dilute solution of sodium hydroxide;
− the determination of soluble salts in the pore water, which allows the correlation of the percentage of sodium to the total dissolved salts in a saturation extract.

5.5.9.2 Requirements

(1)P The following shall be specified:
− the storage of samples such that the samples are not allowed to dry before testing;
− the testing procedures to be applied;
− the specimen preparation method.

(2)P The results from the dispersibility tests shall be linked to the grain size distribution and consistency limits of the sample.

(3) For the pinhole test, the compaction conditions of the soil specimens, for example wet or dry of optimum, and the mixing water (e.g. distilled versus reservoir water) should be specified.

(4) For the double hydrometer test, a third hydrometer test may be specified if it appears necessary to study the effect of reservoir water on the soil in suspension.

(5) For the crumb test, the use of distilled water may be requested in addition to the sodium hydroxide solution.

5.5.10 Frost susceptibility

5.5.10.1 Objective

(1) The frost susceptibility of soil materials plays an essential role in the design of foundations placed above the freezing front in frost susceptible soil.

(2) Roads, airport runways, railways, buildings on spread foundations, buried pipelines, dams and other structures can be subject to frost heave due to freezing of a frost-susceptible soil having access to water. Frost-susceptible soil can be used in its natural state or as a constructed base for structures.

(3) The risk of frost heaving may be estimated from correlation with soil classification properties (particle size distribution, height of capillary rise and/or fines content) or from laboratory tests on natural, re-compacted and re-consolidated, or reconstituted samples.

NOTE   An example can be found in M.8 and X.5.

5.5.10.2 Requirements

(1) If the estimation of frost susceptibility based on classification properties of the soil does not clearly indicate the absence of risk of frost heaving, frost heaving tests in the laboratory should be run. Examples of soil types indicating the need of laboratory tests in addition to correlations to classification properties include organic soil, peat, saline soil, artificial soil and coarse soil with a wide range of grain size.

(2) To determine the frost susceptibility of a soil in its natural state, natural samples should be tested. To estimate the frost susceptibility of a constructed fill, frost heave tests should be run on re-compacted and then re-consolidated specimens or on reconstituted specimens.

(3) The frost susceptibility test in the laboratory is a frost heave test. If the risk of thaw weakening is to be tested, a California Bearing Ratio test should be carried out after thawing of the specimen. The re-compacted or reconstituted specimen should be subjected to one or more freeze-thaw cycles before testing.

5.5.10.3 Evaluation of test results

(1) The results should be interpreted as a function of the type of construction work, the rules used in design and the available comparable experience, considering the consequence of the frost effects.

5.6 Chemical testing of soil and groundwater

5.6.1 Requirements for all chemical tests

5.6.1.1 Scope

(1) Although the detailed chemical composition of soil is often of limited interest for civil engineering purposes, the presence of certain chemical constituents in soil can be very significant, for example for the durability of the geotechnical structure.

(2) Routine chemical testing in a soil laboratory is usually limited to organic content (loss on ignition, total organic content, organic matter), carbonate content, sulfate content, pH value (acidity or alkalinity) and chloride content. This standard deals with these five chemical tests only.

NOTE 1   Annex N provides more details on each chemical test and its interpretation, and some guidelines.

NOTE 2   There are other chemical components that may cause an environment very aggressive to steel and concrete, for example sulfide, magnesium and ammonium. The corresponding chemical testing is not covered in this standard.

NOTE 3   Corrosiveness to steel constructions in soil is usually evaluated by means of electrical resistivity tests and determination of the redox potential (not covered in this standard), pH, chlorides and sulfate determinations.

5.6.1.2.Objective

(1) The purpose of the chemical tests described herein is to classify the soil and to assess the detrimental effect of the soil and groundwater on concrete, steel and the soil itself. The tests are not intended for environmentally related purposes.

5.6.1.3 Requirements

(1)P The following requirements shall be specified for all chemical tests:
− the samples to be tested;
− the number of samples to be tested;
− the test procedures to be applied;
− the pre-treatments including treatment of oversize particles (i.e. $D > 2$ mm);
− the number of tests per stratum and number of duplicate tests;
− the number of separate tests for the determination of a mean value;
− the format of reporting;
− the required supplementary classification tests for each test or series of tests.

(2)P The proper procedures of mixing, riffling and quartering shall be strictly followed in order to avoid inconsistent results.

(3) Disturbed soil samples may be used for the chemical tests, but particle size and water content need to be representative of the field conditions (Quality Classes 1 to 3).

(4) For the determination of organic content, the particle size distribution only needs to be representative (Quality Class 4).

NOTE   Recommended test procedures are available in Annex N.

5.6.1.4 Evaluation of test results

(1)P The test results shall be reviewed together with the geological description and the prevailing environment.

(2)P Where appropriate, account shall be taken of recognised classifications in terms of the parameter measured.

5.6.2 Organic content determination

5.6.2.1 Objective

(1) Organic content tests are used to classify the soil. In soil with little or no clay particles and carbonate content, the organic content is often determined from the loss on ignition at a controlled temperature. Other suitable tests can also be used. For example, organic content can be determined from the mass loss on treatment with hydrogen peroxide (H_2O_2), which provides a more specific measure of organics.

(2) The presence of organic matter can have undesirable effects on the engineering behaviour of soil. For example, the bearing capacity is reduced, the compressibility is increased, swelling and shrinkage potential is increased due to organic content. Gas can lead to large immediate settlements and can affect the consolidation coefficients and shear strength derived from laboratory tests. Organic matter is detrimental for the stabilisation of soil used for roads and is usually associated with low pH and at times with presence of sulfates which can have negative effects on foundations.

5.6.2.2.Requirements

(1)P For each test or series of tests, in addition to the list in 5.6.1.3, the following shall be specified:
− the drying temperature;
− the ignition temperature;
− the required corrections for bound water, carbonates, etc.;
− the factor used for converting carbon content into organic content.

(2)P Non-homogeneous samples require larger specimens and the appropriate apparatus. Correspondingly larger crucibles shall be used.

(3)P The loss on ignition shall be reported as a percentage of original dry matter, also giving the drying temperature, ignition temperature and drying and ignition times.

(4)P The organic content shall be reported as a percentage of original dry matter, also giving the method of determination.

5.6.2.3 Evaluation of test results

(1) In clays and silty soil with moderate organic content, the errors involved in the correction for bound water or carbonates can be so large that special testing methods are necessary.

5.6.3 Carbonate content determination

5.6.3.1 Objective

(1) The carbonate content is used as an index to classify natural carbonate soil and rock or as an index to indicate the degree of cementation.

(2) Measurement of the carbonate content depends on the reaction with hydrochloric acid (HCl) which liberates carbon dioxide. It is usually assumed that the only carbonate present is calcium carbonate ($CaCO_3$). The carbonate content is calculated from the content of carbon dioxide measured on treatment of the soil with HCl.

5.6.3.2 Requirements

(1)P The soil shall be assessed visually before the selection of the appropriate pre-treatment.

(2) When appropriate, large initial samples can be used to cope with non-homogeneous carbonate distribution in soil and rock. Representative test samples can be established by crushing and riffling.

(3)P The carbonate content shall be reported as a percentage of the original dry matter.

5.6.3.3 Evaluation of tests results

(1)P Some carbonates, e.g. dolomite, need not dissolve using the standard solution of hydrochloric acid during the specified time. Special methods shall be used for soil or rock types containing such carbonates.

5.6.4 Sulfate content determination

5.6.4.1 Objective

(1) The objective of the test is to determine the sulfate content as an index for the possible detrimental effect of the soil on steel and concrete. All naturally occurring sulfates, with rare exceptions, are soluble in hydrochloric acid. Some are soluble in water.

(2) The acid-soluble sulfate content is referred to as the total sulfate content, as distinct from the water-soluble sulfate content. It is important to appreciate which value is relevant.

(3) Groundwater containing dissolved sulfates, especially sodium and magnesium sulfates, can attack concrete and other materials placed in the ground or on the ground surface. Classification of soil and groundwater in terms of sulfate content is therefore necessary so that suitable precautionary measures can be taken, if required.

5.6.4.2 Requirements

(1)P It shall be specified for each test or group of tests, whether acid- or water-soluble sulfate is required for the test, in addition to the items listed in 5.6.1.3.

(2)P Non-homogeneous soil containing visible crystals of gypsum require large samples, which shall be crushed, mixed and riffled to provide representative test specimens. A visual assessment is needed before selecting the appropriate specimen preparation method.

5.6.4.3 Evaluation of test results

(1)P The content in SO_3^{2-} or SO_4^{2-} shall be reported as a percentage of dry substance or in grams per litre, relating to acid- or water-soluble sulfate.

5.6.5 pH value determination (acidity and alkalinity)

5.6.5.1 Objective

(1) The pH value of groundwater or solution soil in water is used to assess the possibility of excessive acidity or alkalinity.

5.6.5.2 Requirements

(1)P The following shall be specified for each test or group of tests, in addition to the general requirements for chemical testing:
−   whether or not the soil shall be dried;
−   the ratio of soil to water.

(2)P Standard buffer solutions shall be used for calibration of the pH meter.

(3)P The pH value of the soil suspensions or the groundwater shall be reported. The test method shall be stated.

5.6.5.3 Evaluation of test results

(1) The evaluation should consider that, in some soil, the measured values can be influenced by oxidation.

5.6.6 Chloride content determination

5.6.6.1 Objective

(1) The objective of the test is the determination of the water-soluble or acid-soluble chloride content so that the salinity of the pore water or soil can be assessed. The results provide an index for the possible effect of the groundwater towards concrete, steel, other materials and soil.

5.6.6.2 Requirements

(1)P The following shall be specified for each test or group of tests, in addition to the items listed in 5.6.1.3:
−   whether water-soluble or acid-soluble chlorides shall be determined;
−   whether or not the soil shall be dried.

(2)P After drying, the soil shall be mixed thoroughly to redistribute any salts which may have migrated to form a surface crust.

5.6.6.3 Evaluation of test results

(1)P The chloride content in grams per litre or as a percentage by dry mass of the soil shall be reported. The test procedures used shall state whether water-soluble or acid-soluble chlorides have been determined.

5.7 Strength index testing of soil

5.7.1 Objective

(1) The purpose of strength index tests is to determine in a rapid and simple manner the undrained shear strength c_u of clayey soil.

(2) This standard covers the following strength index tests:
– laboratory vane test;
– fall cone test.

5.7.2 Requirements

(1)P The tests shall be performed on undisturbed specimens of quality class 1.

NOTE 1 Further information on a procedure, presentation and evaluation for strength index testing can be found in CEN ISO/TS 17892-6, (see X.4.3).

NOTE 2 Annex O gives information on the strength index tests considered and a checklist on the test procedures for the tests.

5.7..3 Use of test results

(1) It should be considered that the values of c_u represent the undrained shear strength of the soil at its state in the laboratory. They need not represent the undrained strength of the soil in-situ.

(2) Depending on the characteristics of the soil and on the particular strength index test selected, the test results can be only an approximate estimate of c_u.

(3) Strength index tests should only be used for design where well–documented, comparable experience with similar soil exists.

(4) If well–documented, comparable evidence exists, the undrained shear strength derived from the strength index tests may be used if the sample analytical method of EN 1997-1:2004, D.3 is applied.

(5) The test results may be used to check variability of undrained shear strength within a stratum.

5.8 Strength testing of soil

5.8.1 Objective and scope

(1) The objective of the test is to establish the drained and/or undrained shear strength parameters.

(2) The following strength tests are covered:

− unconfined compression test;
− unconsolidated undrained triaxial compression test;
− consolidated triaxial compression test;
− translational shear box test and torsional shear box (ring shear) test.

(3) Translational and ring shear box tests should be used for testing soil under drained conditions.

NOTE   Under quick strain rate, clays of very low permeability may sometimes be considered loaded undrained in both types of the shear box apparatus. The test then gives an indication of the undrained strength.

(4) In this section, only strength tests on fully saturated or dry soil are dealt with.

5.8.2 General requirements

(1) For the determination of the shear strength of clay, silt and organic soil, undisturbed samples (Quality Class 1) should be used. For certain soil or special purposes, tests can be carried out on reconstituted or remoulded specimens.

(2) For coarse silts and sands, the test specimens may be re-compacted or reconstituted. Care should be taken to select a preparation method that reproduces as closely as possible the structure and density relevant for the design at hand.

(3)P For re-compacted or reconstituted specimens, the composition, density and water content of the prepared specimens relevant for the in-situ conditions and the specimen preparation method shall be specified.

(4)P For a strength test, the following shall be evaluated or specified:
− the number of tests required;
− the selection of location of test specimens in the recovered samples;
− the required quality of sample;
− the specimen preparation method;
− the orientation of specimen;
− the type of test;
− the classification tests that need to be done;
− the consolidation stresses (if applicable);
− the time for consolidation increments (if applicable);
− the shearing rate;
− the failure criteria;
− the criteria for terminating tests (e.g., strain at which the test shall be stopped);
− the acceptability criteria (e.g. saturation, scatter);
− the accuracy of measurements;
− the format for presentation of test results;
− any procedure used in addition to those referenced in an accepted standard.

(5) The shear strength of a sample should be determined by a set of three or more tests under different normal stresses.

(6)P When determining the shear strength of a soil stratum, the following shall be taken into account:

– the type of shearing;
– the specimen preparation method;
– the need for additional classification tests.

(7) If samples of Quality Class 2 are tested, the effects of the sample disturbance should be considered in the interpretation of the results.

NOTE   Annex P provides a guideline for the minimum number of samples and tests for one stratum, as well as additional information on the test and its evaluation.

5.8.3 Evaluation and use of test results

(1)P The presentation of the test results shall include, where applicable:
– the effective stress paths;
– the Mohr circles;
– the stress-strain curves;
– the pore pressure-strain curves;
– the pore pressure parameters.

NOTE   Linear extrapolation of the test results can give erroneous values of the strength of the soil as the failure envelope is generally not a straight line, especially at low normal stresses.

(2)P The stress range over which the strength parameters have been determined shall be given.

(3) There are several methods to obtain the stress-strain and strength parameters of soil in the laboratory and in-situ. If appropriate, the results from these different tests should be compared when evaluating the test results.

(4) The results should be evaluated taking into account the strain rate used for testing.

(5) The compression and direct shear tests provide commonly accepted strength parameters which may be applied to routine design methods, but which are not necessarily applicable to other analyses.

(6) It should be considered that unconfined compression and unconsolidated, undrained compression tests need not represent the undrained strength of the soil in-situ.

5.8.4 Unconfined compression test

5.8.4.1 Requirements

(1) The unconfined compression test should be carried out on specimens of soil of sufficiently low permeability to maintain undrained conditions during the test.

(2) Delays between trimming and testing should be avoided to prevent change of water content of specimens.

NOTE   Further information on a procedure, presentation and evaluation for the unconfined compression test can be found in CEN ISO/TS 17892-7, (see X.4.4.1).

5.8.4.2 Evaluation and use of test results

(1) The test result is the approximate value of the unconfined compressive strength of the tested soil.

(2) The undrained shear strength c_u may be determined as one half of the measured unconfined compressive strength.

(3) The effective stresses within the laboratory specimen may deviate from the effective stresses in-situ. The test result need not represent the undrained strength of the soil in-situ due to this difference.

5.8.5 Unconsolidated, undrained triaxial compression test

5.8.5.1 Requirements

(1)P The test shall be carried out without allowing any drainage from the specimens.

(2)P During preparation and testing, soil specimens shall not be given access to water (e.g. from the drainage leads or pore pressure transducers, etc.).

(3)P Water content before and after the test and bulk density before the test shall be determined for each specimen.

NOTE   Further information on a procedure, presentation and evaluation for the unconsolidated undrained triaxial compression test can be found in CEN ISO/TS 17892-8, (see X.4.4.2).

5.8.5.2 Evaluation and use of test results

(1) The test result is the undrained shear strength c_u of the tested soil.

(2) The effective stresses within the laboratory specimen may deviate from the effective stresses in-situ. The test result need not represent the undrained strength of the soil in-situ due to this difference.

5.8.6 Consolidated triaxial compression test

5.8.6.1 Requirements

(1)P The tests shall be performed on undisturbed specimens of quality class 1.

NOTE   Further information on a procedure, presentation and evaluation of the consolidated triaxial compression test can be found in CEN ISO/TS 17892-9, (see X.4.4.3).

(2)P For a consolidated triaxial compression test, the following shall be evaluated or specified further to 5.8.2:
− the saturation method and saturation criterion;
− the back-pressure required;
− any procedure used in addition to those referenced in an accepted standard (e.g. lubricated ends, local measurements of strains or pore pressure).

(3)P For consolidated, undrained triaxial tests, the requirements on measuring pore pressures, and the total stress path for shearing, shall be specified.

(4)P For consolidated drained tests, the volume change measurement device and the stress path for shearing shall be specified.

(5)P Water content before and after the test and bulk density before the test shall be determined for each specimen.

(6) One consistency limit determination and particle size analysis should be made per set of triaxial tests in one stratum.

(7)P The results shall clearly state the type of test that was carried out, which strength parameters are given, the shearing rate and the failure criterion used to select the shear strength (e.g. peak deviator stress, maximum stress ratio).

(8)P The report shall indicate any known deviation from the standard testing procedure, for example in the degree of saturation of the test specimens, testing procedures, composition of the specimen or in any other aspect.

(9) In conformity to 2.4.2.3 (4), more advanced laboratory strength tests, such as triaxial extension tests, simple shear tests, plane strain compression and extension tests, true triaxial tests, directional shear tests, all with the possibility of anisotropic instead of isotropic consolidation, should be considered.

5.8.6.2 Evaluation and use of test results

(1) The evaluation of the test results should take into account that undrained shear strength, pore pressure parameters and stress-strain relationships are affected by sample disturbance to a greater extent than the drained strength parameters.

NOTE   Reliable stress-strain modulus values, especially in the case of stiff soil, can only be obtained from advanced tests with special procedures to measure strains and axial force accurately (see 5.9).

(2) Depending on the type of test, drained or undrained strength of the soil may be derived. Accordingly, the values are effective angle of shearing resistance (φ') and effective cohesion (c'), or undrained shear strength (c_u).

(3) The values can be used in both drained and undrained stability analyses

NOTE   See EN 1997-1:2004, Annex D.

5.8.7 Consolidated direct shear box tests

5.8.7.1 Requirements

(1)P The tests shall be performed on undisturbed specimens of Quality Class 1.

NOTE   Further information on a procedure, presentation and evaluation of the consolidated direct shear box test can be found in CEN ISO/TS 17892-10, (see X.4.4.4).

(2)P The location and orientation of the specimen shall be considered carefully so as to duplicate as closely as possible the in-situ conditions. In the translational shear box and ring shear tests, failure is forced to occur on or near a horizontal plane in the middle of the test specimen.

(3)P Negative or positive pore water pressures due to shear shall be avoided during the test as they cannot be measured and accounted for in the interpretation of the test. To maintain drained conditions, the rate of shearing shall be slow enough so that pore water pressures may dissipate.

5.8.7.2 Establishment and use of values

(1) The results of standard shear box tests represent the strength under drained conditions. The values are effective angle of shearing resistance and effective cohesion (see NOTE in 5.8.1 (3)).

(2) The values can be used in stability analyses.

NOTE   See EN 1997-1:2004, Annex D.

5.9 Compressibility and deformation testing of soil

5.9.1 General

(1) This standard covers the requirements for measurement of deformation characteristics of soil in the triaxial apparatus and the oedometer.

5.9.2 Oedometer compressibility testing

5.9.2.1 Objective

(1) In the oedometer, the cylindrical specimen is confined laterally, and is subjected to discrete increments of vertical axial loading or unloading and allowed to drain axially. Oedometer compression and swelling tests and evaluation of the collapse potential of a soil are covered.

(2) Alternatively, a test with continuous loading (constant rate of strain) may be carried out.

(3) The objective of the oedometer incremental compression and swelling tests is to determine the compression, consolidation and swelling characteristics of soil.

(4) The objective of the collapse potential test is to establish compressibility parameters for the soil in its unsaturated state, and to evaluate the additional compression upon inundation due to structural collapse of the soil.

5.9.2.2 Requirements

(1)P For the determination of the compressibility of a stratum of clay, silt or organic soil, undisturbed samples (Quality Class 1) shall be used.

NOTE   Small strain moduli of soil (e.g. moduli at less than 1 % strain for soft to medium clays) are very sensitive to all perturbations during sampling. According to 3.4.3 (3)P, specific sampling equipment and methods may be used, for example block sampling or stationary piston sampling or any other method known to give the best results for the soil to be tested.

(2)P For re-compacted specimens, the composition, density and water content of the prepared specimens relevant to the in-situ conditions and the specimen preparation method shall be specified.

(3)P When determining the compressibility characteristics of a soil stratum, the following items shall be taken into account:
− existing results of field investigations;
− existing settlement measurements from neighbouring sites;
− the number and quality of samples;
− the number and type of field tests;
− special consideration needed for sensitive and cemented samples;
− the specimen preparation;
− the orientation of the specimen;
− the need for additional classification tests.

(4) Carrying out alternative tests to the incremental oedometer test, for example constant rate of strain oedometer tests, may be considered.

(5)P The initial vertical stress shall not exceed the in-situ vertical effective stress.

NOTE   For example for strain-softening clays, an initial stress of one fourth of the in-situ vertical effective stress can be considered an appropriate value.

(6) In a compression test, the highest vertical stress applied should be well in excess of the maximum effective vertical stress likely to occur in-situ. In a swelling test, the range of decrements of vertical stress to be applied during the test should include the range of stresses likely to be applied in-situ.

(7)P When testing the collapse potential, the test specimens shall be selected with adequate consideration of existing knowledge on the behaviour of the soil when subjected to inundation. The stress on the specimen at which inundation is applied shall be related to the range of vertical stresses likely to occur in-situ.

NOTE 1   Further information on a procedure, presentation and evaluation of Oedometer compressibility testing can be found in CEN ISO/TS 17892-5, (see X.4.5).

NOTE 2   Annex Q provides a guideline for the minimum number of samples and tests for one stratum, as well as additional information on the test and its evaluation.

5.9.2.3 Evaluation and use of test results

(1) The oedometer test results may be used to estimate the yield stress (pre-consolidation pressure) for clays, organic soil and silty soil.

(2) It should be taken into account that the pre-consolidation pressure determined by the oedometer test can be greatly affected by sampling disturbance.

[AC₁] (3) The most common values to characterise compressibility are the oedometer modulus (E_{oed}), the coefficient of compressibility (m_v), the compression index (C_c) and the pre-consolidation pressure (σ'_p). Unloading and recompression can be represented by the swelling index (C_s). All these quantities are derived directly from the appropriate parts of the test curves. [AC₁]

(4) Settlements due to creep can be computed using the coefficient of secondary compression (C_α).

(5) The coefficient of consolidation c_v can be derived using the one-dimensional consolidation theory.

(6) Any parameter in 5.9.2.3 (3) can be used for simple analyses of the settlement of spread foundations.

(7) If sample methods are applied, the oedometer modulus may be used.

NOTE   Examples of such sample methods are given in EN 1997-1:2004, F.1 and F.2

5.9.3 Triaxial deformability testing

5.9.3.1 Objective

(1) The objective of the triaxial deformability testing of soil is to determine the deformation moduli (stiffness parameters).

(2) Depending on the loading path, various stiffnesses can be measured.

(3) According to the drainage conditions, drained or undrained moduli E' or E_u can be determined.

(4) Due to the non-linearity of soil behaviour, various moduli, e.g. tangent and/or secant moduli, may be defined at different stress or strain levels.

5.9.3.2 Requirements

(1)P For the determination of the stiffness of a stratum of a soil, undisturbed samples (Quality Class 1) shall be used.

NOTE   Small strain moduli of soil (e.g. moduli at less than 1 % strain) are very sensitive to all perturbations during sampling. According to 3.4.3 (3)P, specific sampling equipment and methods may be used, for example block sampling or stationary piston sampling or any other method known to give the best results for the soil to be tested.

(2)P A special instrumentation capable of measuring stresses and strains with high resolution shall be used for stiffness determination at strain levels below 0,1 %.

NOTE   For determination of very small strain moduli, the techniques based on propagation of shear waves or other dynamic methods can be used.

(3)P When determining the stiffness characteristics of a soil stratum, the following items shall be considered:
− the quality of samples;
− the sensitivity, saturation, state of consolidation and cementation of soil;
− the specimen preparation;
− the orientation of the specimen.

5.9.3.3 Evaluation and use of test results

(1) The stiffness may be characterised by a complete curve, or by conventional values. For example, by the initial Young's modulus of elasticity (E_0), or by the E_{50} corresponding to 50 % of the maximum shear stress, etc.

(2) Young's modulus and stress-strain curves of soft, normally consolidated soil may in some cases be determined from the standard triaxial strength tests.

5.10 Compaction testing of soil

5.10.1 Scope

(1) This standard covers compaction tests (Proctor tests) and the California Bearing Ratio test.

NOTE   Annex R provides a guideline for the minimum number of samples to be tested for one stratum, as well as additional information on the test and its evaluation.

5.10.2 Compaction tests

5.10.2.1 Objective

(1) Soil compaction tests (Proctor tests) are used to determine the relationship between dry density and water content when a given compaction effort is applied.

5.10.2.2 Requirements

(1)P The following shall be specified or checked:
− the handling of soil with oversize fractions;
− the treatment of stiff finesoil;
− the specimen preparation and maturing;
− the testing procedures and compaction effort to be applied;
− whether the equipment (moulds and rammers) used is as specified in the standard(s).

(2) The option of carrying out in-situ tests instead of laboratory tests should be considered for special types of soil.

5.10.2.3 Evaluation and use of test results

(1)P The compaction characteristics of soil shall be reported together with grain size distribution curves and proportion of oversize material by dry mass with correction, if appropriate.

(2) The optimum water content (w_{opt}) and the corresponding maximum dry density achieved under the applied compaction effort ($\rho_{d,max}$) can be used in evaluating the quality of compacted fills.

5.10.3 California Bearing ratio (CBR) test

5.10.3.1 Objective

(1) The objective of the test is to determine the California Bearing Ratio (CBR) of a compacted or undisturbed sample.

(2) The CBR value is obtained as the percentage of the standard load corresponding to the standard penetration, when a cylindrical plunger of a standard cross-sectional area is made to penetrate into the soil material.

5.10.3.2 Requirements

(1)P The following shall be specified or checked:
− the method of preparation of each test specimen;
− how many tests are to be run in a set of test specimens;
− the handling of soil with oversize fractions ($D > 16$ mm);
− the maturing of specimens;
− whether or not a specimen is to be subjected to soaking;
− if soaking is used, whether swelling is to be measured;
− the surcharge to be applied for soaking and for testing;
− the water content at which compacted specimens are to be prepared;
− the specimen dry density or compactive effort;
− whether equipment (moulds and rammers) used is as specified in standard;
− whether or not the test is to be carried out on one end or both ends of the specimen.

5.10.3.3 Evaluation and use of test results

(1)P The CBR test results shall be reported together with grain size distribution and proportion of oversize material by dry mass, if relevant.

(2) The CBR value can be used as a basic parameter for the design of flexible pavements. It can be used to evaluate the potential strength of sub-grade, sub-base and base course materials (including recycled materials) for supporting road, railways and airfield pavements.

5.11 Permeability testing of soil

5.11.1 Objective

(1) The objective of the test is to establish the coefficient of permeability (hydraulic conductivity) for water flow through water-saturated soil.

5.11.2 Requirements

(1)P When determining the coefficient of permeability of a soil stratum, the following items shall be considered:
− the preferred test type for permeability determination;
− the orientation of the specimen;
− the need for additional classification tests.

NOTE   Further information on a procedure, presentation and evaluation of the permeability test can be found in CEN ISO/TS 17892-11, (see X.4.7).

(2)P Depending on the conditions where the test results will be used, the following shall be specified:

a) In clay, silt and organic soil:

    − the stress conditions under which the specimen is to be tested;

    − the criterion for achieving and maintaining the steady-state flow condition;

    − the direction of flow through the specimen;

    − the hydraulic gradient under which the specimen is to be tested;

    − the need for back-pressure and the required degree of saturation;,

    − the chemistry of percolating fluid.

b) In sand and gravel:

    − the density index to which the specimen is to be prepared;

    − the hydraulic gradient under which the specimen is to be tested;

    − the need for back-pressure and the required degree of saturation.

(3) The hydraulic gradient in the laboratory should be close to that in the field, unless otherwise required due to a particular problem.

(4)P When selecting the hydraulic gradient, it shall be checked that the gradient in the laboratory test and the gradient in-situ lie within the domain of application of Darcy's law.

(5) The report should indicate any known deviation from the standard testing procedure, for example in the degree of saturation of the test specimens, testing procedures, composition of the specimen or in any other aspect.

(6) For permeability tests on clay, silt or organic soil, only soil specimens of Quality Classes 1 or 2 should be used.

(7) For sand and gravel materials, specimens of Quality Class 3 and remoulded or re-compacted soil samples may be used.

(8) It should be checked that the volume changes due to the consolidation of the specimen shall only negligibly affect the measured permeability.

NOTE   Annex S provides a guideline for the minimum number of samples and tests for one stratum, as well as additional information on the test and its evaluation.

5.11.3 Evaluation and use of test results

(1) The evaluation should assess:
−   the extent to which the boundary conditions (degree of saturation, direction of flow, hydraulic gradient, stress conditions, density and layering, side leakage and head loss in filter and tubing) affect the test results;
−   how well these conditions match the situation in the field.

(2) For partially saturated soil, much smaller values may be relevant than the values measured in the case of full saturation.

(3) Due consideration should be given to whether a temperature correction should be applied.

(4) The coefficient of permeability can be computed from the test data under the assumption that Darcy's law is valid.

(5) The coefficient of permeability can be used in design of excavations and earth dams to estimate the amount of seepage, to assess the feasibility of groundwater level management (lowering), to design the sheet pile walls, to estimate seepage pressures, etc.

5.12 Tests for classification of rocks

5.12.1 General

(1) The following tests are included in this standard:
− rock identification and description;
− water content;
− density and porosity.

(2) Classification relates to the division of identified rock into specific types defined for particular civil engineering purposes. The classification is related to mineralogical components, structure, induration, rock density, water content, porosity and rock strength.

NOTE   Annex U provides more details and guidelines on the classification tests.

5.12.2 Requirements for all classification tests

(1)P The classification test results shall be reviewed together, compared with drilling logs, corresponding geophysical logs, photographs of cores and comparable experience.

(2) The soil and rock classification should be compared with available geological background information in order to provide an engineering geological model.

(3) Geological maps should be used as a guide for the classification of rock and rock masses, whenever available.

(4) Second opinion evaluations and the use of typical examples with rock comparisons may be required in order to achieve a consistent description.

5.12.3 Rock identification and description

5.12.3.1 Objective and requirements

(1) The identification and description of rock material and mass are carried out on the basis of mineralogical composition, predominant grain size, genetic group, structure, weathering and other components. The description may be carried out on cores and other samples of natural rock and on rock masses in-situ.

(2)P The laboratory procedure shall conform to EN ISO 14689-1.

(3) More detailed description of rock may be applied. Then the following should be specified:
− the rock classification system;
− the need for advanced geological analyses;
− the format of reporting.

(4) Rock identification and description should be carried out on all samples received in the laboratory, regardless of rock homogeneity, as the identification and description constitutes the framework for all testing and evaluations.

5.12.3.2 Evaluation of results

(1) The classification of the rock mass using cores should be based on the highest possible core recovery to identify discontinuities and possible cavities.

(2) The disturbance of the core from the drilling process should be evaluated since most rock mass quality designations relate to the fractures found in the cores and their quality.

5.12.4 Water content determination

5.12.4.1 Objective and requirements

(1)P With the exception of rocks mentioned in 5.12.4.2 (2), water content of rock shall be determined by oven-drying at (105 ± 5) ℃.

(2)P Measures shall be specified to retain water during sampling and storage, if relevant.

(3)P The following shall be specified:
−   the selection of test samples;
−   the storage in the laboratory before testing;
−   possible re-saturation of desiccated samples using vacuum saturation technique;
−   the number of tests per strata;
−   the number of tests to be run in parallel with other tests from the same formation;
−   the number of accuracy checks to be run.

(4) A minimum sample of at least 50 g or a core piece with a dimension of at least 10 times the maximum grain size of mineral components should be used.

(5)P The report shall state whether the measured water content corresponds to the in-situ water content.

NOTE   As no ISO/CEN standard for rock testing is currently available, the laboratory methods described in U.3, with reference to X.4.9.2 can be applied.

5.12.4.2 Evaluation of test results

(1) The results of the determination of the water content should be compared with the fully-saturated water content as a function of the density (or porosity) of the test specimen. Anomalous results should be investigated by repeated testing.

(2) Rock types with significant amounts of gypsum should be tested at 50 °C as the bound water may partly dehydrate at 105 °C.

(3) For rock types in which the pore water contains dissolved salts or rock types with closed pores, the reported water content should be evaluated in this respect.

(4) The water content should be used for correlation of strength and deformation characteristics of the rock types in boreholes and at test sites.

(5) Comparisons with available correlations of water content and rock type should be made.

5.12.5 Density and porosity determination

5.12.5.1 Objective and requirements

(1) The test is used to determine the bulk and dry density to obtain the porosity and related properties of a rock sample. The bulk and dry density is derived based on weight analyses provided reliable determination of the sample volume is available.

(2) The pore volume may be calculated based on the dry density and the particle density determined using methods as for soil, provided no closed pores exist in the rock specimen. Porosity is the ratio of pore volume to total volume.

(3)P The following shall be specified:
− the selection of test samples;
− the conditions of storage before testing;
− whether desiccated samples are to be re-saturated and by which technique;
− the number of tests required per formation;
− whether parallel tests are to be run on the same formation.

(4) A test specimen of at least 50 g, with a minimum dimension of 10 times the maximum grain size of the mineral components, should be tested.

NOTE   As no ISO/CEN standard for testing of rock is currently available, the laboratory methods described in U.4 with reference to the bibliography in X.4.9.3 can be applied.

5.12.5.2 Evaluation of test results

⟨AC₁⟩ (1) The density and porosity should be integrated in the reporting of rock description and established strength and deformation characteristics of the rock types in boreholes and at test sites. ⟨AC₁⟩

(2) The density and porosity results should be used for comparison of rock strength and deformation properties and to establish correlations for the different rock types.

(3) The existence of closed pores can influence the porosity. The determination of the total pore volume should be based on the density of solids of a powdered sample.

NOTE   Further information on a procedure, presentation and evaluation of density and porosity determination can be found in CEN ISO/TS 17892-3, (see X.4.9.3).

5.13 Swelling testing of rock material

5.13.1 General

(1)This standard deals with the following tests for determination of swelling potential exposed to wetting and drying or unloading in an aqueous environment:
− the swelling pressure index under conditions of zero volume change;
− the swelling strain index for radially-confined specimens with axial surcharge;

− the swelling strain developed in unconfined rock specimens.

NOTE   Some rock materials, notably those with high clay content, are prone to swelling, weakening and disintegration when exposed to wetting and drying or unloading in an aqueous environment. The index tests provide an indication to estimate swelling properties under well-controlled conditions. The tests are usually done on softer rock materials like claystone and shale. The tests can be used for the characterization of harder rocks subjected to weathering.

(2) Rocks that disintegrate during the tests should be further classified using relevant soil classification tests such as the shrinkage, liquid and plastic limits, particle size distribution and the type and content of clay minerals.

NOTE   Annex V provides more details on each of the swelling tests and their interpretation as well as some guidelines.

5.13.2 General requirements

(1) The specimens should conform closely to the practice recommended for right cylinders or for rectangular prisms. The sample size should allow for preparation of test specimens by re-coring and/or machining in a lathe, with the axis for one direction of the swell measurement perpendicular to a bedding or foliation.

NOTE   For recommendations for right cylinders and rectangular prisms, see X.4.8.

(2)P The following shall be specified:
− the selection of test samples;
− the test specimen preparation, orientation and dimensions;
− the numbers of test per formation required;
− the test method, equipment and calibrations;
− the water to be used (natural or distilled water, water chemistry);
− the recording period;
− the need for curves of swell pressure or displacement as a function of time elapsed since flooding;
− the selection of required additional parameters;
− the reporting requirements.

5.13.3 Evaluation of test results

(1)P The results shall be reviewed in the light of the description, and classification parameters shall be established.

(2) The value used in design derived from laboratory tests should be compared with field experience with comparable rock types under similar climatic, loading and wetting conditions.

(3) The short and especially long term weathering processes of swelling, weakening or disintegration due to wetting and drying may only be partly mirrored by the laboratory tests, even for similar conditions of loading and water content, because of the influence of among others natural fissuring, stress, drainage and pore water chemistry.

5.13.4 Swelling pressure index under zero volume change

5.13.4.1 Objective and requirements

(1) The test is intended to measure the pressure necessary to constrain an undisturbed rock specimen at constant volume when immersed in water.

(2) The test may be used to estimate the swell pressure in-situ by comparison of documented experience for the rock stratum.

(3)P The test specimen shall be cored using Category A sampling methods.

NOTE   The test can be carried out following the recommendation given in V.2.

5.13.4.2 Evaluation of test results

(1)P The applied force to maintain the condition of zero volume change shall be corrected for the deformation in the test cell system itself (ball bearings and bedding of filter stones towards the end platens).

(2) The maximum swell pressure under zero volume change should be used as an upper limit of swell pressure under the specified laboratory conditions.

(3) Before the laboratory-determined maximum swell pressure is used in design, field evidence should be taken into consideration related to short and especially long term weathering processes of swelling, weakening or disintegration due to wetting and drying, loading conditions, water content and pore water chemistry.

5.13.5 Swelling strain index for radially-confined specimens with axial surcharge

5.13.5.1 Objective and requirements

(1) The test is intended to measure the axial swelling strain developed against a constant axial surcharge, when a radially confined, undisturbed rock specimen is immersed in water.

(2)P The test specimen shall be cored using Category A sampling methods.

NOTE   The test can be carried out following the recommendation given in V.3.

5.13.5.2 Evaluation of test results

(1)P The measured strain under the applied force during the test stages shall be corrected for the deformation in the test cell system itself (ball bearings and bedding of filter stones towards the end platens).

(2) The axial swelling strain under constant axial surcharge should be used to estimate the swell potential in-situ taking into account the documented experience for the rock stratum.

(3) Depending on the applied vertical stress, the test provides background for the evaluation of the vertical heave or the lateral deformation of a rock/structure interface.

5.13.6 Swelling strain developed in unconfined rock specimen

5.13.6.1 Objective and requirements

(1) The test is intended to measure the swelling strain developed when an unconfined undisturbed rock specimen is immersed in water.

(2)P The test shall only be applied to specimens prepared from at least category B sampling methods that do not change their geometry appreciably during testing.

(3) It is advised that slaking, less durable, rocks should be tested using a confined swelling test.

NOTE   The test may be carried out following the recommendation given in V.4.

(4)P The report shall clearly indicate that the specimen was not radially confined during the swelling test.

5.13.6.2 Evaluation of test results

(1) The test may be used to estimate the swell potential in-situ by comparison to documented experience for the rock stratum.

(2) Unconfined swelling strains and their directions with respect to bedding or foliation should be used only as an estimate of in-situ swelling potential.

5.14 Strength testing of rock material

5.14.1 General

(1) This standard includes five laboratory methods for determining the strength of rock:
−   the uniaxial compression and deformability test;
−   the point load test;
−   the direct shear test;
−   the Brazil test;
−   the triaxial compression test.

NOTE   Annex W provides more detail on each strength test and its interpretation.

5.14.2 Requirements for all strength tests

(1)P The following shall be specified:
−   the samples that are to be tested;
−   the specimen preparation;
−   the number of tests per formation;
−   any required additional parameters;
−   the testing methods.

NOTE   Annex W provides a guideline for the minimum number of test specimens to be tested for one formation for uniaxial compression tests, Brazil tests and triaxial tests for projects of geotechnical category 2 with respect to standard deviations of measured strength and comparable experience.

5.14.3 Evaluation of test results

(1) The evaluation of the test results should include comparison with recognised databases to help screen the data for anomalous results, while accounting for the natural range in compressive strength and deformation parameters in rock, and correlations with the results of classification tests.

(2) All test results should be grouped and analysed with respect to the geological description and classification properties using statistical methods when appropriate.

(3) The values can be used to evaluate in-situ strength and deformation properties and to classify rock element and rock mass properties.

5.14.4 Uniaxial compression and deformability test

5.14.4.1 Objective and requirements

(1) The uniaxial compression test measures the compressive strength, the Young's modulus of elasticity and Poisson's ratio of cylindrical test specimens of rock.

(2) The test is intended for classification and characterisation of intact rock.

(3)P The following shall be specified in addition to the requirements in 5.14.2:
− the specimen orientation and dimensions;
− the testing method;
− if relevant, the definition of modulus (tangent, average or secant) and Poisson's ratio as a function of stress or strain.

(4)P The test specimens shall be prepared from cores taken with Category A sampling.

(5) Recommendations for testing of the uniaxial compression strength and deformability should be followed.

NOTE   Recommendations for such tests are given in Annex W.

5.14.4.2 Evaluation of test results

(1) The uniaxial compression strength should be determined as the maximum vertical stress obtained during the compression test.

(2) The modulus of elasticity defined as the ratio of the axial stress change to axial strain produced by the stress change should be determined using one of the three definitions below:
− the tangent Young's modulus of elasticity measured at a fixed percentage of ultimate strength (i.e. 50 %);
− the mean value of Young's modulus of elasticity from the linear section of the axial stress-strain curve;
− the secant modulus measured from zero stress to some fixed percentage of ultimate strength (i.e. 50 %).

(3) The Poisson's ratio should be determined as the slope of the radial strain versus axial strain curve.

(4) Young's elasticity modulus and Poisson's ratio should be calculated within the same interval of vertical stress.

(5) Test results should be evaluated with respect to rock classification properties and the rupture pattern illustrated by a sketch of the tested rock specimen.

(6) The unconfined compression strength (σ_C) can be used as a classification parameter for the intact rock quality and it could be used in combination with triaxial compression test results in a Mohr diagram to define the Mohr-Coulomb rupture parameters angle of shearing resistance (φ) and cohesion (c).

NOTE   Young's modulus of elasticity E and Poisson's ratio ν may be used for settlement calculation according to EN 1997-1:2004, Annex F.

5.14.5 Point load test

5.14.5.1 Objective and requirements

(1) The point load test is intended as a strength index test for the classification of rock materials. The test results can also be used for estimating the strength of a group of rocks of the same range of competence.

(2) The point load test is not a direct way to measure rock strength but an index test. The correlation between point load test results and strength should be documented in each case.

(3)P In addition to the requirements in 5.14.2 (1)P, the testing methods with reference to cores, blocks and irregular lumps shall be specified.

(4)P The test specimens shall be prepared from cores taken with Category A sampling.

(5) Test specimens of blocks and irregular lumps taken in pits may be used provided this is reported accordingly and the rock specimens are taken using Category B sampling.

(6) Recommendations for the point load test should be followed.

NOTE   Recommendations for such a test are given in W.2.

5.14.5.2 Evaluation of test results

(1)P Because of large variability, the evaluation of rock characterisation and predictions of other strength parameters shall be based on a statistical approach. From test data consisting of at least 10 single tests, the two highest and two lowest values shall be deleted before calculating the mean from the remaining.

(2) In order to classify samples or strata using a mean value of Point Load Strength Index, the minimum number of tests should be five.

(3) The test measures the Point Load Strength Index of rock specimens, and their Strength Anisotropy Index, which is the ratio of the Point Load Strengths in directions which give the greatest and least values.

5.14.6 Direct shear test

5.14.6.1 Objective and requirements

(1) The direct shear test measures peak and residual direct shear strength as a function of the stress normal to the plane of shearing.

(2) This standard deals with the laboratory testing for the determination of the basic shear strength parameters and the surface characteristics of a discontinuity that controls the shear strength.

(3) If the surface characteristics of a discontinuity that controls the shear strength are determined, an accurate description should be made, including type and roughness of the joint, type and thickness of fill material, and the presence of water in the joint.

(4)P The following shall be specified in addition to the requirements in 5.14.2 (1)P:
− the test specimen orientation and dimensions;
− the specifications of the testing machine;
− the rate of shear displacement during test;
− the selection of normal stress to be maintained during the single shear tests.

(5)P The test specimens shall be prepared from cores taken with category A sampling or from blocks taken in a pit using at least Category B sampling.

(6) Recommendations for direct shear tests should be followed.

NOTE   Recommendations for such a test are given in W.3.

5.14.6.2 Evaluation of test results

(1) The evaluation of test results of shear strength versus stress perpendicular to the rupture plane should include a study of the shearing plane in order to take into account bedding and schistosity, cleavage of the rock specimen, the interface properties between rock and concrete, or what was tested.

(2) Shear strength parameters angle of shearing resistance (φ) and cohesion (c) may be established using a number of shear tests on different specimens taken from a rock stratum using Mohr-Coulombs rupture criterion. Alternatively, residual parameters may be found using multiple testing with different normal stresses on an established rupture plane.

(3) The test measures the shear strength in a forced rupture plane under certain stresses perpendicular to the rupture plane. Peak and residual shear strength after some shear deformation can be established. Usually the rupture plan is intentionally established along a known discontinuity.

(4) The test is intended for strength classification and characterization of intact rock and should not be used without geological correlation and rock classification for field conditions.

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5.14.7 Brazil test

5.14.7.1 Objective and requirements

(1) The Brazil test is intended to measure indirectly the uniaxial tensile strength of a cylindrical rock specimen.

(2)P The following shall be specified in addition to the requirements in 5.14.2 (1)P:
− the test specimen orientation and dimensions;
− the testing method.

(3)P Due to the variability of the test results, duplicate testing of test specimens cut in parallel shall be performed.

(4) For shales and other anisotropic rock, it is recommended to cut test specimens parallel to and perpendicular to bedding. For specimens cut parallel to bedding, the direction of the load related to bedding should be specified.

(5)P The test specimens shall be prepared from cores taken with category A sampling.

(6) Recommendations for the Brazil test should be followed.

NOTE   Recommendations for such a test are given in W.4.

5.14.7.2 Evaluation of test results

(1) The evaluation of tensile strength should take into consideration that the presence of hidden weakness planes in the test specimen may disturb the result and the failure plane should be sketched after test and evaluated.

(2) The test provides an indirect determination of the tensile strength σ_T in a forced rupture plane.

(3) The tensile strength (σ_T) may be used as a classification parameter for the intact rock quality and it may be used in a Mohr diagram at a corresponding maximal stress σ_1 together with Mohr circles from uniaxial or triaxial compression tests to define the Mohr-Coulomb strength parameters angle of shearing resistance (φ) and cohesion (c).

(4) The test is intended for strength classification and characterisation of intact rock and the test results should not be used without geological correlation and rock classification for field conditions.

5.14.8 Triaxial compression test

5.14.8.1 Objective and requirements

(1) The triaxial compression test is intended to measure the strength of cylindrical rock specimens subjected to triaxial compression. A number of tests provide the values necessary to determine the strength envelope in a Mohr-Coulomb diagram. From this envelope, the angle of shearing resistance and the cohesion intercept may be determined.

NOTE   No provisions are usually made for drainage of the pore water, nor for the measurement of pore water pressure. In certain rock types (e.g. shales and porous limestone and chalk) and under certain conditions, the pore water pressure may influence the results. For such rock types, advanced triaxial test systems allowing for measuring pore water pressure and volumetric strains are necessary. Such testing may include similar measuring techniques as used for uniaxial compressive strength according to W.1 .

(2)P In addition to the requirements in 5.14.2 (1)P, the test specimen orientation and dimensions, reflecting the testing method shall be specified.

(3)P The test specimens shall be prepared from cores taken with Category A sampling.

(4) Recommendations for triaxial compression testing should be followed.

NOTE   Recommendations for such tests are given in W.5.

5.14.8.2 Evaluation of test results

(1) A triaxial test consists of a series of compression tests carried out under different confining pressures in a triaxial cell. A strength envelope of confining pressures versus axial stress at rupture can be used to establish the Mohr-Coulomb strength parameters angle of shearing resistance (φ) and cohesion (c).

(2) The homogeneity of a series of test specimens to establish the test parameters should be evaluated based on the geological description and rock classification parameters.

(3) The determined strength parameters relate to intact rock. In-situ properties can only be established taking into account the upscaling from element testing of intact rock to the mass properties of the rock in-situ.

Section 6   Ground investigation report

6.1 General requirements

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

(2)P The Ground Investigation Report shall consist of the following:
−   a presentation of all appropriate geotechnical information including geological features and relevant data;
−   a geotechnical evaluation of the information, stating the assumptions made in the interpretation of the test results.

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

(4) The Ground Investigation Report may include derived values.

(5)P The Ground Investigation Report shall state known limitations of the results, if appropriate.

(6) The Ground Investigation Report should propose necessary further field and laboratory investigations, with comments justifying the need for this further work. Such proposals should be accompanied by a detailed programme for the further investigations to be carried out.

6.2 Presentation of geotechnical information

(1)P The presentation of geotechnical information shall include a factual account of all field and laboratory investigations.

(2) The factual account should include the following information, as relevant:
−   the purpose and scope of the geotechnical investigation including a description of the site and its topography, of the planned structure and the stage of the planning the account is referring to;
−   a classification of the structure into a geotechnical category;
−   the names of all consultants and subcontractors;
−   the dates between which field and laboratory investigations were performed;
−   the field reconnaissance of the site of the project and the surrounding area noting particularly:
    a) evidence of groundwater;
    b) behaviour of neighbouring structures;
    c) exposures in quarries and borrow areas;
    d) areas of instability;
    e) any exposures of mining activity at the site and in the neighbourhood;
    f) difficulties during excavation;
    g) history of the site;
    h) geology of the site, including faulting;
    i) survey data with plans showing the structure and the location of all investigation points;
    j) information from aerial photographs;
    k) local experience in the area;
    l) information about the seismicity of the area.

(3)P The presentation of geotechnical information shall include documentation of the methods, procedures and results including all relevant reports of:
− desk studies;
− field investigations, such as sampling, field tests and groundwater measurements;
− laboratory tests.

(4)P The results of the field and laboratory investigations shall be presented and reported according to the requirements defined in the EN and/or ISO standards applied in the investigations.

6.3 Evaluation of geotechnical information

(1)P The evaluation of the geotechnical information shall be documented and include, as appropriate:
− the results of the field investigations and laboratory tests evaluated according to Sections 3 to 5 of this standard;
− a review of the results of the site and laboratory investigations and all other information listed in 6.2;
− a description of the geometry of the strata;
− detailed descriptions of all strata including their physical properties and their deformation and strength characteristics, referring to the results of the investigations;
− comments on irregularities such as cavities and zones of discontinuous material.

(2)P It shall be documented, if appropriate, that:
− the results were interpreted taking into account the groundwater table, ground type, drilling method, sampling method, transport, handling and specimen preparation;
− the strata subdivision assumed from desk studies and site inspections were reconsidered in light of the results obtained.

(3) The documentation of the evaluation of the geotechnical information should include as appropriate:
− the tabulation and graphical presentation of the results of field investigation and laboratory testing in cross-sections of the ground showing the relevant strata and their boundaries including the groundwater table in relation to the requirements of the project;
− the values of the geotechnical parameters for each stratum;
− a review of the derived values of geotechnical parameters (see 6.4).

(4) Averaging can mask the presence of a weaker zone and should be used with caution. It is important that weak zones are identified. Variations in geotechnical parameters or coefficients can indicate significant variations in site conditions.

(5) The documentation should include comparisons of the specific results with experience for each geotechnical parameter, giving special consideration to anomalous results for a given stratum when compared with any results from other types of laboratory and field tests capable of measuring the same geotechnical parameter.

(6) The documentation of the evaluation should substantiate the following aspect: strata in which ground parameters differ only slightly may be considered as one stratum.

(7) A sequence of fine layers with greatly differing composition and/or mechanical properties may be considered as one stratum if the overall behaviour is relevant, and the behaviour can be adequately represented by ground parameters selected for the stratum.

(8) When deriving the boundary between different ground layers and the groundwater level, there may be interpolated linearly between the investigation points provided the spacing is sufficiently small and the geological conditions are sufficiently homogeneous. Such application of linear interpolations and their justification should be reported.

6.4 Establishment of derived values

(1)P If correlations have been used to derive geotechnical parameters or coefficients, the correlations and their applicability shall be documented.

Annex A

(Informative)

List of test results of geotechnical test standards

(1) In Table A.1, field and laboratory tests are listed together with the respective test results which should be presented in the Ground Investigation Report (if applicable).

Table A.1 — List of test results of geotechnical standards

Field test [a]	Test results
CPT	– Cone penetration resistance (q_c) – Local unit side friction (f_s) – Friction ratio (R_f)
CPTU	– Corrected cone resistance (q_t) – Local unit side friction (f_s) – Measured pore pressure (u)
Dynamic probing	– Number of blows N_{10} for the following tests: DPL, DPM, DPH – Number of blows (N_{10}) or (N_{20}) for the DPSH test
SPT	– Number of blows N – Energy correction E_r – Soil description
Ménard pressuremeter test	– Pressuremeter modulus (E_M) – Creep pressure (p_f) – Limit pressure (p_{LM}) – Expansion curve
Flexible dilatometer test	– Dilatometer modulus (E_{FDT}) – Deformation curve
All other pressuremeter tests	– Expansion curve
Field vane test	– Undrained shear strength (uncorrected) (c_{fv}) – Remoulded undrained shear strength (c_{rv}) – Torque-rotation curve

Weight sounding test	– Continuous record of weight sounding resistance
	– Weight sounding resistance is:
	  – either the penetration depth for a standard load;
	  – or the number of half-turns required for every 0,2 m penetration, at the standard load of 1 kN

Plate loading test	– Ultimate contact pressure (p_u)
Flat dilatometer test	– Corrected lift-off pressure (p_0) – Corrected expansion pressure (p_1) at 1,1 mm – Dilatometer modulus E_{DMT}, material index (I_{DMT}) and horizontal stress index (K_{DMT})

Table A.1 (*continued*)

Laboratory test [b]	Test results
Water content (soil)	− Value of (w)
Bulk mass density (soil)	− Value of (ρ)
Particle mass density (soil)	− Value of (ρ_s)
Particle size distribution (soil)	− Grain size distribution curve
Consistency limits (soil)	− Plastic and liquid limit values (w_P), (w_L)
Density index (soil)	− Values of e_{max}, e_{min} and I_D
Organic content (soil)	− [AC1] Loss on ignition [AC1] (C_{OM})
Carbonate content (soil)	− Value of carbonate content (C_{CaCO3})
Sulfate content (soil)	− Value of sulfate content ($C_{SO4}{}^{2-}$) or ($C_{SO3}{}^{2-}$)
Chlorite content (soil)	− Value of chlorite content (C_{Cl})
pH (soil)	− Value of pH
Compressibility oedometer (soil)	− Compressibility curve (different options) − Consolidation curves (different options) − Secondary compression curve (creep curve) − Values of E_{oed} (stress interval) and σ'_p or C_s, C_c, σ'_p − Value of C_α
Laboratory vane (soil)	− Value of strength index (c_u)
Fall cone (soil)	− Value of strength index (c_u)
Unconfined compression (soil)	− Value of strength index $q_u = 2c_u$
Unconsolidated undrained compression (soil)	− Value of undrained shear strength (c_u)
Consolidated triaxial compression (soil)	− Stress-strain curve(s) and pore pressure curve − Stress paths − Mohr circles − c', φ' or c_u − Variations of c_u with σ'_c − Deformation parameter(s) (E') or (E_u)
Consolidated direct shear box (soil)	− Stress-displacement curve − τ−σ diagram − c', φ' − Residual parameters

Table A.1 (*continued*)

Laboratory test [b]	Test results
California bearing ratio (soil)	− Value of the CBR index (I_{CBR})

Permeability (soil)	− Value of the coefficient of permeability (k):
	  − from direct laboratory permeability test
	  − from field permeability tests
	  − from oedometer test

Water content (rock)	− Value of w
Density and porosity (rock)	− Value of ρ and n
Swelling (rock)	− Swelling Strain Index − Swelling pressure − Free swell − Swell under constant load
Uniaxial compression and deformability (rock)	− Value of σ_C − Value of deformation modulus (E) − Value of Poisson's ratio (ν)
Point-load test (rock)	− Strength index I_{s50}
Direct shear test (rock)	− Stress-displacement curve − Mohr diagram − c', φ' − Residual parameters
Brazil test (rock)	− Tensile strength (σ_T)
Triaxial compression test (rock)	− Stress-strain curve(s) − Stress paths − Mohr circles − c', φ' − Values of deformation modulus (E) and Poisson's ratio (ν)
[a] See Section 4.	
[b] See Section 5.	

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Annex B

(Informative)

Planning of geotechnical investigations

B.1 Stages of ground investigations in geotechnical design, execution of works and exploitation of the structure

| STRATEGY FOR DESIGN | Proposed structure | Geology | Existing information and experience |

Strategy for geotechnical design — EC7 Parts 1-2

GEOTECHNICAL INVESTIGATIONS ⟷ **DESIGN**

Planning and reevaluation of the investigation program — EC7 Parts 1-2

Proposed structure

Geology → Geological model

Field investigations — Field sampling and testing standards → Test results

Laboratory investigations — Laboratory standards → Test results

Reporting → Ground investigation report

Evaluation of geotechnical parameters and coefficients — EC7, parts 1 and 2 → Geotechnical model

Design
- geotechnical
- structural
Specifications — EC7 part 1 →
Geotechnical design report
Design report (final project)
Program for inspection, supervision and monitoring

Call for bids based on a project

EXECUTION OF WORKS

Execution studies → Execution project

Inspections, supervision and monitoring during construction → Report from inspection, supervision and monitoring

Reception of the structure

EXPLOITATION OF STRUCTURE

Inspections and monitoring for maintenance → Report from inspection, supervision and monitoring

B.2 Selection of ground investigation methods in different stages

Table B.1— Example of the selection of ground investigation methods in different stages

Preliminary investigations		Design investigations			Control investigations	
Desk study of topographical, historical, geological and hydrogeological maps Mineral extraction Aerial photo - interpretation Archives of previous construction works and investigations Site inspection Preliminary geophysical surveys Preliminary intrusive investigations	**Fine soil** CPT, SS, DP, SE FVT or SPT OS TP, PS, OS GW **Coarse soil** SS, CPT, DP, SR SPT AS, OS, TP GW **Rock** SR, CPT, MWD PLT CS, AS, TP GW	Preliminary choice of foundation method Preliminary choice of foundation method Preliminary choice of foundation method	**Pile foundation**	SS, CPT, DP, SR FVT, SPT, PIL PS, OS, CS, PMT GWC	Verification of choice of foundation method and design procedure, control of ground improvement works and stability during construction	PIL, Pile driving tests, Stress wave measurements GWC, settlements, Inclinometers
			Shallow foundation	SS or CPT, DP FVT, DMT or PMT, BJT PS, OS, CS, TP GWC		Check of the soil type Check of the stiffness (CPT) Settlements, Inclinometers, GWC Volume change potential due to water content change
			Pile foundation	CPT, DP, SR SPT, DMT, PIL OS,, TP GWO		PIL, Pile driving tests Stress wave measurements GWC, settlements Inclinometers
			Shallow foundation	CPT,DP SPT, PMT, BJT, DMT, PLT OS, TP GWO		Check of the soil type Check of the stiffness (CPT, DP, SPT) Settlements
			Pile or shallow foundation	SR, MWD, mapping of discontinuities RDT, PMT, BJT TP, CS GWO		Check inclination and discontinuities in the rock and its surface Check contact between pile toe/ foundation and rock surface Verify water conditions of flow and pressure

Table B.1 (*continued*)

Abbreviations	
Field testing	**Sampling**
BJT Borehole jack test	PS Piston sampler
DP Dynamic probing	CS Core sampler
SR Soil/rock sounding	AS Auger sampler
SS Static sounding (e,g, weight sounding test, WST)	OS Open sampler
CPT(U) Cone penetration test (with pore pressure recording)	TP Test pit sampling
SPT Standard penetration test	
PMT Pressuremeter test	
DMT Dilatometer test	**Groundwater measurements**
FVT Field vane test	GW Groundwater measurements
PLT Plate load test	GWO Groundwater measurements with open system
MWD Measuring while drilling	GWC Groundwater measurements with closed system
SE Seismic measurements	
PIL Pile load test	
RDT Rock dilatometer test	
	Notes:
	Soils include naturally deposited and anthropogenic deposits
	Surveying and logging are not included in this chart
	Laboratory tests are not presented on this table

AC₁

B.3 examples of recommendations for the spacing and depth of investigations

(1) The following spacing of investigation points should be used as guidance:
- for high-rise and industrial structures, a grid pattern with points at 15 m to 40 m distance;
- for large-area structures, a grid pattern with points at not more than 60 m distance;
- for linear structures (roads, railways, channels, pipelines, dikes, tunnels, retaining walls), a spacing of 20 m to 200 m;
- for special structures (e.g. bridges, stacks, machinery foundations), two to six investigation points per foundation;
- for dams and weirs, 25 m to 75 m distance, along relevant sections.

(2) For the investigation depth z_a the following values should be used as guidance. (The reference level for z_a is the lowest point of the foundation of the structure or structural element, or the excavation base.) Where more than one alternative is specified for establishing z_a, the one which yields the largest value should be applied.

NOTE   For very large or highly complex projects, some of the investigation points generally extend to greater depths than those specified under B.3 (5) to B.3 (13).

(3) Greater investigation depths should always be selected, where unfavourable geological conditions, such as weak or compressible strata below strata of higher bearing capacity, are presumed.

(4) Where structures under B.3 (5) to B.3 (8) and B.3 (13) are built on competent strata, the depth of investigation can be reduced to $z_a = 2$ m, unless the geology is indistinct, in which case at least one borehole should be taken down to a minimum of $z_a = 5$ m. If a bedrock formation is encountered at the proposed base of the structure, this should be taken as the reference level for z_a. Otherwise, z_a refers to the surface of the bedrock formation.

(5) For high-rise structures and civil engineering projects, the larger value of the following conditions should be applied (see Figure B.1 a)):

- $z_a \geq 6$ m;
- $z_a \geq 3{,}0 b_F$.

where b_F is the smaller side length of the foundation.

(6) For raft foundations and structures with several foundation elements whose effects in deeper strata are superimposed on each other:

$z_a \geq 1{,}5 \cdot b_B$

where b_B is the smaller side of the structure, (see Fig. B.1 b)).

a) foundation   b) structure
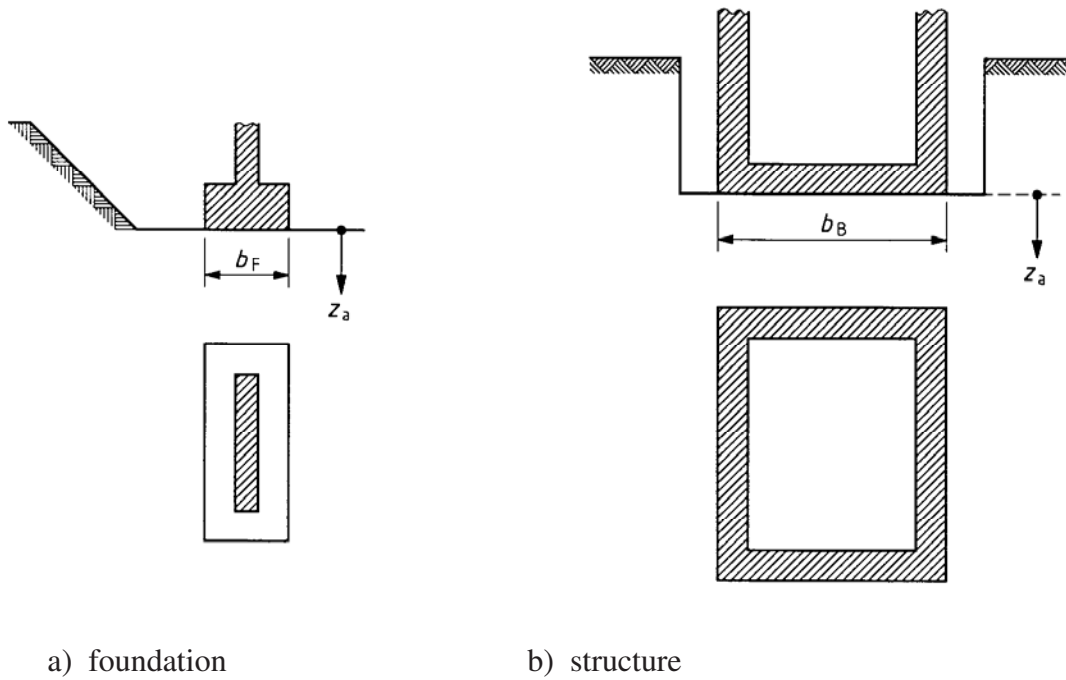
Figure B.1 — High-rise structures, civil engineering projects

(7) Embankments and cuttings, the larger value of the following conditions should be met (see Figure B.2):

a) embankment   b) cutting
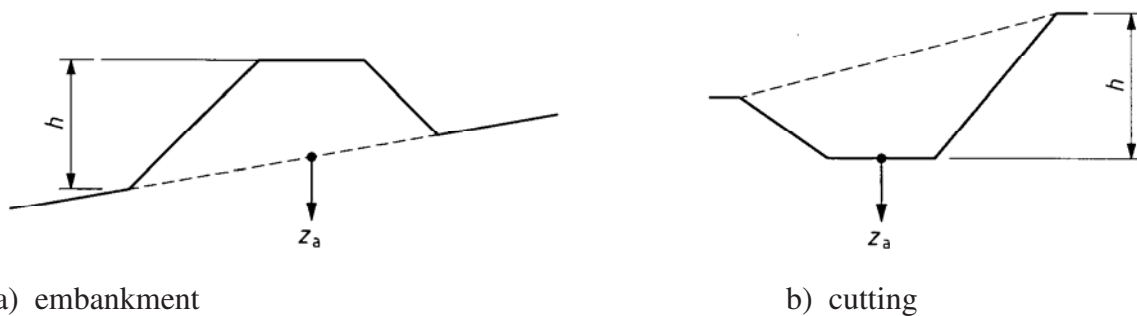
Figure B.2 — Embankments and cuttings

a) For dams:

   – $\quad 0,8h < z_a < 1,2h$
   – $\quad z_a \geq 6$ m

   where h is the embankment height.

b) For cuttings:

   – $\quad z_a \geq 2,0$ m
   – $\quad z_a \geq 0,4h$

   where h is the dam height or depth of cutting.

(8) Linear structures, the larger value of the following conditions should be met (see Figure B.3):

a) road   b) trench
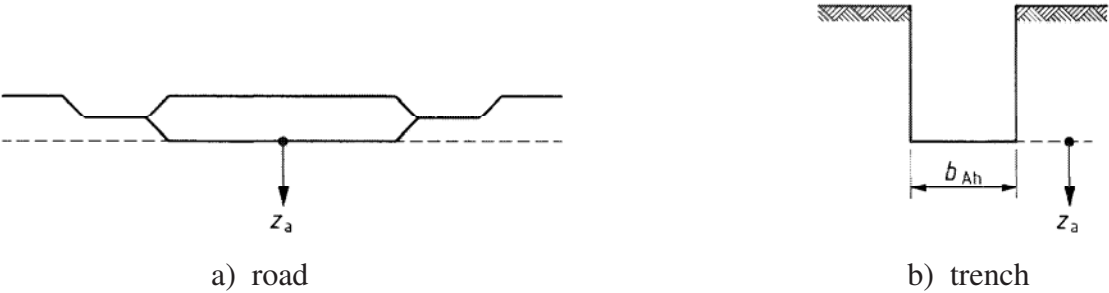
Figure B.3 — Linear structures

a) For roads and airfields:

$z_a \geq 2$ m below the proposed formation level.

b) For trenches and pipelines, the larger value of:

-   $z_a \geq 2$ m below the invert level;
-   $z_a \geq 1{,}5b_{Ah}$

where b_{Ah} is the width of excavation.

c) Where relevant, the recommendations for embankments and cuttings should be followed.

(9) For small tunnels and caverns, (see Figure B.4):

$$b_{Ab} < z_a < 2{,}0b_{Ab}$$

where b_{Ab} is the width of excavation.

The groundwater conditions described in (10) b) should also be taken into account.
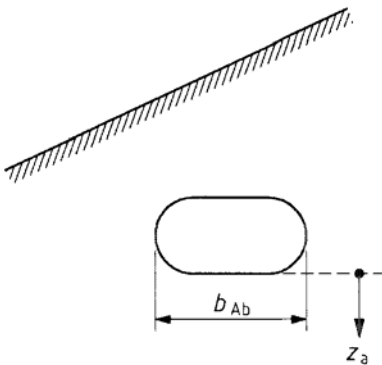
Figure B.4 — Tunnels and caverns

(10) Excavations (see Figure B.5).

a) Where the piezometric surface and the groundwater tables are below the excavation base, the larger value of the following conditions should be met:

  – $z_a \geq 0,4h$
  – $z_a \geq (t + 2,0)$ m

where
t is the embedded length of the support; and
h is the excavation depth.

b) Where the piezometric surface and the groundwater tables are above the excavation base, the larger value of the following conditions should be met:

  – $z_a \geq (1,0H + 2,0)$ m
  – $z_a \geq (t + 2,0)$ m

where
H is the height of the groundwater level above the excavation base; and
t is the embedded length of the support.

⟨AC₁⟩ If no stratum of low permeability is encountered down to these depths: ⟨AC₁⟩
    $z_a \geq t + 5$ m.
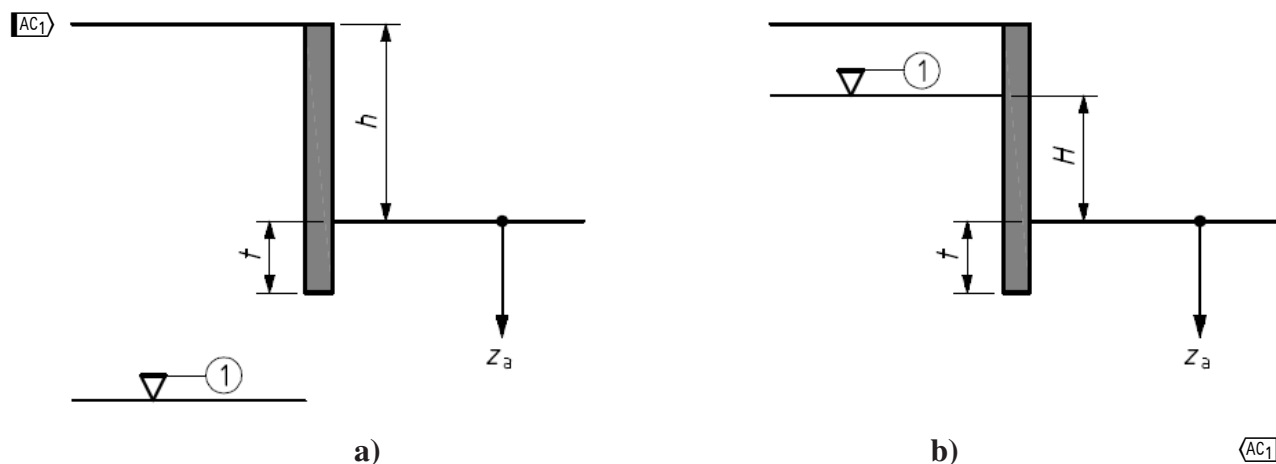
Key

1   groundwater

Figure B.5 — Excavations

(11) For water-retaining structures, z_a should be specified as a function of the proposed level of impounded water, the hydrogeological conditions and the construction method.

(12) For cut-off walls (see Figure B.6):

  – $z_a \geq 2$ m below the surface of the stratum impermeable to groundwater.
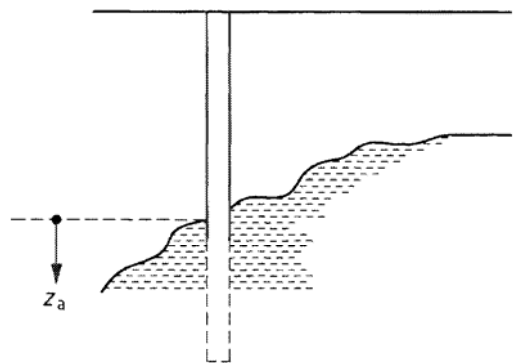
Figure B.6 — Cut-off wall

(13) For piles (see Figure B.7), the following three conditions should be met:

– $z_a \geq 1{,}0 b_g$
– $z_a \geq 5{,}0$ m
– $z_a \geq 3 D_F$

where
D_F is the pile base diameter; and
b_g is the smaller side of the rectangle circumscribing the group of piles forming the
         foundation at the level of the pile base.
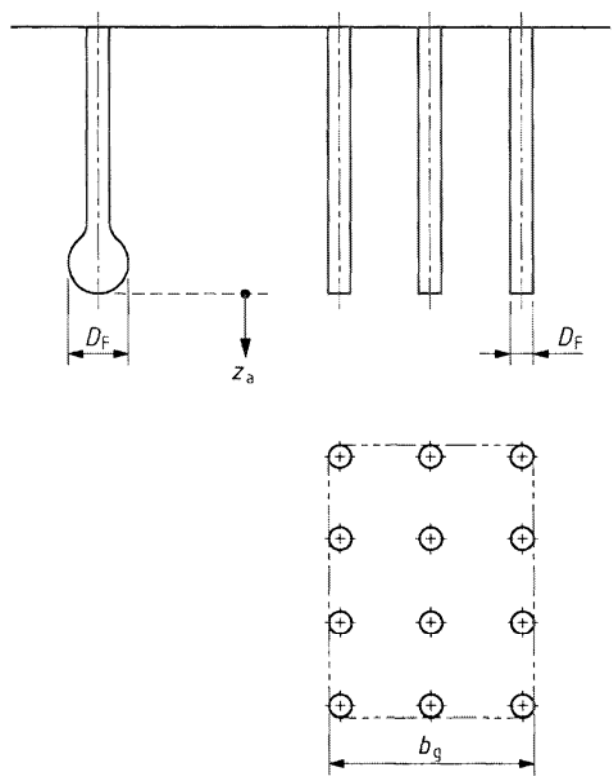
Figure B.7 — Pile groups

Annex C

(Informative)

Example of groundwater pressure derivations based on a model and long term measurements

(1) The natural groundwater pressure is part of the hydrological cycle being influenced by the precipitation, evapotranspiration, snow melting, surface run-off etc.

(2) In order to establish a model of the groundwater situation for a building or a civil engineering project site and the surrounding area, available hydrogeological information should be compiled and compared to the actual groundwater measurements. Such information could be:
− water level fluctuations;
− geohydrological maps;
− previous measurements in the surroundings;
− typical water levels of surface water or in wells;
− long term measurements in similar aquifers.

(3) The groundwater measurements for a project normally only contain a short series of measurements. In these cases it is important to make a prediction of the expected groundwater pressure for the actual design situation and site. Such a prediction can be based on the model mentioned above and on long term measurements of the groundwater in a similar aquifer in the same region as the project in combination with a short term measurement on the site.

(4) By using statistical methods, it became possible to predict the groundwater pressure within a few kPa based on 15 years measurements in a reference system and a 3 month measuring period on the actual site, see Figure C.1.

(5) It is also possible to simulate the groundwater fluctuations by a conceptual model. Precipitation and air temperature can be used as input in the model. The groundwater response is calibrated against long term measured groundwater fluctuations in the region.

NOTE   For additional information and examples, see X.2.

A) reference pipe   B) prediction pipe
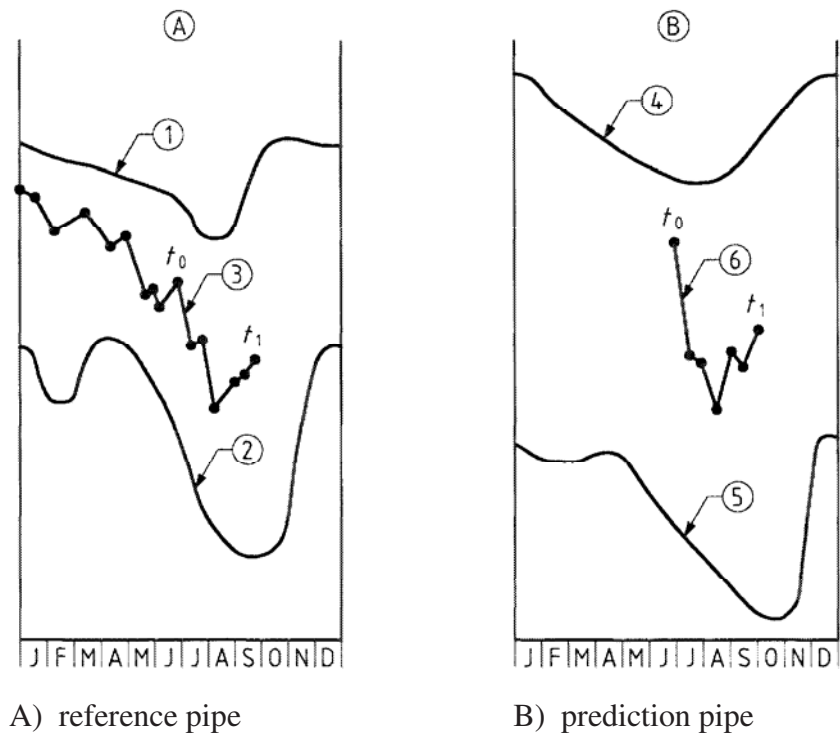
Key

1   maximum groundwater levels measured in the reference pipe through 15 years.
2   minimum groundwater levels measured in the reference pipe through 15 years.
3   measured groundwater levels in the reference pipe during the same year as the observations in the prediction pipe on the actual test site.
4   predicted maximum groundwater level in the prediction pipe on the actual test site.
5   predicted minimum groundwater level in the prediction pipe on the actual test site.
6   measured values of groundwater level in the prediction pipe on the test site from time t_0 through t_1.

NOTE   The diagram on the left is the maximum and minimum groundwater level for a reference pipe. The diagram on the right is the actual measurements together with the predicted maximum/minimum groundwater levels.

Figure C.1 — Measured and predicted groundwater levels

Annex D
(Informative)

Cone and piezocone penetration tests

D.1 Example for deriving values of the effective angle of shearing resistance and drained Young's modulus

(1) Table D.1 is an example that can be used to derive values, from the value of q_c, of the effective angle of shearing resistance (ϕ) and drained (long term) Young's modulus of elasticity (E') for quartz and feldspar sands, for calculations of the bearing resistance and settlement of spread foundations.

(2) This example was obtained by correlating the mean value of q_c in a layer to the mean values of ϕ and E'.

Table D.1 — An example for deriving values of the effective angle of shearing resistance (ϕ') and drained Young's modulus of elasticity (E') for quartz and feldspar sands from cone penetration resistance (q_c)

Density index	Cone resistance (q_c) (from CPT) MPa	Effective angle of shearing resistance[a], (ϕ') °	Drained Young's modulus[b], (E') MPa
Very loose	0,0 – 2,5	29 – 32	< 10
Loose	2,5 – 5,0	32 – 35	10 – 20
Medium dense	5,0 – 10,0	35 – 37	20 – 30
Dense	10,0 – 20,0	37 – 40	30 – 60
Very dense	> 20,0	40 – 42	60 – 90

[a] Values given are valid for sands. For silty soil a reduction of 3° should be made. For gravels 2° should be added.

[b] E' is an approximation to the stress and time dependent secant modulus. Values given for the drained modulus correspond to settlements for 10 years. They are obtained assuming that the vertical stress distribution follows the 2:1 approximation. Furthermore, some investigations indicate that these values can be 50 % lower in silty soil and 50 % higher in gravelly soil. In over-consolidated coarse soils, the modulus can be considerably higher. When calculating settlements for ground pressures greater than 2/3 of the design bearing pressure in ultimate limit state, the modulus should be set to half of the values given in this table.

NOTE   This example was published by Bergdahl et al. (1993). For additional information and documents giving examples, see X.3.1.

D.2 Example of a correlation between the cone penetration resistance and the effective angle of shearing resistance

(1) The following is an example for deriving the effective angle of shearing resistance (ϕ) from CPT cone penetration resistance (q_c) in sands.

(2) The deterministic correlation reads as follows:

$$\varphi' = 13{,}5 \times \lg q_c + 23$$

where

φ' is the effective angle of shearing resistance, in °;

q_c is the cone penetration resistance, in MPa.

This relationship is valid for poorly-graded sands ($C_U < 3$) above groundwater and cone penetration resistances in the range 5 MPa $\leq q_c \leq$ 28 MPa.

NOTE 1   The example was established from electrical cone penetrometer tests and laboratory triaxial tests.

NOTE 2   This example was published by Stenzel et al. (1978) and in DIN 4094-1 (2002). For additional information and documents giving examples, see X.3.1.

D.3 Example of a method to determine the settlement for spread foundations

(1) The following is an example of a semi-empirical method for calculating settlements of spread foundations in coarse soil. The value for Young's modulus of elasticity (E') derived from the cone penetration resistance (q_c), to be used in this method is:

— $E' = 2{,}5\ q_c$, for axisymmetric (circular and square) foundations; and

— $E' = 3{,}5\ q_c$, for plane strain (strip) foundations.

(2) The settlement (s) of a foundation under load pressure (q) is expressed as:

$$s = C_1 \times C_2 \times (q - \sigma'_{v0}) \times \int_0^{z_1} \frac{I_z}{C_3 \times E'} \mathrm{d}z$$

where

C_1 is $1 - 0{,}5 \times [\sigma'_{v0}/(q - \sigma'_{v0})]$;

C_2 is $1{,}2 + 0{,}2 \times \lg t$;

C_3 is the correction factor for the shape of the spread foundation:

       — 1,25 for square foundations; and

       — 1,75 for strip foundations with $L > 10B$;

σ'_{v0} is the initial effective vertical stress at the level of the foundation;

〈AC₁〉 z_i is the depth influenced by the foundation pressure and width, respectively, in m. 〈AC₁〉

I_z is a strain influence factor (see below).

(3) Figure D.1 gives for axisymmetric (circular and square) spread foundations and for plane strain (strip spread foundations) the distribution of the vertical strain influence factor (I_z).

NOTE 1   The cone penetration resistance (q_c) in this example stems from measurements carried out with an electrical cone penetrometer.

NOTE 2   This example was published by Schmertmann (1970) and Schmertmann et al (1978). For additional information and examples, see X.3.1.

$$I_{ZP} = 0.5 + 0.1 \sqrt{\frac{q - \sigma'_{vo}}{\sigma'_{vp}}}$$
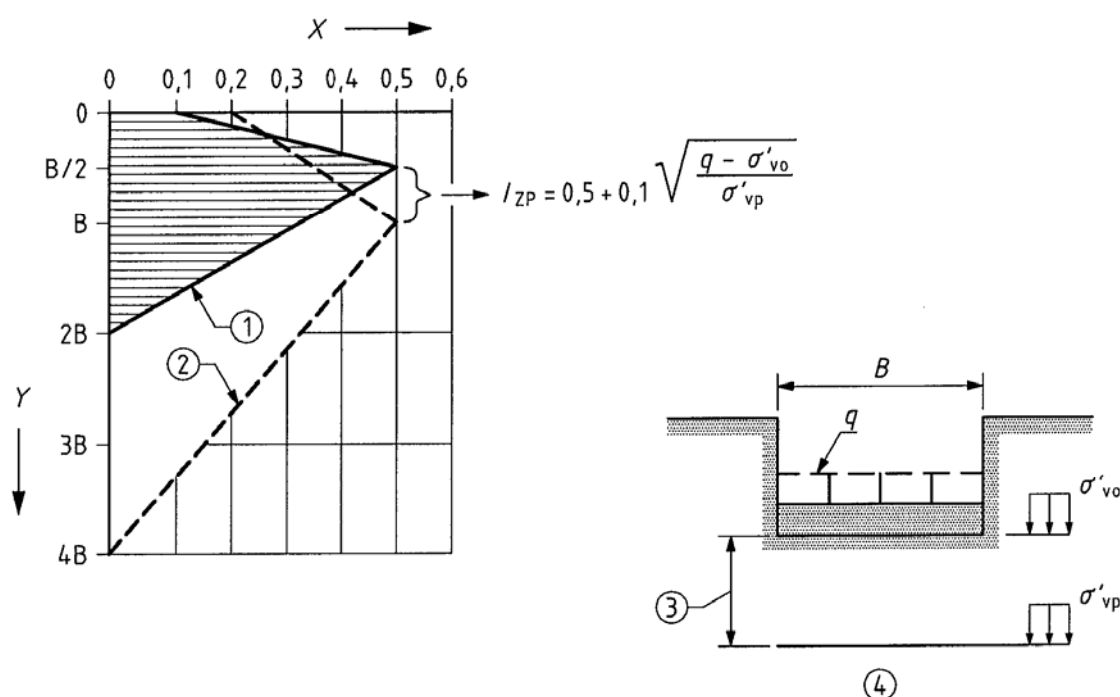
Key

x	rigid footing vertical strain influence factor I_z
y	relative depth below footing
1	axi-symmetric (L/B=1)
2	plane strain ($L/B > 10$)
3	$B/2$ (axi-symmetric); B (plane strain)
4	depth to I_{zp}

Figure D.1 — Strain influence factor diagrams

D.4 Example of a correlation between the oedometer modulus and the cone penetration resistance

(1) Table D.2 gives example of values of α (see 4.3.4.1 (9) Equation 4.3) for various types of soil as function of the cone penetration resistance.

NOTE   This example was published by Sanglerat (1972). For additional information and examples, see X.3.1.

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Table D.2 — Examples of values of α

Soil	q_c	α
Low-plasticity clay	$q_c \leq 0,7$ MPa	$3 < \alpha < 8$
	$0,7 < q_c < 2$ MPa	$2 < \alpha < 5$
	$q_c \geq 2$ MPa	$1 < \alpha < 2,5$
Low-plasticity silt	$q_c < 2$ MPa	$3 < \alpha < 6$
	$q_c \geq 2$ MPa	$1 < \alpha < 2$
Very plastic clay Very plastic silt	$q_c < 2$ MPa	$2 < \alpha < 6$
	$q_c > 2$ MPa	$1 < \alpha < 2$
Very organic silt	$q_c < 1,2$ MPa	$2 < \alpha < 8$
Peat and very organic clay	$q_c < 0,7$ MPa	
	$50 < w \leq 100$ (%)	$1,5 < \alpha < 4$
	$100 < w \leq 200$ (%)	$1 < \alpha < 1,5$
	$w > 200$ (%)	$0,4 < \alpha < 1.0$
Chalks	$2 < q_c \leq 3$ MPa	$2 < \alpha < 4$
	$q_c > 3$ MPa	$1,5 < \alpha < 3$
Sands	$q_c < 5$ MPa	$\alpha = 2$
	$q_c > 10$ MPa	$\alpha = 1,5$

D.5 Examples of establishing the stress-dependent oedometer modulus from CPT results

(1) This is an example of the derivation of the vertical stress dependent oedometer settlement modulus (E_{oed}), frequently recommended for settlement calculation of spread foundations, defined as follows:

$$E_{oed} = w_1\, p_a \left(\frac{\sigma'_v + 0,5\, \Delta\sigma'_v}{p_a} \right)^{w_2}$$

where

w_1 is the stiffness coefficient;

w_2 is the stiffness exponent;

      for sands with a uniformity coefficient $C_U \leq 3$, $w_2 = 0,5$;

      for clays of low plasticity ($I_p \leq 10$; $w_L \leq 35$), $w_2 = 0,6$;

σ'_v is the effective vertical stress at the base of the foundation or at any depth below it due to overburden of the soil;

$\Delta\sigma'_v$ is the effective vertical stress caused by the structure at the base of the foundation or at any depth below it;

p_a is the atmospheric pressure;

I_p is the plasticity index

w_L is the liquid limit.

(2) Values for the stiffness coefficient w_1 can be derived from CPT results using for example the following equations, depending on the soil type:

Poorly-graded sands ($C_U \leq 3$) above groundwater;

[AC1] $w_1 = 167 \lg q_c + 113$ \qquad (range of validity: 5 MPa $\leq q_c \leq$ 30 MPa) [AC1]

well-graded sands ($C_U > 6$) above groundwater;

[AC1] $w_1 = 463 \lg q_c - 13$ \qquad (range of validity: 5 MPa $\leq q_c \leq$ 30 MPa) [AC1]

low plasticity clays of at least stiff consistency ($0,75 \leq I_c \leq 1,30$) and above groundwater (I_c is the consistency index);

[AC1] $w_1 = 15,2q_c + 50$ \qquad (range of validity: 0,6 MPa $\leq q_c \leq$ 3,5 MPa) [AC1]

NOTE 1   The example was established from the results of tests carried out with an electrical cone penetrometer and from laboratory oedometer tests.

NOTE 2   These examples were published by Stenzel et al. (1978) and Biedermann (1984) and in DIN 4094-1:2002. For additional information and examples, see X.3.1.

D.6 Example of a correlation between compressive resistance of a single pile and cone penetration resistance

(1) In Tables D.3 and D.4 examples are given of established correlations between the results of static load tests and CPT results for coarse soil with little or no fines. Unit base resistance p_b and unit shaft resistance p_s of cast in-situ piles are given as a function of cone penetration resistance (q_c) (CPT) and normalised pile head settlement.

Table D.3 — Unit base resistance p_b of cast in-situ piles in coarse soil with little or no fines

Normalised settlement s/D_s; s/D_b	Unit base resistance p_b, in MPa, at average cone penetration resistance q_c (CPT) in MPa			
	$q_c = 10$	$q_c = 15$	$q_c = 20$	$q_c = 25$
0,02	0,70	1,05	1,40	1,75
0,03	0,90	1,35	1,80	2,25
0,10 (= s_g)	2,00	3,00	3,50	4,00
NOTE Intermediate values may be interpolated linearly. In the case of cast in-situ piles with pile base enlargement, the values shall be multiplied by 0,75. s is the normalised pile head settlement D_s is the diameter of the pile shaft D_b is the diameter of the pile base s_g is the ultimate settlement of pile head				

Table D.4 — Unit shaft resistance p_s of cast in-situ piles in coarse soil with little or no fines

Average cone penetration resistance q_c (CPT) MPa	Unit shaft resistance p_s MPa
0	0
5	0,040
10	0,080
≥ 15	0,120
NOTE Intermediate values may be interpolated linearly	

NOTE 1   The example was established from results of tests carried out with an electrical cone penetrometer.

NOTE 2   This example was published in DIN 1054 (2003-01). For additional information and examples, see X.3.1.

D.7 Example of a method to determine the compressive resistance of a single pile

(1) The following is an example of the determination of the maximum bearing resistance of a single pile on the basis of the q_c values from an electrical CPT. In case of over consolidation, or in case of an excavation after execution of a CPT, the q_c values should be reduced.

(2) The maximum compressive resistance of a pile follows from:

$$F_{max} = F_{max;base} + F_{max;shaft}$$

where

$$F_{max;base} = A_{base} \times p_{max;base}$$

and

$$F_{max;shaft} = C_p \int_0^{\Delta L} p_{max;shaft;z} \, dz$$

where
A_{base} is the cross sectional area of the base, in m^2;
C_p is the circumference of the part of the pile shaft in the layer in which the base of the pile is placed, in m;
F_{max} is the maximum compressive resistance of the pile, in MN;
$F_{max;base}$ is the maximum base resistance, in MN;
$F_{max;shaft}$ is the maximum shaft resistance, in MN;
⟨AC1⟩ $p_{max;shaft;z}$ is the maximum unit shaft resistance, at depth z, in MPa ⟨AC1⟩
$p_{max;base}$ is the maximum unit base resistance, in MPa;
ΔL is the distance from the base of the pile to the bottom of the first soil layer above the base with $q_c < 2$ MPa; moreover $\Delta L \leq$ the length of the enlarged part of the pile point if applied, in m;
z is the depth or vertical direction (positive downwards).
D_{eq} is the equivalent diameter of the base, in m;

$$D_{eq} = 1,13 \times a \sqrt{\frac{b}{a}}$$

where

a is the length of the smaller side of the base area, in m;

b is the larger side, in m, with $b \leq 1,5 \times a$;

(3) The maximum base resistance $p_{max;base}$ can be derived from the following equations:

$$p_{max;base} = 0,5\, \alpha_p\, \beta\, s \left\{ \frac{q_{c;I;mean} + q_{c;II;mean}}{2} + q_{c;III;mean} \right\}$$

and

$$p_{max;base} \leq 15 \text{ MPa}$$

where

α_p is the pile class factor given in Table D.5,

β is the factor which takes account of the shape of the pile point as shown in Figure D.3; β is found by interpolation between the borders given in Figure D.3

s is the factor which accounts for the shape of the pile base and is established as follows:

$$s = \left(1 + \frac{\sin \varphi'}{r} \right) / (1 + \sin \varphi')$$

where

r is L/B

L is the larger side of the rectangular pile point;

B is the smaller side of the rectangular pile point;

φ' is the effective angle of shearing resistance.

$q_{c;1;mean}$ is the mean of the $q_{c;I}$ values over the depth running from the pile base level to a level which is at least 0,7 times and at most 4 times the equivalent pile base diameter D_{eq} deeper (see Figure D.2);

$$q_{c;\,I;\,mean} = \frac{1}{d_{crit}} \int_0^{d_{crit}} q_{c;\,I}\, dz$$

with

      $\boxed{\text{AC}_1}$ $0,7 D_{eq} < d_{crit} < 4 D_{eq}$ $\boxed{\text{AC}_1}$

At the critical depth the calculated value of $p_{max;base}$ becomes a minimum;

$q_{cII;mean}$ is the mean of the lowest $q_{c;II}$ values over the depth going upwards from the critical depth to the pile base (see Figure D.2);

$$q_{c;II;mean} = \frac{1}{d_{crit}} \int_{d_{crit}}^0 q_{c;II}\, dz$$

$q_{cIII;mean}$ is the mean value of the $q_{c;III}$ values over a depth interval running from pile base level to a level of 8 times the pile base diameter above the pile base, or, in case $b > 1{,}5 \times a$ upto $8 \times a$ above the pile base. This procedure starts with the lowest $q_{c;II}$ value used for the computation of $q_{c;II;mean}$ (see Figure D.2);

$$q_{c;\,III;\,mean} = \frac{1}{8D_{eq}} \int_{0}^{-8D_{eq}} q_{c;\,III}\, dz$$

For continuous flight auger piles, $q_{c;III;mean}$ cannot exceed 2 MPa, unless the results of CPTs, which were performed at a distance from the pile ≤ 1 m after pile fabrication are used for the calculation of the compressive resistance;

(4) The maximum shaft resistance $p_{max;shaft;z}$ should be determined as follows:

$$p_{max;shaft;\,z} = \alpha_s \times q_{c;z;a}$$

where

α_s is the factor according to Tables D.5 and D.6;

$q_{c;z;a}$ is the cut-off value of q_c at depth z, in MPa.

If $q_{c;z} \geq 12$ MPa over a continuous depth interval of 1 m or more, then $q_{c;z;a} \leq 15$ MPa over this interval.

If the depth interval with $q_{c;z;a} > 12$ MPa is less than 1 m thick, then $q_c \leq 12$ MPa over this interval.

Table D.5 — Maximum values of α_p and α_s for sands and gravely sands

Pile class or type	α_p	α_s [a]
Soil displacement type piles, diameter > 150 mm		
– driven prefabricated piles,	1,0	0,010
– cast in place piles made by driving a steel tube with closed end. The steel pipe is reclaimed during concreting.	1,0	0,012
Soil replacement type piles, diameter > 150 mm		
– flight auger piles,	0,8	0,006 [b]
– bored piles (with drilling mud).	0,6	0,005
[a] Values valid for fine to coarse sands. For very coarse sands a reduction factor of 0,75 is necessary; for gravel this reduction factor is 0,5.		
[b] This value is used in the case of applying the results of CPTs which were carried out before pile installation. When CPTs are used that have been carried out in the vicinity of the flight auger piles, α_s may be raised to 0,01.		

Table D.6 — Maximum α_s values for clay, silt and peat

Soil type	q_c, MPa	α_s
clay	> 3	< 0,030
clay	<3	< 0,020
silt		< 0,025
peat		0

Figure D.2 — Explanation of $q_{c;I}$, $q_{c;II}$ and $q_{c;III}$
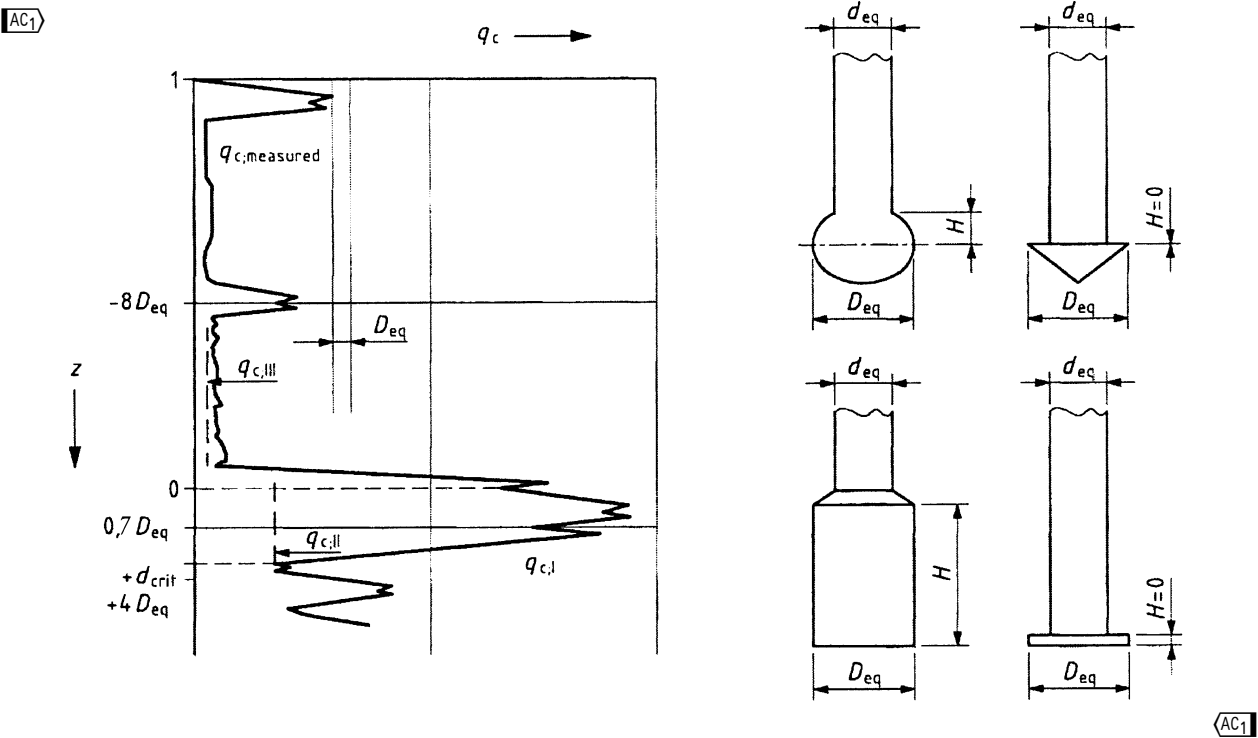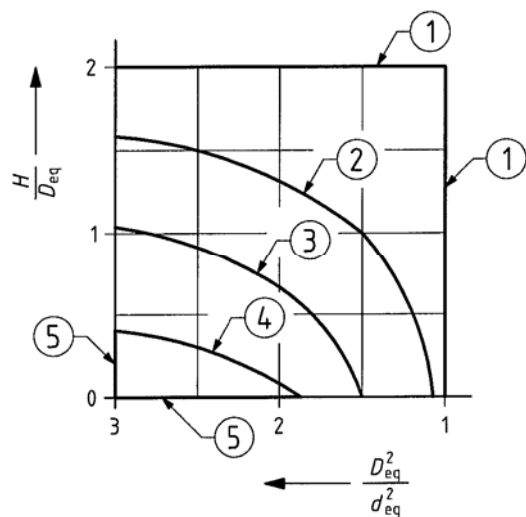
Key

1   borderline 1; $\beta = 1,0$;
2   borderline 2; $\beta = 0,9$;
3   borderline 3; $\beta = 0,8$;
4   borderline 4; $\beta = 0,7$;
5   borderline 5; $\beta = 0,6$;
For H, D_{eq} and d_{eq} see Figure D.2

Figure D.3 — Pile foot shape factor (β)

NOTE   This example was published in NEN 6743-1. For additional information and examples, see X.3.1.

Annex E

(Informative)

Pressuremeter Test (PMT)

E.1 Example of a method to calculate the bearing resistance of spread foundations

(1) The following is an example of a method to calculate the bearing resistance of spread foundations using a semi-empirical method and the results of an MPM test.

(2) The bearing resistance is calculated from:

$$R / A' = \sigma_{v0} + k(p_{LM} - p_0)$$

where

R is the resistance of the foundation against normal loads;

A' is the effective base area as defined in EN 1997-1;

σ_{v0} is the total (initial) vertical stress at the level of the foundation base;

p_{LM} is the representative value of the Ménard limit pressures at the base of the spread foundation;

p_0 is $[K_0 (\sigma_{v0} - u) + u]$ with K_0 conventionally equal to 0,5, σ_{v0} is the total (initial) vertical stress at the test level and u is the pore pressure at the test level;

k is a bearing resistance factor given in Table E.1;

B is the width of the foundation;

L is the length of the foundation;

D_e is the equivalent depth of foundation.

Table E.1 — Correlations for deriving the bearing resistance factor, k, for spread foundations

Soil category	p_{LM} category	p_{LM} MPa	k
Clay and silt	A B C	<0,7 1,2–2,0 >2,5	$0,8[1 + 0,25 (0,6 + 0,4\,B/L) \times D_e/B]$ $0,8[1 + 0,35 (0,6 + 0,4\,B/L) \times D_e/B]$ $0,8[1 + 0,50 (0,6 + 0,4\,B/L) \times D_e/B]$
Sand and gravel	A B C	<0,5 1,0–2,0 >2,5	$[1 + 0,35 (0,6 + 0,4\,B/L) \times D_e/B]$ $[1 + 0,50 (0,6 + 0,4\,B/L) \times D_e/B]$ $[1 + 0,80 (0,6 + 0,4\,B/L) \times D_e/B]$
Chalk			$1,3[1 + 0,27 (0,6 + 0,4\,B/L) \times D_e/B]$
Marl and weathered rock			$[1 + 0,27 (0,6 + 0,4\,B/L) \times D_e/B]$

NOTE   This example was published by the French Ministère de l'Equipement du Logement et des Transport (1993). For additional information and examples, see X.3.2.

E.2 Example of a method to calculate the settlements for spread foundations

(1) The following is an example of a method to calculate the settlement, (s), of spread foundations using a semi-empirical method developed for MPM tests.

$$\boxed{\text{AC}_1} \; s = (q - \sigma_{v0}) \times \left[\frac{2B_0}{9E_d} \times \left(\frac{\lambda_d B}{B_0} \right) + \frac{\alpha \lambda_c B}{9E_c} \right] \boxed{\text{AC}_1}$$

where

B_o is a reference width of 0,6 m;

B is the width of the foundation;

λ_d , λ_c are shape factors given in Table E.2;

α is a rheological factor given in Table E.3;

E_c is the weighted value of E_M immediately below the foundation;

E_d is the harmonic mean of E_M in all layers up to $8 \times B$ below the foundation;

σ_{v0} is the total (initial) vertical stress at the level of the foundation base;

q is the design normal pressure applied on the foundation.

Table E.2 — The shape coefficients, λ_c, λ_d, for settlement of spread foundations

L/B	**Circle**	**Square**	**2**	**3**	**5**	**20**
λ_d	1	1,12	1,53	1,78	2,14	2.65
λ_c	1	1,1	1,2	1,3	1.4	1,5

Table E.3 — Correlations for deriving the coefficient α for spread foundations

Type of ground	Description	E_M/p_{LM}	α
Peat			1
Clay	Over-consolidated	$\boxed{\text{AC}_1}$ >16 $\boxed{\text{AC}_1}$	1
	Normally consolidated	9–16	0,67
	Remoulded	7–9	0,5
Silt	Over-consolidated	>14	0,67
	Normally consolidated	5–14	0,5
Sand		>12	0,5
		5–12	0,33
Sand and gravel		>10	0,33
		6–10	0,25
Rock	Extensively fractured		0,33
	Unaltered		0,5
	Weathered		0,67

NOTE   This example was published by the French Ministère de l'Equipement du Logement et des Transport (1993). For additional information and examples, see X.3.2.

E.3 Example of a method to calculate the compressive resistance of a single pile

(1) The following is an example of a method to calculate the ultimate compressive resistance, Q, of piles from the MPM test, using:

$$Q = A \times k \times [p_{LM} - p_o] + P \Sigma [q_{si} \times z_i]$$

where
A — is the base area of the pile which is equal to the actual area for close ended piles and part of that area for open-ended piles;

p_{LM} — is the representative value of the limit pressure at the base of the pile corrected for any weak layers below;

p_o — is $[K_o(\sigma_{v0} - u) + u]$ with K_o conventionally equal to 0,5, and σ_{v0} is the total (initial) vertical stress at the test level and u is the pore pressure at the test level;

k — is a compression resistance factor given in Table E.4;

P — is the pile perimeter;

q_{si} — is the unit shaft resistance for soil layer i given by Figure E.1 read in conjunction with Table E.5;

z_i — is the thickness of soil layer i.

NOTE   This example was published by the French Ministère de l'Equipement du Logement et des Transport (1993). For additional information and examples, see X.3.2.

Table E.4 — Values of the compression resistance factor, k, for axially loaded piles

Soil category	p_{LM} category	p_{LM} MPa	Bored piles and small displacement piles	Full displacement piles
Clay and silt	A	<0,7	1,1	1,4
	B	1,2–2,0	1,2	1,5
	C	>2,5	1,3	1,6
Sand and gravel	A	<0,5	1,0	4,2
	B	1,0 – 2,0	1,1	3,7
	C	>2,5	1,2	3,2
Chalk	A	<0,7	1,1	1,6
	B	1,0–2,5	1,4	2,2
	C	>3,0	1,8	2,6
Marl	A	1,5–4,0	1,8	2,6
	B	>4,5	1,8	2,6
Weathered rock	A	2,5–4,0	[a]	[a]
	B	>4,5		
a) Choose k for the closest soil category.				

Table E.5 — The selection of design curves for unit shaft resistance

Soil category		Clay and silt			Sand and gravel			Chalk			Marl		Weathered rock
p_{LM} catagory		A	B	C	A	B	C	A	B	C	A	B	
Pile type													
Bored piles and caissons	No support	1	1/2	2/3	—		—	1	3	4/5	3	4/5	6
	Mud support	1	1/2	1/2	1	1/2	2/3	1	3	4/5	3	4/5	6
	Temporary casing	1	1/2	1/2	1	1/2	2/3	1	2	3/4	3	4	—
	Permanent casing	1	1	1	1	1	2				2	3	—
Hand-dug caisson		1	2	3	—	—	—	1	2	3	4	5	6
Displacement piles	Closed end steel tube	1	2	2	2	2	3				3	4	4
	Prefab. concrete	1	2	2	3	3	3				3	4	4
	Driven cast in-situ	1	2	2	2	2	3	1	2	3	3	4	—
	Coated shaft (concrete driven steel [a])	1	2	2	3	3	4				3	4	—
Grouted piles	Low pressure	1	2	2	3	3	3	2	3	4	5	5	—
	High pressure	1	4	5	5	5	6	—	5	6	6	6	7

a   A preformed steel pile of tubular or H-section, with enlarged shoe, is driven with simultaneous pumping of concrete (or mortar) into the annular space.
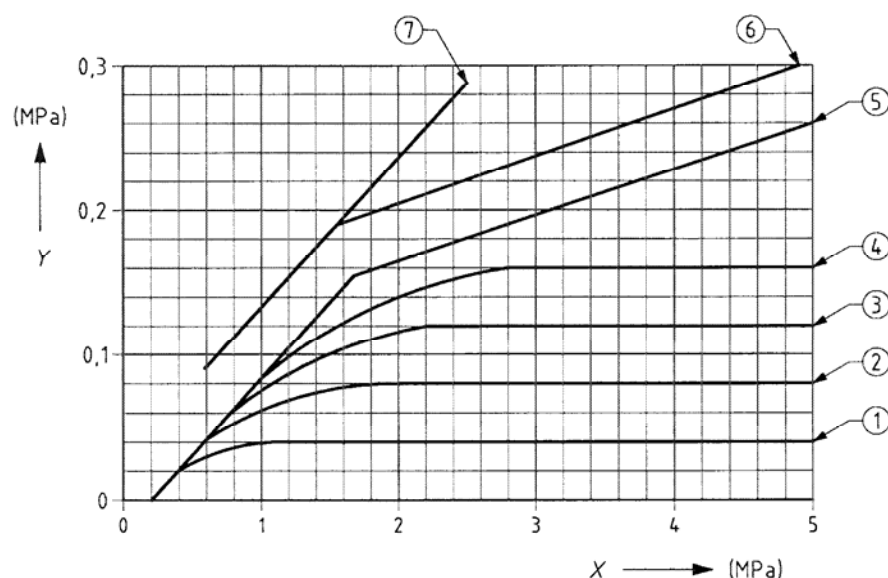
Key

(X)   Limit pressure (p_{LM}),   (Y)   Unit shaft resistance (q_{si})
1 to 7 design curves for unit shaft resitance

Figure E.1 — Unit shaft resistance for axially loaded piles

Annex F
(Informative)

Standard penetration test (SPT)

F.1 Examples of correlations between blow counts and density indices

(1) Examples of correlations of blow counts and density indices are given below.

(2) The relationship between the blow count (N_{60}), density index $I_D = (e_{max} - e)/(e_{max} - e_{min})$ and the effective total (initial) stress $\sigma'_{v;0}$ (kPa × 10^{-2}) in a given sand can be represented by the expression:

$$\frac{N_{60}}{I_D^2} = a + b\,\sigma'_{v0}$$

The parameters a and b in normally consolidated sands are nearly constant for $0,35 < I_D < 0,85$ and $0,5 < \sigma'_{v0} < 2,5$, in kPa × 10^{-2}. (see Skempton 1986, Table 8)

(3) For normally consolidated natural sand deposits, the correlation shown in Table F.1 has been established between the normalised blow count $(N_1)_{60}$ and I_D.

Table F.1 — Correlation between the normalised blow count $(N_1)_{60}$ and the density index I_D

	Very loose	Loose	Medium	Dense	Very dense
$(N_1)_{60}$	0 – 3	3 – 8	8 – 25	25 – 42	42 – 58
I_D	0 % – 15 %	15 % – 35 %	35 % – 65 %	65 % – 85 %	85 % – 100 %

(4) For fine sands, the *N*-values should be reduced in the ratio 55:60 and for coarse sands increased in the ratio 65:60.

(5) The resistance of sand to deformation is greater the longer the period of consolidation. This "ageing" effect is reflected in higher blow counts, and appears to cause an increase in the parameter *a*.

Typical results for normally consolidated fine sands are given in Table F.2.

Table F.2 — Effect of ageing in normally consolidated fine sands

	Age years	$(N_1)_{60}/I_D^2$
Laboratory tests	10^{-2}	35
Recent fills	10	40
Natural deposits	$>10^2$	55

(6) Over-consolidation increases the coefficient *b* by the factor:

$$\frac{1 + 2 \times K_0}{1 + 2 \times K_{ONC}}$$

where

K_0 and K_{0NC} are the in-situ stress ratios between horizontal and vertical effective stresses for the over-consolidated and normally-consolidated sand respectively.

(7) All the above mentioned correlations have been established for predominantly silica sands. Their use in more crushable and compressible sands, such as calcareous sands or even silica sands containing a non-negligible amount of fines, may lead to an underestimation of I_D.

NOTE   These examples were published by Skempton (1986). For additional information and examples, see X.3.3.

F.2 Examples of deriving values for the effective angle of shearing resistance

(1) Table F.3 is an example that can be used to derive values of the effective angle of shearing resistance of silica sands, (φ'), from the density index (I_D). The values of φ' are also influenced by the angularity of the particles and the stress level (see Table F.3).

Table F.3 — Correlation between the density index (*I*d) and the effective angle of shearing resistance of silica sands, (φ'), in degrees

Density index I_D	Fine		Medium		Coarse	
%	Uniform	Well-graded	Uniform	Well-graded	Uniform	Well-graded
40	34	36	36	38	38	41
60	36	38	38	41	41	43
80	39	41	41	43	43	44
100	42	43	43	44	44	46

NOTE   This example was published by the US Army Corps of Engineers (1993). For additional information and examples, see X.3.3. 3.

F.3 Example of a method to calculate the settlement of spread foundations

(1) This is an example of an empirical direct method for the calculation of settlements in granular soil of spread foundations.

(2) The settlement for stresses below the over consolidation pressure is assumed to be 1/3 of that corresponding to the normally consolidated sand. The immediate settlement, s_i, in mm, of a square footing of width *B*, in m, for an over consolidated sand, if $q' \geq \sigma'_p$, is then given by:

$$s_i = \sigma'_p \times B^{0,7} \times \frac{I_{cc}}{3} + (q' - \sigma'_p) \times B^{0,7} \times I_{cc}$$

where

σ'_p is maximum previous overburden pressure, in kPa;

q' is average effective foundation pressure, in kPa;

I_{cc} is $a_f/B^{0,7}$;

a_f is the foundation subgrade compressibility, $\Delta s_i/\Delta q'$, in mm/kPa.

If $q' \le \sigma'_p$ then the equation becomes:

$$s_i = \sigma'_p \times B^{0,7} \times \frac{I_{cc}}{3}$$

And for normally consolidated sands:

$$s_i = (q' - \sigma'_p) \times B^{0,7} \times I_{cc}$$

(3) Through a regression analysis of settlement records, the value of I_{cc} is obtained through the expression:

$$I_{cc} = 1,71/\overline{N}^{1,4}$$

where

$\overline{N}$ is the average SPT blow count over the depth of influence.

The standard error of a_f varies from about 1,5 for $\overline{N}$ greater than 25 to 1,8 for $\overline{N}$ less than about 10.

(4) The N-values for this particular empirical method should not be corrected for the overburden pressure. No mention is made of the energy ratio (E_r) corresponding to the N-values. The effect of the water table is supposed to be already reflected in the measured blow count, but the correction $N' = 15 + 0,5 \times (N - 15)$ for submerged fine or silty sands should be applied for $N > 15$.
In cases involving gravels or sandy gravels, the SPT blow count should be increased by a factor of about 1,25.

(5) The value of $\overline{N}$ is given by the arithmetic mean of the measured N-values over the depth of influence, $z_I = B^{0,75}$, within which 75 % of the settlement takes place for cases where N increases or is constant with depth. Where N shows a consistent decrease with depth, the depth of influence is taken as $2B$ or the bottom of the soft layer whichever is the lesser.

(6) A correction factor f_s for the length-to-width ratio (L/B) of the foundation

$$f_S = \left[\frac{1,25 \times \frac{L}{B}}{\frac{L}{B} + 0,25} \right]^2$$

should be applied. The value of f_s tends to 1,56 as L/B tends to infinity. No depth (D) correction factor has to be applied for $D/B < 3$.

(7) Foundations in sands and gravels exhibit time-dependent settlements. A correction factor, f_t, should be applied to the immediate settlement given by:

$$f_t = (1 + R_3 + R_t \lg t/3)$$

where

f_t is the correction factor for time $t \geq 3$ years;

R_3 is the time-dependent factor for the settlement that takes place during the first 3 years after construction; and

R_t is the time-dependent factor for the settlement that takes place each log cycle of time after 3 years.

(8) For static loads, conservative values of R_3 and R_t are 0,3 and 0,2 respectively. Thus at $t = 30$ years, $f_t = 1,5$. For fluctuating loads (tall chimneys, bridges, silos, turbines etc.), values of R_3 and R_t are 0,7 and 0,8 respectively so that at $t = 30$ years, $f_t = 2,5$.

NOTE   This example was published by Burland and Burbridge (1985). For additional information and examples, see X.3.3.

Annex G
(Informative)

Dynamic probing test (DP)

G.1 Examples for correlations between number of blows and density index

(1) These are examples of the density index (I_D) from the dynamic probing DP test, for different values of the uniformity coefficient (C_U) (range of validity $3 \leq N_{10} \leq 50$):

poorly-graded sand ($C_U \leq 3$) above groundwater
$I_D = 0,15 + 0,260 \lg N_{10L}$ (DPL)
$I_D = 0,10 + 0,435 \lg N_{10H}$ (DPH)

poorly-graded sand ($C_U \leq 3$) below groundwater
$I_D = 0,21 + 0,230 \lg N_{10L}$ (DPL)
$I_D = 0,23 + 0,380 \lg N_{10H}$ (DPH)

well-graded sand-gravel ($C_U \geq 6$) above groundwater
$I_D = -0,14 + 0,550 \lg N_{10H}$ (DPH).

NOTE   These examples were published by Stenzel et al. (1978) and in DIN 4094-3. For additional information and examples, see X.3.4

G.2 Example of a correlation between the effective angle of shearing resistance and the density index

(1) This is an example of deriving the effective angle of shearing resistance (φ') from the density index (I_D), for bearing capacity calculations of coarse soil (see Table G.1).

Table G.1 — Effective angle of shearing resistance (φ') of coarse soil as function of the density index (I_D) and the uniformity coefficient (C_U)

Soil type	Grading	Range of I_D %		Effective angle of shearing resistance (φ') °
Slightly fine-grained sand, Sand, sand-gravel	Poorly graded, ($C_U < 6$)	15–35	(loose)	30
		35–65	(medium dense)	32,5
		>65	(dense)	35
Sand, sand-gravel, gravel	Well-graded, ($6 \leq C_U \leq 15$)	15–35	(loose)	30
		35–65	(medium dense)	34
		>65	(dense)	38

NOTE   This example was published in DIN 1054-100. For additional information and examples, see X.3.4.

G.3 Example of establishing the stress-dependent oedometer modulus from DP results

(1) This is an example of the derivation of the vertical stress dependent oedometer settlement modulus (E_{oed}), frequently recommended for settlement calculation of spread foundations, defined as follows:

$$E_{oed} = w_1 \, p_a \left(\frac{\sigma'_v + 0,5 \, \Delta\sigma'_v}{p_a} \right)^{w_2}$$

where

w_1 is the stiffness coefficient;

w_2 is the stiffness exponent;

for sands with a uniformity coefficient $C_U \leq 3$: $w_2 = 0,5$;

for clays of low plasticity ($I_p \leq 10$; $w_L \leq 35$): $w_2 = 0,6$;

σ'_v is the effective vertical stress at the base of the foundation or at any depth below it due to overburden of the soil;

$\Delta\sigma'_v$ is the effective vertical stress caused by the structure at the base of the foundation or at any depth below it;

p_a is the atmospheric pressure;

I_p is the plasticity index;

w_L is the liquid limit.

(2) Values for the stiffness coefficient (w_1) can be derived from DP tests using for example the following equations, depending on the soil type:

poorly-graded sands ($C_U \leq 3$) above groundwater

$w_1 = 214 \lg N_{10L} + 71$ (DPL; range of validity: $4 \leq N_{10L} \leq 50$)

$w_1 = 249 \lg N_{10H} + 161$ (DPH; range of validity: $3 \leq N_{10H} \leq 10$)

low-plasticity clays of at least stiff consistency ($0,75 \leq I_c \leq 1,30$) and above groundwater (I_c is the consistency index)

$w_1 = 4N_{10L} + 30$ (DPL; range of validity: $6 \leq N_{10L} \leq 19$)

$w_1 = 6N_{10H} + 50$ (DPH; range of validity: $3 \leq N_{10H} \leq 13$).

NOTE   These examples were published by Stenzel et al. (1978) and Biedermann (1984) and in DIN 4094-3:2002. For additional information and examples, see X.3.4.

G.4 Example of correlations between the cone penetration resistance and the number of blows

(1) This is an example of estimating the cone penetration resistance (q_c) in sands and sand-gravel mixtures from results with the dynamic probing test DPH to derive ultimate bearing capacities of piles from corresponding correlations established from static pile load rests
(see Figure G.1, 4.3.4.2 (1)P and D.6).
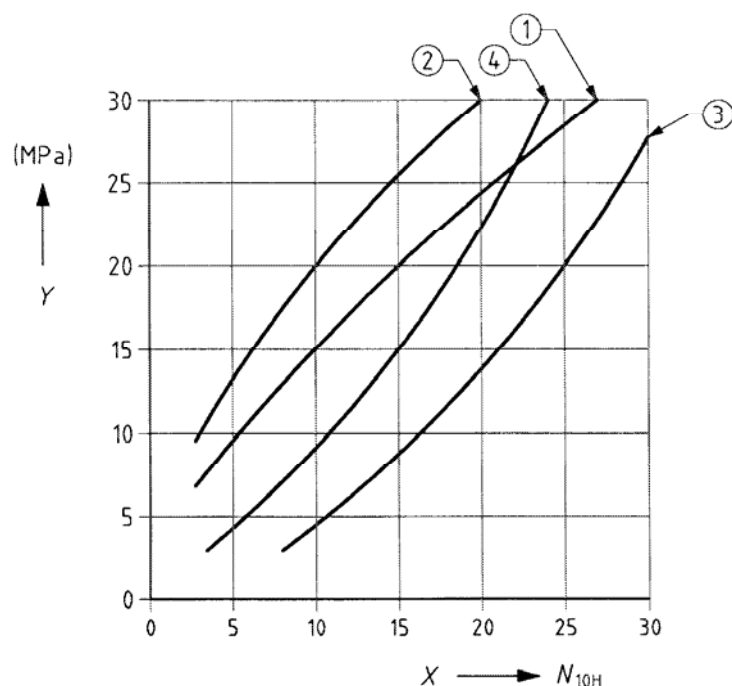
Key

(*x*)	Number of blows,	(*y*)	Cone penetration resistance (q_c),
1	Poorly-graded sand above groundwater,		
2	Poorly-graded sand below groundwater,		
3	Well-graded sand and gravel above groundwater,		
4	Well-graded sand and gravel below groundwater.		

Figure G.1 — An example of correlations between the number of blows N_{10H} and the cone penetration resistance (q_c) for poorly-graded sands and for well-graded sand-gravel

NOTE   This example was published by Stenzel et al (1978) and in DIN 4094-3. For additional information and examples, see X.3.4.

G.5 Example of a correlation between number of blows of different dynamic penetrometers

(1) This is an example for correlations between the number of blows N_{10L} of the dynamic probing test DPL and the number of blows N_{10H} of the dynamic probing test DPH for poorly-graded sands ($C_U < 3$) above the groundwater level:

a)   Input: DPH results;
       $N_{10L} = 3N_{10H}$;
       Range of validity: $3 \leq N_{10H} \leq 20$.

b)   Input: DPL results;
       $N_{10H} = 0{,}34N_{10L}$;
       Range of validity: $3 \leq N_{10L} \leq 50$.

NOTE   These examples were published by Stenzel (1978) and Biedermann (1984) and in DIN 4094-3:2002. For clays, see Butcher, A.P. McElmeel, K., Powell, J.J.M.(1995). For additional information and examples, see X.3.4.

Annex H
(Informative)

Weight sounding test (WST)

(1) This Annex gives an example of values of the effective angle of shearing resistance (φ) and drained Young's modulus of elasticity (E'), estimated from weight sounding resistance based on Swedish experience. This example correlates the mean value of weight sounding resistance in a layer to the mean values of φ and E'. (see Table H.1)

Table H.1 — Values of the effective angle of shearing resistance (φ) and drained Young's modulus of elasticity (E') for naturally deposited quartz and feldspar sands estimated from weight sounding resistance in Sweden

Density index	Weight sounding resistance[a], half-turns / 0,2 m	Effective angle of shearing resistance[b], (φ), °	Drained Young's modulus[c], (E') MPa
Very loose	0–10	29–32	<10
Loose	10–30	32–35	10–20
Medium dense	20–50	35–37	20–30
Dense	40–90	37–40	30–60
Very dense	>80	40–42	60–90

[a] Before determination of the relative density, the weight sounding resistance in silty soil should be divided by a factor of 1,3.

[b] Values given are valid for sands. For silty soil, a reduction of 3 ° should be made. For gravels, 2 ° may be added.

[c] E' is an approximation to the stress and time-dependent secant modulus. Values given for the drained modulus correspond to settlements after 10 years. They are obtained assuming that the vertical stress distribution follows the 2:1 approximation. Furthermore, some investigations indicate that these values can be 50 % lower in silty soil and 50 % higher in gravelly soil. In over-consolidated coarse soil, the modulus can be considerably higher. When calculating settlements for ground pressure greater than 2/3 of the design pressure in ultimate limit state, the modulus should be set to half the values given in this table.

(2) If only results of weight sounding tests are available, the lower value in each interval for the angle of shearing resistance and Young's modulus in Table H.1 should be selected.

(3) When evaluating weight sounding resistance diagrams for application in Table H.1, peak values caused for example by stones or pebbles should not be accounted for. Such peak values are common in weight sounding tests in gravel.

NOTE   This example was published by Bergdahl et al. (1993). For additional information and examples, see X.3.5.

Annex I

(Informative)

Field vane test (FVT)

I.1 Examples of procedures to determine correction factors for the undrained shear strength

(1) Examples of procedures for the determination of correction factors of field vane test results to obtain the undrained shear strength (c_u) from the measured value (c_{fv}) of the field vane test are given in I.2 to I.5. These correction factors are based mainly on the back-analysis of embankment failures and load tests in soft clays. All procedures lead to a value of the correction factor (μ) which is used in the following equation for assessing the undrained shear strength.

$$c_u = \mu \times c_{fv}$$

where

c_{fv} is the undrained shear strength measured in the Field vane test

μ is the correction factor

(2) The procedure to be used should be based on local experiences in the actual type of clay. It should also be considered that the drained shear strength might be lower than the undrained shear strength.

NOTE   For additional information see X.3.6.

I.2 Example of the determination of the correction factor μ based on Atterberg limits

(1) For soft, normally-consolidated clays, the correction factor (μ) is linked to the limit of liquidity or to the plasticity index. A sample correction curve is presented in Figure I.1.

(2) A correction factor greater than 1,2 should not be used without support from supplementary investigations.

(3) In fissured clays, a correction factor as low as 0,3 can be necessary. In fissured clays, the undrained shear strength should be determined from other methods than from Field vane tests e.g. Plate load tests.

NOTE   The Danish Geotechnical Institute (1959) gives examples of correction factors in fissured clays. For additional information see X.3.6.
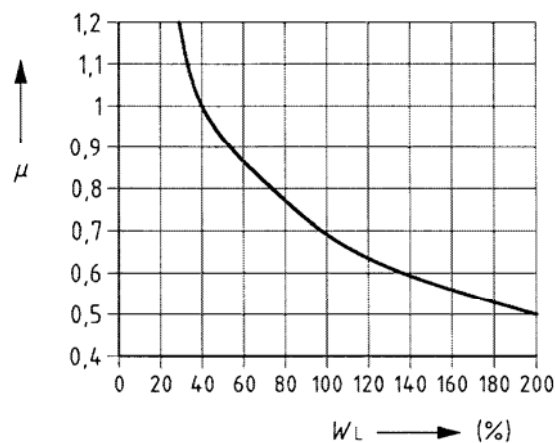
Key

w_L is the liquid limit

Figure I.1 — An example of correction factors for c_{fv} based on the liquid limit for normally consolidated clays

NOTE   Figure I.1 was published by Larsson et al. (1984). For additional information see X.3.6
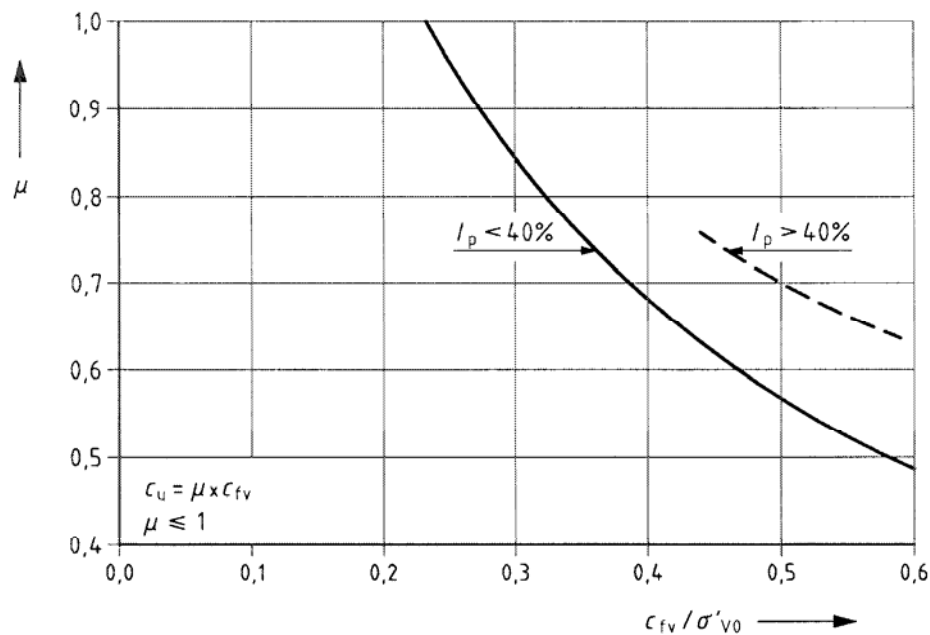
Figure I.2 — Examples of correction factors for c_{fv} based on plasticity index and effective vertical stress (σ'_{v0}) for over-consolidated clays

I.3 Example of the determination of the correction factor μ on Atterberg limits and the state of consolidation

(1) This correction is linked to the plasticity index (I_P) and the effective vertical stress (σ'_{v0}) in the ground. Sample curves are presented in Figure I.2.

NOTE   Figure I.2 was published by Aas (1979). For additional information see X.3.6

I.4 Example of the determination of the correction factor μ based on Atterberg limits and state of consolidation

(1) This procedure has been elaborated in order to take into account the effect of over-consolidation.

(2) An estimate is first made of whether the clay is over-consolidated or not, using the relationship shown in Figure I.3 (relationship between the quotient of measured shear strength (c_{fv}) by the field vane test to the effective stress (σ'_{v0}) and the plasticity index (I_P) for clays). If the corresponding parameters fall between the curves for "young" and "aged", the clays are considered normally-consolidated (NC), whereas clays falling above the curve "Aged" are considered over-consolidated (OC).

(3) Normally-consolidated soils are then corrected according to the curve marked NC in Figure I.4 and over-consolidated soils are corrected according to the curve marked OC.
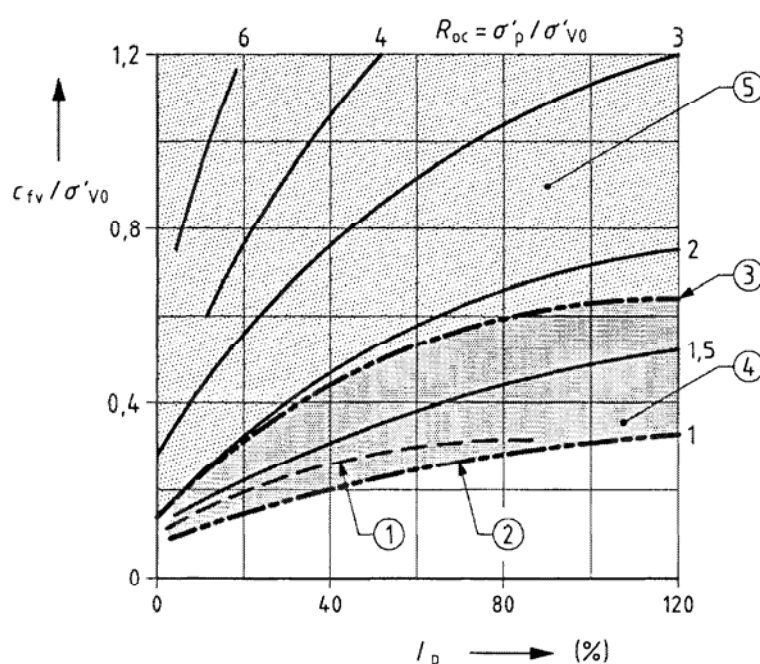
Key

1   curve of Fig. I.2
2   lower limit of young clay
3   upper limit of young clay; lower limit of aged clay
4   range of normally consolidated clays (NC)
5   range of over-consolidated clays (OC)

Figure I.3 — Diagram for separating normally-consolidated and over-consolidated clays

NOTE   This example was published by Aas et al. (1986). For additional information see X.3.6
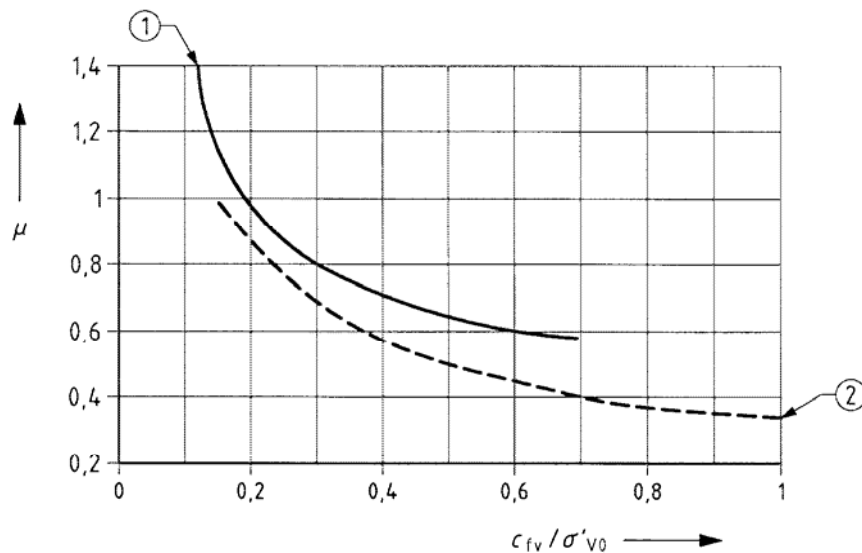
Key

1   normally consolidated (NC),
2   over-consolidated (OC)

Figure I.4 — Correction factors for normally-consolidated and over-consolidated clays

NOTE   For additional information see X.3.6

I.5 Example of the determination of the correction factor μ based on Atterberg limits and the state of consolidation

(1) This procedure has also been presented in order to take into account the effect of over-consolidation.

(2) The correction factor (μ) for normally-consolidated and slightly over-consolidated clays can be determined as

$$\mu = \left(\frac{0,43}{w_L} \right)^{0,45} \geq 0,5$$

where
w_L is the liquid limit (see Figure I.1).

(3) In clays with a higher over-consolidation ratio than 1,3, the following correction factor (μ) can be applied:

$$\boxed{\text{AC}_1} \; \mu = \left(\frac{0,43}{w_L} \right)^{0,45} \times \left(\frac{R_{OC}}{1,3} \right)^{-0,15} \boxed{\text{AC}_1}$$

where
R_{OC} is the over-consolidation ratio.

NOTE   This equation stems from Larsson and Åhnberg, (2003). For additional information see X.3.6

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(4) If the over-consolidation ratio has not been determined, it can be estimated empirically from the relation $c_{fv} = 0.45 \times w_L \times \sigma'_p$. The correction factor (μ) then becomes:

$$\boxed{\text{AC}_1}\!\!\!\rangle \quad \mu = \left(\frac{0.43}{w_L}\right)^{0.45} \times \left(\frac{c_{fv}}{0.585w_L \times \sigma'_{v0}}\right)^{-0.15} \quad \langle\!\!\!\boxed{\text{AC}_1}$$

$\langle\!\!\!\boxed{\text{AC}_1}\!\!\!\rangle$ NOTE   The equation $c_{fv} = 0.45 \times w_l \times \sigma'_p$ stems from Hansbo (1957). For additional information see X.3.6. $\langle\!\!\!\boxed{\text{AC}_1}$

Annex J
(Informative)

Flat dilatometer test (DMT)

(1) This Annex gives an example of correlations between E_{oed} and DMT results. These correlations may be used to determine the value of the one-dimensional tangent modulus ($E_{oed} = d\sigma'/d\varepsilon$) from results of DMT tests, through:

$$E_{oed} = R_M \times E_{DMT}$$

in which R_M is estimated either on the basis of local experience or using the following relationships:

if $I_{DMT} \leq 0,6$; then $R_M = 0,14 + 2,36 \lg K_{DMT}$

if $0,6 < I_{DMT} < 3,0$; then $R_M = R_{M0} + (2,5 - R_{M0}) \lg K_{DMT}$, in which $R_{M0} = 0,14 + 0,15 (I_{DMT} - 0,6)$

⟨AC1⟩ if $I_{DMT} \geq 3$; then $R_M = 0,5 + 2 \lg K_{DMT}$ ⟨AC1⟩

if $K_{DMT} > 10$; then $R_M = 0,32 + 2,18 \lg K_{DMT}$

if values of $R_M < 0,85$ are obtained in the above relationships, R_M is taken to be equal to 0,85.

where
I_{DMT} is material index from the flat dilatometer index
K_{DMT} is the horizontal stress index from the flat dilatometer test

NOTE   This example was published by Marchetti (2001). For additional information and design examples, see X.3.7.

Annex K

(informative)

Plate loading test (PLT)

K.1 Example of deriving the value of undrained shear strength

(1) This is an example of deriving the undrained shear strength (c_u), which can be obtained using the following equation:

$$c_u = \frac{p_u - (\gamma \times z)}{N_c}$$

where
p_u is the ultimate contact pressure from the PLT results;
$\gamma \times z$ is the total stress (density times depth) at test level when the test is conducted in a borehole with a diameter smaller than three times the diameter or width of the plate;
N_c is the bearing capacity factor; for circular plates:
     $N_c = 6$ (typically for PLT on the subsoil surface);
     $N_c = 9$ (typically for PLT in boreholes of depths greater than four times the diameter or width of the plate).

NOTE   This example was published by Marsland (1972). For additional information and examples, see X.3.8.

K.2 Example of deriving the value of the plate settlement modulus

(1) This is an example of deriving the plate settlement modulus E_{PLT} (secant modulus).

(2) For loading tests made at ground level or in an excavation where the bottom width/diameter is at least five times the plate diameter, the plate settlement modulus (E_{PLT}) may be calculated from the general equation:

$$E_{PLT} = \frac{\Delta p}{\Delta s} \times \frac{\pi b}{4} \left(1 - \nu^2\right)$$

where
Δp is the selected range of applied contact pressure considered;
Δs is the change in total settlement for the corresponding change in the applied contact pressure Δp including creep settlements;
b is the diameter of the plate;
ν is Poisson's ratio for the conditions of the test.

(3) If not determined in other ways, ν is equal to 0,5 for undrained conditions in fine soil and 0,3 for coarse soil.

(4) If the test is made at the base of a borehole, the value of E_{PLT} may be calculated from the equation:

$$E_{PLT} = \frac{\Delta p}{\Delta s} \times \frac{\pi b}{4}(1 - v^2)C_z$$

where

C_z is a depth correction factor; it is defined as the ratio of the depth load to settlement of the corresponding surface load.; an example for suggested values is given in Figure K.1.
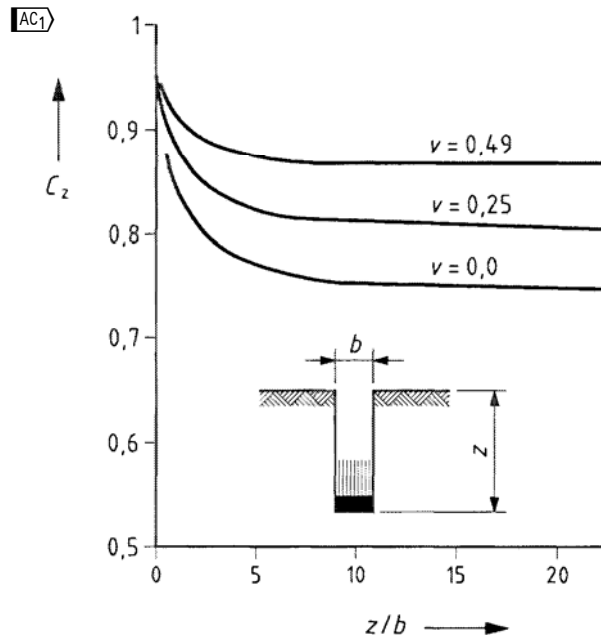
Figure K.1 — Depth correction factor C_z as a function of plate diameter b and depth z for PLT results obtained with a uniform circular load at the base of an unlined shaft ⟨AC1⟩

NOTE   This example was published by Burland (1969). For additional information and examples, see X.3.8.

K.3 Example of deriving the value of coefficient of sub-grade reaction

(1) This is an example of deriving the coefficient of sub-grade reaction (k_s) which may be calculated from the equation:

$$k_s = \frac{\Delta p}{\Delta s}$$

where

Δp is the selected range of applied contact pressure considered;

Δs is the change in settlement for the corresponding change in applied contact pressure Δp including creep settlements.

(2) The dimensions of the loading plate should be stated, when calculating values of k_s.

NOTE   This example was published by Bergdahl (1993). For additional information see X.3.8.

K.4 Example of a method to calculate the settlement of spread foundations in sand

(1) This is an example of deriving settlements directly. The settlements of a footing in sand may be derived empirically according to the relations given in Figure K.3, if the ground beneath the footing to a depth larger than two times the width is the same as the ground beneath the plate (see Figure K.2).

Key

b_1 width of the test plate
b width of the foundation
p is the load
s_1 is settlement measured in PLT
s is predicted settlement for the footing
1   test plate
2   footing
3   influenced zones
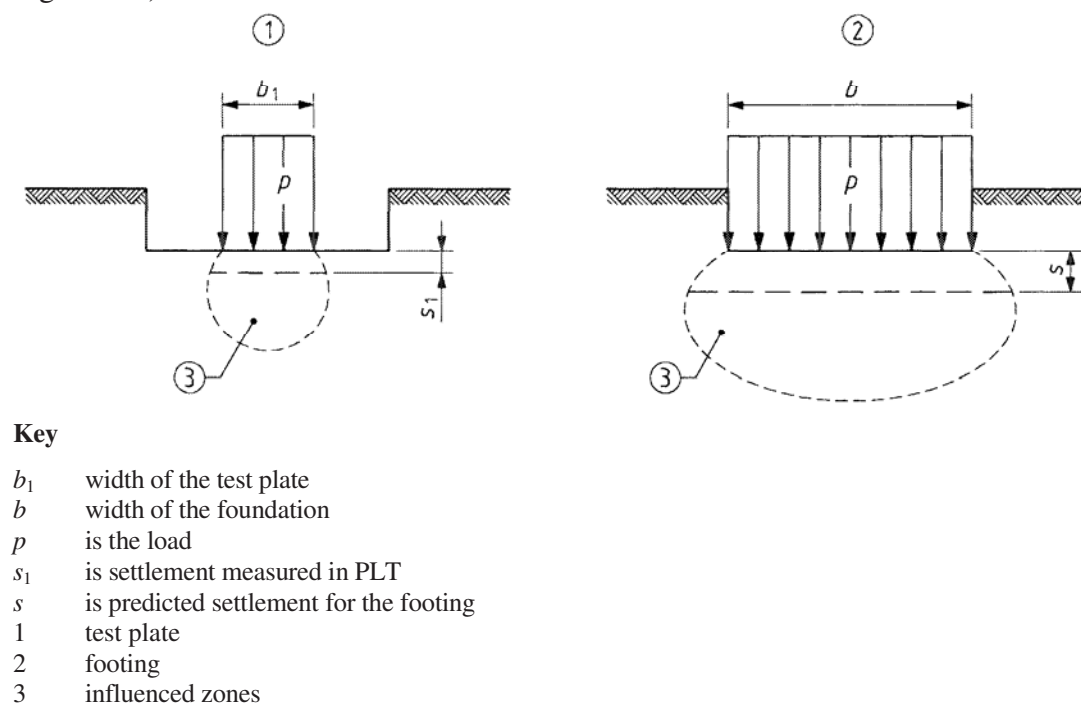
Figure K.2 — Influenced area beneath a test plate and a footing
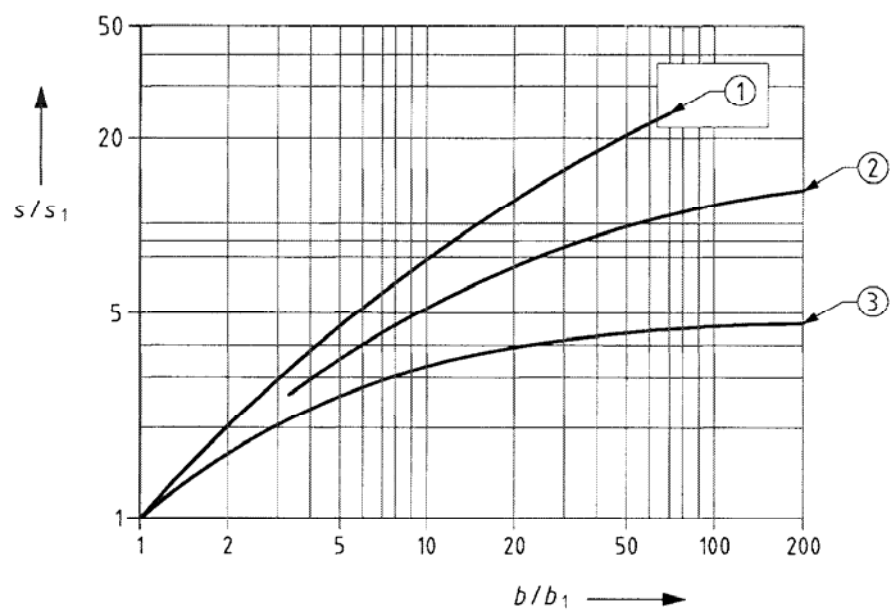
Key

b/b_1	is width ratio
s/s_1	is settlement ratio
1	loose
2	medium dense
3	dense

Figure K.3 — Graph for calculations of settlement based on plate loading tests

NOTE   This example was published by Bergdahl et al (1993). For additional information and examples, see X.3.8.

Annex L

(Informative)

Detailed information on preparation of soil specimens for testing

L.1 Introduction

(1) Detailed procedures are in the scope of CEN/TC 341, based on the test procedures, recommended by the European Technical Committee 5 on "Laboratory Testing" (ETC 5) of the International Society of Soil Mechanics and Geotechnical Engineering. The main requirements are given in this annex.

L.2 Preparation of disturbed soil for testing

L.2.1 Drying of soil

(1) Normally soil should not be dried before testing, unless otherwise specified, but should be used in its natural state. When drying of soil is necessary, one of the following methods should be used:
— oven drying to constant mass in ventilated oven at a temperature of (105 ± 5) °C;
— oven drying in ventilated oven at a specified temperature less than 100 °C (i.e. partial drying, since drying at a lower temperature needs not be complete);
— air drying (partial) by exposure to air at room temperature, with or without a fan.

L.2.2 Disaggregating

(1) The extent of disaggregating to be applied, and the treatment of any remaining cemented material, should be related to the specific requirements and conditions, and should be specified. In particular, the disaggregating and the treatment should be done at the natural water content of the soil.

(2) Aggregations of particles should be broken down in such a way as to avoid crushing of individual particles. The action should be no more severe than that applied by a rubber-headed pestle. Special care is necessary when the soil particles are friable. If a large quantity of soil is to be prepared, disaggregating should be done in batches.

L.2.3 Subdividing

(1) Disaggregated soil should be mixed thoroughly before subdividing. The subdivision process should be repeated until representative samples of the specified minimum masses are obtained for use as test specimens.

L.2.4 Mass of disturbed soil for testing

(1) The minimum masses of disturbed soil required for testing are summarised in Table L.1. Where the minimum mass depends on the size of the largest particles present in significant quantity, this is related to the minimum mass required for sieving (denoted by "MMS") which is given in Table L.2.

(2) The required mass listed in Table L.1 allows for the preparation of one test specimen, with some allowance for wastage but not for the inclusion of over-size particles. When only the fine fraction of soil is required for testing, the prepared sample of the original soil should be large enough to provide the specified mass of the desired fraction.

(3) When it is necessary to remove large particles from the initial sample in order to prepare test specimens, the size range and the proportion by dry mass of the oversize material removed should be recorded.

Table L.1 — Mass of soil required for tests on disturbed samples

Test	Initial mass required		Minimum mass of prepared test specimen			
			Clay and silt	Sand	Gravelly soil	
Water content	At least twice specimen mass		30 g	100 g	D =2 mm – 10 mm MMS	$D > 10$ mm $0,3 \times$ MMS, min 500 g
Particle density	100 g		10g (particle size < 4 mm)			
Grain size						
Sieving	$2 \times$ MMS		MMS			
Sedimentation						
Hydrometer	250 g		50 g	100 g		
Pipette	100 g		12 g	30 g		
Consistency limits	500 g		300 g (particle size < 0,4 mm)			
Density index	8 kg		[a]			
Dispersibility	400 g		[a]			
Compaction	S	NS	[a]			
"Proctor" mould	25 kg	10 kg				
"CBR" mould	80 kg	50 kg				
CBR	6 kg		[a]			
Permeability [b]			[a]			
Diameter						
100 mm	4 kg					
75 mm	3 kg					
50 mm	500 g					
38 mm	250 g					

Notation:
D   Largest particle diameter in significant proportion (10 % or more by dry mass)
MMS   Minimum mass to be taken for sieving (see Table L.2)
NS   Soil particles not susceptible to crushing
S   Soil particles susceptible to crushing during compaction
[a]   Mass of specimen depends on soil behaviour during test
[b]   Permeability specimens with height equal to twice the diameter

L.2.5 Preparation of soil for compaction

(1) Soil that is to be used for compaction-related tests should not be allowed to dry. If it is necessary to reduce the water content of the soil, this should be done by air drying.

(2) The upper limit of allowable particle sizes depends upon the size of the mould to be used. Particles larger than the sizes stated below should be removed before preparing the soil for testing (see Table L.3).

L.3 Preparation of undisturbed specimens

(1) The method of preparation of test specimens from undisturbed samples of soil depends on the type of sample and the type of specimen to be prepared.

(2) The approximate mass of soil required for typical laboratory test specimens is given in Table L.4. The stated mass is sufficient for one test specimen with some allowance for wastage due to trimming.

L.4 Preparation of re-compacted specimens

L.4.1 General requirements

(1) Disturbed soil may be re-compacted to form test specimens in accordance with either of the following criteria:
−   compaction using a specified compactive effort at a specified water content;
−   achieving a specified dry density at a specified water content.

Table L.2 — Minimum mass for sieving

Largest particle diameter (*D*) mm	Minimum mass for sieving (MMS) kg
75	120
63	70
45	25
37,5	15
31,5	10
22,4	4
20	2
16	1,5
Largest particle diameter (*D*) mm	**Minimum mass for sieving (MMS) g**
11,2	600
10	500
8	400
5,6	250
4	200
2,8	150
≤2	100

Table L.3 — Allowable size of particles for compaction tests

Type of test	Maximum size of particle
Compaction — in one-litre mould — in CBR mould	20 mm 37,5 mm
CBR determination	20 mm

Table L.4 — Mass of soil required for tests on undisturbed samples

Type of test	Specimen dimensions		Minimum mass required
	Diameter mm	**Height** mm	g
Oedometer	50 75 100	20 20 20	90 200 350
Compression [b] -Unconfined -Unconsolidated-undrained -Triaxial compression test	35 38 50 70 100 150	70 76 100 140 200 300	150 200 450 1 200 3 500 12 000
Shear box	Planar size 60×60 100×100 300×300	 20 20 150	 150 450 30 000
Density Largest particle size [a]	Largest particle size $D = 5,6$ mm $D = 8$ mm $D = 10$ mm $D > 10$ mm		 125 300 500 1,4 (MMS) [c]
a D is the largest particle diameter in significant proportion (10 % or more by mass).			
b Specimen dimensions and minimum required volume apply to all three tests			
c MMS is the minimum mass to be taken for sieving, as specified in Table L.2.			

(2) Clay soil that is to be re-compacted to form test specimens should not be allowed to dry. If it is necessary to reduce the water content of the soil, this should be done by air drying. If it is necessary to add water in order to increase the water content, the water should be well mixed in and the soil should be allowed to stand in a sealed container for at least 24 hours before use.

(3) The soil should be broken down before re-compaction.

(4) The upper limit of allowable particle sizes depends upon the size of the test specimen to be formed. Particles larger than the sizes given in Table L.5 should be removed before preparing the soil for re-compaction.

(5) The particle size distribution of a re-compacted specimen should be checked before and after compaction.

Table L.5 — Allowable particle size as a function of size of test specimen

Type of test specimen	Maximum size of particle
Oedometer consolidation	$H/5$ [a]
Direct shear (shear box)	$H/10$
Compressive strength (cylindrical specimen with H/d of about 2)	$d/5$ [b]
Permeability	$d/12$
[a] H = height of specimen, [b] d = diameter of specimen	

L.4.2 Re-compacted sample larger than test specimen

(1) When preparing specimens for oedometer consolidation, direct shear or compressive strength tests, the soil should normally be compacted in the specified manner into a suitable mould that is of a larger size than the desired test specimen. The compacted sample should then be extruded from the mould and the test specimen should be prepared using the procedures described for undisturbed samples.

NOTE   The method is not suitable for granular soil.

(2) Specimens for permeability tests may be compacted directly into the mould or container in which the test is performed.

(3) For compaction using a specified effort, the compactive effort applied should normally correspond to that used in one of the two types of compaction tests specified for the compaction test (see 5.10 and Annex R). Compaction should be applied in layers, and the top of each layer should be lightly scarified before adding the next one.

(4) To obtain a specified density, the soil may either be compacted dynamically or compressed under a static load. Weighings and volume measurements should be made after placing each layer to ensure that the desired density will be achieved. Preliminary trials may be desirable to establish the appropriate method.

(5) If clay is present in the soil, the compacted sample should be sealed and stored for a curing period of at least 24 h before extrusion to form test specimens.

L.4.3 Re-compaction of test specimen

(1) For the preparation of small test specimens for direct shear, oedometer or compressive strength, the soil should be tamped, kneaded or compacted into the appropriate mould, ring or tube. A suitable hand rammer, the Harvard compaction apparatus, or a kneading action may be used. Care should be taken to avoid the formation of cavities within the specimen. The exact procedure required to obtain the desired density or compactive effort should first be determined by trial. Details should be recorded so that the procedure can be repeated to provide a number of specimens of consistent properties.

(2) Compaction of cylindrical test specimens of 100 mm diameter or more may be carried out using a compaction rammer. The number of layers and number of blows per layer should be specified.

(3) If clay is present in the soil, the compacted specimen should be sealed and stored for a curing period of at least 24 h before use, to allow for dissipation of excess pore water pressures.

L.4.4 Re-saturation

(1) A re-compacted specimen will invariably be initially unsaturated. Re-saturation will normally be required before testing, and this should be carried out by using one of the recognised saturation methods given in the test procedures for shear strength or compressibility tests. Full saturation should be confirmed by checking the B value, if applicable.

L.4.5 Remoulded test specimen

(1) Remoulding can be achieved by sealing the soil in a plastic bag where it is squeezed and kneaded with the fingers for several minutes. A remoulded test specimen is formed by working the soil into the appropriate mould, e.g. by using a tamping rod. This operation should be carried out as quickly as possible to avoid change of water content, and without entrapping air. The specimen should then be extruded and trimmed.

L.5 Preparation of reconstituted specimens

L.5.1 Preparation of slurry

(1) The soil should be thoroughly mixed with water to form an homogeneous slurry with a water content above the liquid limit. Preparing the slurry should preferably start from the natural water content without drying of the soil. Drying of the soil and grinding it to a powder may change its properties. If necessary, coarser particles can be removed by wet sieving using an appropriate sieve. The mixing water may be either distilled or de-ionized, or of the appropriate chemistry. The slurry should be fluid enough to be poured; a water content about twice the liquid limit is usually satisfactory.

L.5.2 Consolidation

(1) The cell in which the sample is consolidated should be large enough to provide a test specimen, or a sample for trimming, to the required size after consolidation. Provision should be made for drainage of the sample, without allowing soil particles to escape.

(2) After pouring the slurry into the mould, initial consolidation should be applied under the weight of the top plate only, until the specimen ends are stiffened enough to prevent loss of material under further loading. The vertical stress applied for consolidation should be sufficient to enable the sample to be handled when consolidated, and should be maintained for long enough to ensure that consolidation is substantially complete.

L.5.3 Specimen preparation

(1) The consolidated sample should be extruded from the cell and trimmed as necessary for the preparation of a test specimen or specimens.

(2) If one-dimensional consolidation tests are to be performed on the reconstituted soil, they may be carried out in the cell in which it has been consolidated from slurry.

Annex M

(Informative)

Detailed information on tests for classification, identification and description of soil

M.1 Checklists for classification testing

(1) The number of specimens to be tested depends on the variability of the soil and the amount of experience with the soil and to a lesser extent than other soil tests on the geotechnical problem. Table M.1 gives guidance on the number of classification tests.

(2) Table M.2 presents a checklist for each of soil classification tests included in the document.

Table M.1 — Classification tests. Suggested minimum number of samples to be tested in one soil stratum

Classification test	Comparable experience	
	No	Yes
Particle size distribution	4–6	2-4
Water content	All samples of Quality Class 1 to 3	
Strength index test	All samples of Quality Class 1	
Consistency limits (Atterberg limits)	3–5	1-3
Loss on ignition (for organic and clay soil)	3–5	1-3
Bulk density	Every element test	
Density index	As appropriate	
Particle density	2	1
Carbonate content	As appropriate	
Sulfate content	As appropriate	
pH	As appropriate	
Chloride content	As appropriate	
Soil dispersibility	As appropriate	
Frost susceptibility	As appropriate	

Table M.2 — Checklist for soil classification tests

Classification test	Checklist
Water content	Check storage method of samples
	Coordinate testing programme with other classification tests
	Standard oven-drying method not appropriate for gypsum, organic soil; precautions may be needed
	Report presence of gypsum, organic soil
	For coarse soil, correction of measured water content may be needed
	Correction needed for saline soil

Table M.2 (*continued*)

Classification test	Checklist
Bulk density	Test method needs to be selected Check sampling and handling methods used For large earthwork projects, method may need to be adapted, or use field method
	For sands and gravels, correction of measured density may be needed;
Particle density	Sample preparation (oven-drying versus moist specimen) can influence results Check whether material can have enclosed pores; for such material, special techniques might be appropriate Report if material has enclosed pores If results fall outside the range of typical values, consider additional determinations; mineralogy and organic content will affect result
Particle size analysis	Selection of test method depends on particle size and gradation Carbonates and organic matter influence test results; for such materials, remove carbonates or organic matter if appropriate, or adapt testing method Check that correct quartering is used (particle size and sample representativeness)
Consistency limits (Atterberg limits)	Selection of test method for liquid limit; several methods are acceptable, but fall cone method is recommended Check storage method of samples Check specimen preparation, especially homogenisation and mixing Check whether drying has been used Drying can influence results dramatically, and should be avoided in oven Soil that oxidise should be tested quickly Results need not be reliable for thixotropic soil
Density index for granular soil	Check storage method of samples Select test type to be used Results are very dependent on procedure used Prepared specimens have high degree of non-uniformity
Soil dispersibility	Need to consider specifying different compaction conditions for specimens Avoid drying of the specimen before testing Need to select test procedures to use Need to run classification tests in addition
Frost susceptibility	—

NOTE   Examples of methods for classification, identification and description of soil are given in the documents listed in X.4.1.

M.2 Water content determination

M.2.1 Evaluation of test results

(1) If the water in the soil is saline, the dissolved salts will remain in the soil after drying and may give an incorrect water content result. A more appropriate value may be the fluid content i.e. the mass of fluid (water plus salts) per unit mass of dry soil.

(2) For a coarse soil, the water content obtained in the laboratory on a sample for which the maximum grain diameter is limited by the sample size, may differ from the in place water content. In such a case, the water content should be corrected as a function of the percentage of grains that are larger than the maximum grain diameter.

NOTE   Examples of methods for determination of the water content of soil are given in the documents listed in X.4.1.2.

M.3 Bulk density determination

M.3.1 Test procedures

(1) The linear measurement method is appropriate for fine soil only. For coarse soil, density can normally be determined with sufficient accuracy from in-situ tests, and more accurately from measurements on a frozen "undisturbed" sample.

(2) Table M.3 presents a guideline for the minimum number of tests required for one stratum of clay or silty soil. In the table, a specification of only one test represents a verification of the existing knowledge.

Table M.3 — Density tests. Minimum number of soil specimens to be tested for one soil stratum

Variability in measured density	Comparable experience		
	None	**Medium**	**Extensive**
Range of measured density $\geq 0{,}02$ Mg/m^3	4	3	2
Range of measured density $\leq 0{,}02$ Mg/m^3	3	2	1

NOTE   Examples of methods for determination of the bulk density of soil are given in the documents listed in X.4.1.3.

M.3.2 Evaluation of test results

(1) The test results should be checked by calculating the degree of saturation, which should not exceed 100 %.

(2) For a coarse soil, the density of dry soil obtained in the laboratory on a sample for which the maximum grain diameter is limited by the sample size, may differ from the in place density of dry soil. In such a case, the density of dry soil should be corrected as a function of the percentage of grains that are larger than the maximum grain diameter.

M.4 Particle density determination

(1) The volume of material necessary for the determination of soil particle density is very small (minimum of 10 g with particle size less than 4 mm). The specimen is usually extracted from a specimen used for another laboratory test.

(2) In porous materials with enclosed pores, the particles only have an apparent density. The density of solid material may be found by finely grinding of the specimens and the amount of closed pores by using techniques where the volume of open pores are found using water saturation of these or gas pressure techniques in specialised laboratories. The density of solid particles should then be measured in the laboratory using a special technique.

(3) In the case of soil with organic materials, the laboratory testing should follow special procedures. Otherwise, the measured values should be used with caution.

(4) Modern methods such as the He-pycnometer may be applied. The methods should be calibrated against one of the methods more commonly used, for example the methods described in the documents listed in X.4.1.4.

NOTE   Examples of methods for determination of the particle density of soil are given in the documents listed in X.4.1.4.

M.5 Particle size analysis

(1) For coarse soil (predominantly gravel and/or sand sizes), the particle size distribution of soil is determined by sieving after washing, and sedimentation is not usually necessary. For fine soil (predominantly silt and/or clay sizes), the sedimentation procedure is used, including sieving of any sand-sized particles. For mixed soil (containing all size ranges), both sieving and sedimentation procedures are used.

(2) Special care should be taken for tests on clays and organic soil. For example clay particles may have a cementing effect which can become irreversible during drying at 105 °C, organic matter becomes partly oxidised during drying at 105 °C.

(3) Modern methods that incorporate detection systems using X-rays, laser beams, density measurements, and particle counters may also be applied. They should be calibrated against the methods suggested in (2).

NOTE   Examples of the methods for the particle size analysis of soil are given in the documents listed in X.4.1.5.

M.6 Density index test of granular soil

(1) The recommended minimum number of soil specimens to be tested for one soil stratum is two for the determination of the maximum density and three for the determination of the minimum density.

NOTE   Examples of methods for the determination of the density index of granular soil are given in the documents listed in X.4.1.7.

M.7 Soil dispersibility determination

M.7.1 General

(1) Some natural clayey soils disperse rapidly in slow moving water by colloidal erosion along cracks or other flow channels. Such soil is highly susceptible to erosion and piping. The tendency for dispersive erosion in a soil depends upon the mineralogy and chemistry of the clay, and the dissolved salts in the soil pore water and the eroding water. Dispersive clays are usually high sodium content soil.

M.7.2 Test procedures for all tests

(1) Dispersibility tests are not applicable to soil with clay content of less than 10 % and with a plasticity index less than or equal to 4 %.

(2) The recommended minimum number of soil specimens to be tested for one soil stratum is two for the pinhole test, two for the double hydrometer test, two for the soluble salts in pore water test and three for the crumb test. The specification of the number of tests to be carried out should be based on engineering judgement.

NOTE   Examples of methods for the testing of soil dispersibility are given in the documents listed in X.4.1.8.

M.7.3 Pinhole test

(1) It is recommended to follow the literature listed in X.4.1.8, except that:
−   the specimen should be compacted in a Harvard miniature mould at a water content close to the plastic limit;
−   five layers should be used for the total specimen height of (38 ± 2) mm;
−   a constant compaction effort on each layer should be applied such that the resulting dry density of the sample is equal to 95 % of the maximum dry density determined in the laboratory from standard compaction test(s).

(2) The presentation of the results should include:
−   the results of classification tests;
−   the density of tested specimen;
−   the water heads used and testing time under each head;
−   the flow rates through specimen;
−   the cloudiness of flowing fluid at end of test;
−   the hole size and shape after test;
−   the classification of soil according to standard reference.

NOTE   Examples of methods for the pinhole test are given in the documents listed in X.4.1.8

M.7.4 Double hydrometer test

(1) The presentation of the results should include the grain size curves obtained with and without a dispersant solution and mechanical shaking/stirring, and the percentage dispersion.

M.7.5 Crumb test

(1) Presentation of the results should include the soil classification as dispersive or non-dispersive and details on the reagent used.

NOTE   Examples of such classifications are given in the documents listed in X.4.1.8.

M.7.6 Sodium and dissolved salts in saturation extract

(1) The report should present the exchangeable sodium percentage obtained.

NOTE   Examples of test procedures for the determination of soluble salts in the pore water, are given in the documents listed in X.4.1.8.

M.8 Frost susceptibility determination

M.8.1 Test procedures

(1) A sample in its natural state can be obtained unfrozen in soft clayey and silty soil or frozen in clay, silt and sand (without gravel). If the sample size is not directly suitable for testing, the sample can be reshaped, if done carefully.

(2) The sample that will be re-compacted can be strongly remoulded as long as the grain size distribution has not been modified by the sampling operation.

(3) The diameter of a specimen in its natural state should be at least five times the maximum grain size, and no less than 75 mm. For a reconstituted specimen, a minimum diameter of 100 mm should be used.

(4) Both natural specimens and reconstituted specimens may be saturated with back-pressure prior to the frost heave test.

(5) If a CBR test is needed, the test should be carried out on a specimen compacted at a water content close to the optimum water content, as determined from the compaction curve of a compaction test.

(6) Generally one CBR test is carried out per sample. However, several tests should be carried out to assess the influence of for example water content variations and compactive force.

NOTE   Examples of test procedures for the determination of frost susceptibility of soil and evaluations based on index tests are given in the documents listed in X.4.1.9 and X.5.

M.8.2 Evaluation of test results

(1) A soil is deemed to be frost-susceptible if it exhibits segregational heaving in the frost heave test in the laboratory.

(2) The degree of frost action in clayey soils with low permeability is affected by the length of the winter season, i.e. the altitude and the latitude of the site considered. For these soils, the longer the winter, the more severe the frost action. This should be taken into account in northern and alpine countries.

Annex N

(Informative)

Detailed information on chemical testing of soil

N.1 General

N.1.1 Test procedures

(1) The above routine chemical tests are based on traditional testing methods which are within the capability of many geotechnical laboratories. Chemical tests for the presence of other substances should normally be performed by a specialist chemical laboratory.

(2) One hundred grams of dry soil is sufficient for most chemical testing. Usually a much larger sample of dried soil will be required at the outset, but a very small sample of dried soil is required for the specific testing. Thorough mixing of the initial sample, and correct sub-dividing procedures, are essential.

(3) Storage temperature before testing may influence the biological degradation of organic matter. Whenever possible, the sample material for chemical tests should be kept at a temperature of 5 °C to 10 °C.

(4) Most test methods include a calibration routine using "blind" samples and reference samples. Electrochemical methods like pH have well defined calibration schemes with a number of solutions with known pH.

(5) Special requirements may call for deviations from the standard procedures, including specimen preparation. Any procedural deviations should be clearly reported, including the reasons for the deviations.

NOTE   Examples of test procedures for the five chemical tests dealt with are given in the documents listed in X.4.2. Equivalent methods also exist in other national standards and in textbooks.

N.1.2. Number of tests

(1) The number of tests specified should take into account the fact that the organic content, carbonate content, sulfate content, pH value and chloride content can vary widely even within a geological stratum. Multiple tests on closely-spaced samples may be necessary to define the local variability.

N.2 Organic content determination

N.2.1 Test procedures

(1) The loss on ignition is normally determined on a representative sample of the soil finer than 2 mm as the mass lost by ignition of a prepared specimen at the specified temperature. The organic content is calculated on the assumption that the organic mass is totally burned by the ignition, and that the mass loss is only due to the ignition of the organic matter.

(2) The loss on ignition generally relates to the organic content of soil containing little or no clay and carbonates. For soil with higher percentage of clay and/or carbonate, factors unrelated to organic content could be responsible for the major proportion of the ignition loss.

(3) A drying temperature lower than the usual (105 ± 5) °C is necessary to avoid oxidation of some organic matter during drying. The examples listed in X.4.2.2 specify a drying temperature of (50 ± 2,5) °C, which might not remove all water. Trials may be necessary to establish a suitable drying temperature.

(4) The ignition temperature specified in the examples referred to in (1) is (440 ± 25) °C, but other standards specify temperatures up to 900 °C. Caution should be exercised when specifying an ignition temperature, taking into account the following:
−   some clay minerals can begin to disintegrate at temperatures of about 550 °C;
−   chemically-bound water may vanish at lower test temperatures; for example in some clay minerals, this can start at 200 °C, and gypsum disintegrates at from about 65 °C;
−   sulfide may oxidise, and carbonates disintegrate, in the range 650 °C to 900 °C.
For most purposes, an ignition temperature of 500 °C or 520 °C is appropriate.

(5) The drying and ignition periods should be sufficient to ensure that equilibrium has been achieved. If the period of ignition is less than 3 h, the report should document that constant mass was confirmed by repeated weighing.

NOTE   Examples of test procedures for the determination of organic content are given in the documents listed in X.4.2.2.

N.2.2 Evaluation of test results

(1) The quantity of organic carbon and organic matter can be related to loss on ignition, if the latter is corrected for other expelled constituents.

(2) The organic content can be determined by a direct measurement of the content of organic carbon, by which the errors in the loss on ignition method can be avoided.

N.3 Carbonate content determination

N.3.1 Test procedures

(1) Examples of test procedures for the determination of carbonate content are given in the documents listed in X.4.2.3. For the purpose of this standard, the rapid titration method is the preferred procedure. This method should give results that are accurate enough for soil, provided that care is taken to ensure that the dissolution process is finalised, and that sufficient duplicate tests are performed.

(2) Other examples given in the documents listed in X.4.2.3 determine the carbonate content by measuring the liberated carbon dioxide (CO_2) in a gasometer under controlled temperature and atmospheric pressure.

N.3.2 Evaluation of test results

(1) The carbonate content in percentage carbonate in the sample is expressed as the amount of CO_2. This is formally correct but impractical for design. The results may be given in equivalent calcium carbonate $CaCO_3$, i.e. the carbonate composition for most soil types. The amount of equivalent $CaCO_3$ is obtained from the amount of CO_2 by the equation:

$$CaCO_3 = 2{,}273 \cdot CO_2$$

where
$CaCO_3$ is the $CaCO_3$ content as percentage of dry weight;
CO_2 is the CO_2 content as percentage of dry weight.

N.4 Sulfate content determination

N.4.1 Test Procedures

(1) The gravimetric method for analysis of acid or water extract or groundwater, mentioned here is suggested as the preferred one, unless it can be shown by parallel analysis that an alternative method has equal or better accuracy.

(2) The crystalline form of calcium sulfate, gypsum ($CaSO_4 \cdot 2H_2O$) should be dried out at a temperature of 50 °C. Gypsum-bearing samples begin to lose their water of crystallisation at temperatures higher than about 65 °C, which can give rise to erroneously high measured water contents.

(3) The relationship between SO_3^{2-} and SO_4^{2-} is given by $SO_4^{2-} = 1{,}2\ SO_3^{2-}$, with SO_3^{2-} and SO_4^{2-} contents expressed as a percentage.

NOTE   Examples of test procedures for the determination of sulfate content are given in the documents listed in X.4.2.4.

N.4.2 Evaluation of test results

(1) The interpretation should consider that the solubility of calcium sulfate in water is low, but in geological time, appreciable quantities can dissolve as occurs, for example, in karstic formations. Particular care is needed when results are marginal with respect to classification categories.

(2) The presence of certain other substances (notably sulfides and sesqioxides) can affect the chemical reactions, which then influence the test results. Sulfides in soil can oxidise in the long term to produce additional sulfates.

N.5 pH value determination (acidity and alkalinity)

N.5.1 Test procedures

(1) Several methods are available for determining values of pH. Of these, the electrometric method, which gives a direct reading of pH either in a prepared suspension of soil or in groundwater, is recommended as the definitive method.

N.5.2 Evaluation of test results

(1) Erroneous test results can be caused by:
- omitting or wrong calibration of the pH meter before and after each set of tests;
- inadequate protection of electrodes when the instrument is not in use;
- failure to allow the pH meter to reach a stable condition before taking pH readings;
- contamination due to inadequate washing of containers for sampling groundwater.

NOTE   Examples of test procedure for the determination of the pH value are given in the documents listed in X.4.2.5.

N.6 Chloride content determination

N.6.1 Test procedures

(1) Procedures for determination of chloride content include:
- Mohr's method for water-soluble chlorides;
- Volhard's method for acid-soluble or water-soluble chlorides;
- electrochemical procedures.

(2) The first two methods make use of the exchange reaction between the chlorides and silver nitrate, but different methods of analysis are used. Both methods require careful observation and weighing. The third method is based on the measurement of conductivity in dilutions of the sample with known water content.

(3) The presence of chlorides can be confirmed from a quick qualitative test: take about 5 ml of filtered groundwater, or of 1:1 soil-water extract, in a test-tube. If this is highly alkaline (pH 12–14), add a few drops of nitric acid to acidify it. Add a few drops of 1 % silver nitrate solution. Appreciable turbidity indicates that chlorides are present in a measurable quantity, which can be determined from one of the test procedures.

(4) Volhard's method is the basis of the tests given in 7.2 (water-soluble chlorides) and 7.3 (acid-soluble chlorides) of BS 1377-3:1990 and of the method given in BS 812-118:1988 for mineral aggregates. In principle, an excess of silver nitrate solution is added to the acidified chloride solution and the unreacted portion is back-titrated with potassium thiocyanate, with ferric aluminium used as an indicator.

(5) In Mohr's method, the test solution and a blank for comparison are each titrated with 0,02 N silver nitrate solution, potassium chromate being used as an indicator. This method is preferable for determining chlorides in groundwater.

NOTE   Examples of test procedures are given in the documents listed in X.4.2.6.

N.6.2 Evaluation of test results

(1) The theoretical relationship between salinity expressed as sodium chloride content and chloride content need not hold due to the very mobile nature of the chloride anion.

Annex O

(Informative)

Detailed information on strength index testing of soil

(1) Table O.1 suggests a summary checklist on the test procedures for the strength index tests for soil included in this standard.

Table O.1 — Checklist for strength index tests on clayey soil

Strength index test	Checklist
Any strength index test	Tests provide approximate index of shear strength There is a large uncertainty in the measurements Use results cautiously for non-homogeneous and jointed/slickensided soil All results are affected by the applied testing rate Test repeatability needs to be checked
Laboratory vane	Test provides in addition a measure of sensitivity and remoulded shear strength Check mode of rotation (hand-operated versus motorised) Tests can be performed on extruded samples or in sampling tube
Fall cone	Test can be performed on extruded samples or in sampling tube on intact material. This can be supplemented with a test on remoulded material to establish the sensitivity i.e. the ratio between intact and remoulded strength data Provides in addition a measure of sensitivity on remoulded specimen Check wear on tip of cone Check tip cone angle

NOTE   Examples of test procedures for the laboratory vane and fall cone strength index tests are given in the documents listed in X.4.3.

Annex P
(Informative)

Detailed information on strength testing of soil

P.1 Triaxial compression tests

P.1.1 Number of tests

(1) Table P.1 gives guidelines for the minimum number of tests required as a function of the variability of the soil and existing comparable experience with the type of soil. If only one test set is required, the test is run to provide a verification of existing knowledge. If the new test results do not agree with the existing data, more tests should be run.

(2) The number of tests may be reduced if shear stress data are available from other testing methods, for example, field tests.

NOTE   Examples of test procedures for the triaxial compression tests are given in X.4.4.

Table P.1 — Triaxial compression tests. Recommended minimum number of tests for one soil stratum

Recommended number of tests to determine the effective angle of shearing resistance [a]			
Variability in strength envelope Coefficient of correlation r on regression curve	Comparable experience		
	None	Medium	Extensive
$r \leq 0,95$	4	3	2
$0,95 << r \leq 0,98$	3	2	1
$r \geq 0,98$	2	1	1
Recommended number of tests to determine the undrained shear strength [a]			
Variability in undrained shear strength (for same consolidation stress)	Comparable experience		
	None	Medium	Extensive
Ratio max/min values > 2	6	4	3
1,25 < Ratio max/min value ≤ 2	4	3	2
Ratio max/min value ≤ 1,25	3	2	1
[a] One recommended test means a set of three individual specimens tested at different cell pressures.			

P.1.2 Evaluation of test results

(1) In addition to a factual evaluation, the undrained shear strength should be checked against correlations with the soil type, plasticity index, etc. The undrained shear strength evaluations should be related to the type of test that produced the results.

(2) The angle of shearing resistance should be checked against correlations with the soil type, plasticity, density index, etc. Laboratory and the in-situ stress conditions (e.g. axi-symmetrical versus plane strain conditions) should be carefully considered and in relevant cases, the angle of

shearing resistance should be adjusted. Relations with for example the results of cone penetration tests and existing correlations with angle of shearing resistance should also be included.

P.2 Consolidated direct shear box tests

P.2.1 Test procedures

(1) The direct shear (box or ring shear) test is run preferably for soil and stability conditions where a distinct rupture plane is expected to develop or when the strength characteristics of an interface are to be determined.

(2) Comparative studies show that the test results of direct (translational) shear box and ring shear box tests are in good agreement. The preparation of the specimen is easier in the translational shear box test. The stresses are more homogeneous in the ring shear test but the strains are not uniform. It is easier to produce large strains and thus determine the residual strength of a soil in the ring shear apparatus than in the translational shear box apparatus.

(3) Twice as much material as needed for the number of specimens tested should be taken from the stratum.

NOTE   Examples of procedures for direct shear testing are given in documents listed in X.4.4.4.

P.2.2 Number of tests

(1) Table P.2 gives a guideline for the recommended minimum number of tests required as a function of the variability of the soil and existing comparable experience with the type of soil. The recommendation applies to the case when direct shear tests are used alone to determine the shear strength of a soil stratum.

Table P.2 — Direct shear tests. Recommended minimum number of tests for one soil stratum

Recommended number of tests [a]			
Variability in strength envelope	Comparable experience		
Coefficient of correlation on regression curve	None	Medium	Extensive
Coefficient of correlation < 0,95	4	3	2
0,95 ≤ Coefficient of correlation < 0,98	3	2	2
Coefficient of correlation ≥ 0,98	2	2	1 [b]
[a] One recommended test means a set of three individual specimens tested at different normal stresses.			
[b] A single test and classification tests to verify compatibility with comparable experience. If the test results do not agree with the existing data, additional tests should be run.			

Annex Q

(Informative)

Detailed information on compressibility testing of soil

Q.1 Number of tests

(1) For a soil stratum which contributes significantly to the settlement of a structure, Table Q.1 gives a guideline for the minimum number of oedometer tests required as a function of the variability of the soil and the existing comparable experience with the type of soil.

(2) The number of specimens tested should be increased if the structure is very sensitive to settlements. In Table Q.1, a specification of only one test represents a verification of the existing knowledge. If the new test results do not agree with the existing data, additional tests should be run.

Table Q.1 — Incremental oedometer test. Recommended minimum number of tests for one soil stratum

Variability in oedometer modulus E_{oed} (in the relevant stress range)	Comparable experience		
	None	**Medium**	**Extensive**
Range of values of $E_{oed} \geq 50\ \%$	4	3	2
$\approx 20\ \% <$ Range of values of $E_{oed} < \approx 50\ \%$	3	2	2
Range of values of $E_{oed} < \approx 20\ \%$	2	2	1[a]
[a] One oedometer test and classification tests to verify compatibility with comparable knowledge (see Q.1 (2)).			

Q.2 Evaluation of compressibility characteristics

(1) There are four widely used methods to determine the compressibility of a soil:
−   back-calculations of measured settlements;
−   empirical evaluation of indirect in-situ investigations such as soundings;
−   measurements by in-situ tests, such as plate load and pressuremeter tests;
−   compression tests with soil samples in the laboratory.

(2) Back-calculation from measured settlements under comparable stresses can be a reliable method to assess the compressibility characteristics (layered ground, load redistribution and time effects may be difficult to take into account). For foundations on sand and gravel, field tests such as soundings are often used: these are interpreted empirically, most often based on comparable experience. In cases where sands, coarser soil, silts and clays are expected, a combination of field and laboratory methods is desirable. Laboratory compressibility tests are most reliable for fine and organic soil, when quality class 1 samples are relatively easy to obtain.

NOTE   Examples of test procedures are given in the documents listed in X.4.5.

Annex R

(Informative)

Detailed information on compaction testing of soil

R.1 Test procedures applicable to both test types

(1) The minimum number of soil specimens to be tested for one soil stratum is three. The number of tests specified should be based on engineering judgement.

(2) The number of tests to be carried out should be selected considering the variation of the particle size distribution, the consistency limits and the quantity of material to be compacted. For dams, road construction etc., the number of tests to be run may be found in the relevant standards.

NOTE   Examples of test procedures for compaction testing of soil are given in the documents listed in X.4.6.

R.2 Requirements specific to compaction tests

(1) The most frequently used compaction tests are the Standard and the Modified (Proctor) Compaction Tests.

(2) Some highly permeable soils such as clean gravels, uniformly graded and coarse clean sands do not yield a well defined maximum density. Therefore optimum water content might be difficult to obtain.

(3) For stiff fine soil, suggested methods are to shred the soil so that it can pass through a 5 mm test sieve, or to chop it into pieces to pass through a 20 mm test sieve.

(4) For stiff fine soil which needs to be shredded or chopped into small lumps, the results of a compaction tests depend on the size of the resulting pieces. The densities obtained from the test will not necessarily be directly related to densities obtained in-situ.

(5) For soil not susceptible to crushing, only one sample may be used for testing. The sample can be used several times after increasing progressively the amount of water. The departure from the common procedure should be mentioned in the report.

(6) For soil containing particles that are susceptible to crushing, separate batches at different water contents should be prepared.

R.3 Requirements specific to California Bearing Ratio (CBR) test

(1) In-situ tests may be carried out, but the laboratory test is the definitive procedure.

(2) Tests may be carried out on either undisturbed or re-compacted material.

(3) The moisture content of the soil should be chosen to represent the design conditions for which the test results are required.

(4) The CBR test should be carried out on material passing through the 20 mm test sieve. If the soil contains particles retained on the 20 mm sieve, these particles should be removed and weighed before preparing the test specimen. If the fraction retained on the 20 mm sieve is greater than 25 % by mass of the fraction passing through the 20 mm sieve, the CBR test is not applicable.

(5) Where a range of water contents is to be investigated, water should be added to or removed from the natural soil after disaggregation. The sample should not be allowed to dry.

NOTE   Examples of test procedures are given in the documents listed in X.4.2.

Annex S
(Informative)

Detailed information on permeability testing of soil

S.1 Test procedures

(1) Twice as much material as needed for the number of specimens to be tested should be taken from the stratum.

(2) The specimens to be tested should be selected to represent the extremes in relevant soil properties, i.e. composition, density index, void ratio, etc.

(3) As a guideline, the hydraulic gradient in clays and silts should be less than 30 and less than 10 in sand.

(4) Depending on soil type and required accuracy of coefficient of permeability, the required degree of saturation in the permeability test should be considered.

NOTE   Examples of procedures for testing the permeability of soil are given in the documents listed in X.4.7.

S.2 Number of tests

(1) Table S.1 gives a guideline for the minimum number of tests required as function of the variability of the soil and existing comparable experience with the type of soil.

Table S.1 — Permeability tests. Recommended minimum number of soil specimens to be tested for one soil stratum.

Variability in measured coefficient of permeability (k)	Comparable experience		
	None	**Medium**	**Extensive**
$k_{max}/k_{min} > 100$	5	4	3
$10 < k_{max}/k_{min} \leq 100$	5	3	2
$k_{max}/k_{min} \leq 10$	3	2	1[a]
[a] A single test and classification tests to verify compatibility with existing knowledge.			

(2) In Table S.1, a specification of only one test represents a verification of the existing knowledge. If the test results do not agree with the existing data, additional tests should be run.

S.3 Evaluation of test results

(1) There are four widely used methods to determine the coefficient of permeability (hydraulic conductivity):
−   field tests, such as pumping and borehole permeability tests;
−   empirical correlations with grain size distribution;
−   evaluation from an oedometer test;
−   permeability tests on soil specimens in the laboratory.

The evaluation of the coefficient of permeability can be optimised by a combination of these methods.

(2) Even in a homogeneous soil stratum, there can be a large variation in the coefficient of permeability due to small changes in stresses, void ratio, structure, particle size and bedding. The most reliable method to obtain a value of the coefficient of permeability is a field testing method.

(3) Even in a homogeneous soil stratum, the coefficient of permeability of a soil layer should be described by upper and lower limit values.

(4) For silts and clays, the derivation of the coefficient of permeability from incremental oedometer test results only gives an approximate estimate. Constant rate of strain oedometer tests provide a more direct measure of the permeability.

(5) In homogeneous sand, the coefficient of permeability may be assessed in a reasonably accurate manner from correlations with the grain size distribution.

(6) For clay, silt and organic soil where undisturbed samples of high quality can be obtained, laboratory tests may give reliable test results. The representativeness of the specimens tested should be carefully checked.

(7) For some types of soil, the degree of saturation may influence the coefficient of permeability up to as much as three orders of magnitude.

(8) The chemistry of the permeant may change the coefficient of permeability by several orders of magnitude.

Annex T

(Informative)

Preparation of specimen for testing on rock material

(1) The ISRM Suggested Methods for Rock Characterisation, Testing and Monitoring do not contain a specific requirement for preparation of rock specimens. However, most of the test methods contain a section on preparation of samples, with requirements on sample volume, sample quality, preparation method, specific dimensions and tolerance checks on dimensions and shape.

(2) Examples of the common practice for preparing rock core specimens and determining dimensional and shape tolerances are given in the document listed in X.4.8. In the following paragraphs, extracts of, and comments on, these documents are given.

(3) It is not always possible to obtain or prepare rock core specimens which satisfy the desirable criteria given in the ISRM suggested methods, for example for weaker, more porous, and poorly cemented rock types and rock types containing structural features.

(4) All instruments and assemblies for determining straightness, flatness and perpendicularity of end surfaces should be controlled on a registered regular time basis having tolerances satisfying at least the requirements of the specific rock tests.

(5) Most unfractured cores taken by single tube, double tube or triple tube core barrels using rotary drilling techniques can be used with or without re-coring after a trimming of the end bearing surfaces. Blocks collected directly from a rock formation may also be used, if the orientation of the block is clearly indicated on the sample that will be used for re-coring the test specimens.

(6) The required sample volume depends on the test programme. For many purposes, samples 300 mm to 1000 mm long with a diameter greater than 50 mm should be sufficient for preparing rock specimens for a group of classification, strength and deformation tests.

(7) The required quantity of cores depends highly on the natural and induced fissuring of the rock material. The initial description of the core should include an evaluation of the degree of fissuring and homogeneity. This description should be used when selecting the core sections for testing.

(8) Selection of test specimens from zones of the core without fractures may lead to non-representativeness of the test specimens for the formation. This should be taken into account in the reporting.

(9) For weaker rocks (sedimentary rocks), the sample treatment is extremely important for deformability, strength and swelling tests. The rock samples for such tests should be packed in the field as soon as obtained from the core barrel. Even a short exposure may change the water content and the inherent properties of the rock.

Annex U
(Informative)

Classification testing of rock material

U.1 General

(1) The classification of the rock mass based on cores calls for the highest possible core recovery to identify discontinuities and possible cavities. The disturbance of the core from the drilling process should be minimised since most rock quality designations relate to the fractures found in the cores.

(2) Most classification systems relate to cores and rotary drilling samples with a sample diameter of at least 50 mm. For most tests, a non-fractured core length of 50 mm to 200 mm long is sufficient for element testing.

NOTE 1   Examples of tests for the classification of rock are given in the documents listed in X.4.9.

NOTE 2   Nationally and internationally recognised classification systems exist for different purposes. Rock mass classification systems, based on semi-numerical methods, exist for engineering purposes as summarised by Bieniawski (1989) *Engineering Rock Mass Classification*, (see X.5).

U.2 Rock identification and description

(1) EN ISO 14689-1 applies to the description of rock for geotechnics in civil engineering. The description is carried out on cores and other samples of natural rock and on rock masses.

(2) Any published and locally approved classification system may be used, provided the report gives a traceable reference.

NOTE   Examples of additional description procedures are given in the documents listed in X.4.9.1.

U.3 Water content

U.3.1 Test procedures

(1) If specified, accuracy checks should be carried out by comparing results on specimens taken in parallel within the same formation.

NOTE   Examples of test procedures are given in the documents listed in X.4.9.2.

U.3.2 Number of tests

(1) In general, the water content should be taken at least one per metre of core.

U.4 Density and porosity

U.4.1 Test procedures

(1) The determination of the porosity (or void ratio) calls for a determination of the density of solid particles (or an estimate of it based on local experience with similar rock type).

(2) The existence of closed pores may influence the porosity. Determination of the total pore volume may be based on the density of solids of a powdered sample, however the determination of the amount of open and closed pores calls for specialised analysis.

(3) Methods using mercury displacement should be avoided.

NOTE   Examples of test procedure are given in the documents listed in X.4.9.3.

U.4.2 Number of tests

(1) The density and porosity should be determined once at least every two metres, and at least once for each differentiated rock type unit, regardless of the rock homogeneity. The density/porosity parameters represent part of the framework for most evaluations of rock strength and deformation properties.

Annex V
(Informative)

Swelling testing of rock material

V.1 General

(1) Undisturbed rock specimens should preferably be tested where possible, since rock fabric has an important effect on swelling characteristics. Where the sample is too weak or too broken to allow preparation, such as joint fill material, the swelling index tests may be carried out on remoulded and re-compacted specimens. The procedures used should then be described in the report.

(2) Table V.1 gives a guideline for the minimum number of swelling tests required for different specimen dimensions. The suggestions apply for sites with a limited risk of occurrence of swelling rock types. For sites with rock types more likely to be subject to swelling, the number of tests should be increased to at least the double of the numbers given in the table. Other advanced tests may be better suited to determine the in-situ swelling performance.

NOTE   Examples of tests for swelling of rocks are given in the documents listed in X.4.10.

Table V.1 — Swelling tests on rocks. Recommended minimum number of rock specimens to be tested in one formation

Test type	Minimum thickness	Minimum diameter	Minimum number of test specimens	Notes
(1) Swelling pressure index under zero volume change	15 mm and/or 10 times max particle size	2,5 times thickness	3	Specimen should fit closely in the ring
(2) Swelling strain index for radially confined specimen with axial surcharge	15 mm and/or 10 times max particle size	4 times thickness	3 + duplicate specimens for water content	Specimen should fit closely in the ring
(3) Swelling strain developed in unconfined rock specimen	15 mm and/or 10 times max particle size	15 mm and/or 10 times max particle size	3 + duplicate specimens for water content	—

V.2 Swelling pressure index under zero volume change

(1) The testing apparatus may often be an ordinary oedometer cell for soil consolidation. However, the apparatus should be very stiff in order to avoid influence of deformations of the cell itself.

NOTE   An example of a test for the swelling pressure index under zero volume change is given in the document listed in X.4.10.1.

V.3 Swelling strain index for radially-confined specimen with axial surcharge

(1) The example specifies a loading device capable of applying a sustained pressure of 5 kPa to the specimen under water flooding. However, more appropriate to represent the field, may be specified. The report and any evaluation should include description of any such procedural deviations.

NOTE   An example of a test for swelling strain developed in an unconfined rock specimen is given in the document listed in X.4.10.2.

V.4 Swelling strain developed in unconfined rock specimen

NOTE   An example of tests for swelling strain developed in an unconfined rock specimen is given in the document listed in X.4.10.3.

Annex W
(Informative)

Strength testing of rock material

W.1 Uniaxial compressive strength and deformability

W.1.1 Test procedures

(1) It is recommended to follow the methods of the ISRM or ASTM, as examples for uniaxial compressive strength testing and deformability testing. In addition the modifications prescribed in W.1 may be used.

NOTE   See X.4.11.1

(2) The test procedure described in ISRM contains two levels of testing:
−   Part 1: Method for determining the uniaxial compressive strength of rock materials;
−   Part 2: Method for determining deformability of the rock materials in compression.

(3) The first method provides the compressive strength, the second method gives in addition the axial deformation modulus (Young's modulus) and Poisson's ratio. The second method is preferred.

(4) The procedures suggested by the reference ISRM are extremely difficult to conform to, especially with respect to sample preparation and geometrical tolerances. The practice recommended in this informative annex is less strict. Although the procedures recommended by ISRM are desirable, a set of minimum requirements is given herein. It is considered more important to run a greater number of tests than fewer tests on higher quality specimens.

(5) The following amendments should be made to the ISRM procedure.

−   The diameter of the platens should be between D and $(D + 10)$ mm, where D is the diameter of the sample. Provided it can be ensured that the stiffness of the plate is sufficient, the platen diameter may be greater than $(D + 10)$ mm. Special provisions are required to centre the specimen properly.

−   At least one of the two end platens should incorporate a spherical seat.

−   Test specimens should be right circular cylinders having a height to diameter ratio between 2 and 3 and a diameter not less than 50 mm. The diameter of the specimen related to the largest grain in the rock may in the case of weak rock be as low as 6:1. However, a ratio of 10:1 is preferred.

−   The ends of the specimen should be flat within 0,02 % of the specimen diameter and should not depart from perpendicularity to the axis of the specimen by more than 0,1°.

−   The use of capping materials or end surface treatments other than machining is not permitted except when testing soft rocks, where the mechanical characteristics of the capping materials should be better than the rock to be tested.

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- The diameter and the height of the test specimen should be determined to the nearest 0,1 mm or 0,2 %, whichever is the greatest.

- Regarding strain gauge measurements of radial and axial strains, the length of the gauges should be at least ten times the grain size. The measurements should be carried out on the mid third part of the test specimen to avoid influences from friction and stress non-homogeneities at the ends. Measurement of vertical strain over the whole specimen height is allowed if it can be shown that practically the same result is obtained as when the strain is measured over the middle third of the specimen height.

- The load should be applied on the specimen at a constant stress rate or constant strain rate such that failure occurs within 5 min to 15 min. If cycles of loading and unloading are performed to define better deformation parameters, the time for such should be excluded from the before-mentioned time period.

- The machine to be used for applying and measuring axial load to the specimen should be of sufficient capacity and capable of applying load at constant rate. The parallelism of the machine platens should be checked.

(6) Initial deformations may include bedding of the specimen ends to the compression machine and/or closure or micro-cracks in the test specimen. Measurement of the total vertical deformations by using only the distance between the two steel platens of the machine may lead to false deformation properties.

W.1.2 Number of tests

(1) The characteristics of rock may vary greatly as function of lithology, diagenesis or induration, stress history, weathering, and other natural processes, even within a geological stratum. Table W.1 gives a guideline for the minimum number of uniaxial compressive tests as a function of the variability of the rock and existing comparable experience.

Table W.1 − Uniaxial compression tests. Recommended minimum number of test specimens to be tested for one formation, Brazil tests and triaxial tests

Standard deviation of measured strength, (s)	Comparable experience		
% of mean	**None**	**Medium**	**Extensive**
$s > 50$	6	4	2
$20 < s < 50$	3	2	1
$s < 20$	2	1	0^a
[a] Only valid for very homogeneous rock types with extensive experience from nearby locations.			

W.2 Point load test

W.2.1 Test procedures

(1) It is recommended to follow the method of ISRM as an example of a procedure for the point load test

NOTE   See X.4.11.2.

(2) The test can be run with portable equipment or using a laboratory testing equipment, and may be conducted either in the field or in the laboratory.

(3) Rock specimens in the form of either core (the diametrical and axial tests), cut blocks (the block test), or irregular lumps (the irregular lump test) may be used for testing provided the reference (for instance ISRM) specifications of shape and dimensions are followed.

W.2.2 Number of tests

(1) The mean value of the Point Load Strength Index is used to classify samples or strata. To get a representative mean value, the minimum number of single tests should be five.

(2) For rock characterization and predictions of other strength parameters, a higher number of tests than specified in W.1.2, is necessary. Generally, at least 10 separate tests per stratum should be carried out.

W.3 Direct shear test

W.3.1 Test procedures

(1) It is recommended to follow the method of ISRM as an example for the direct shear test.

NOTE   See X.4.11.3.

(2) The following amendments may be recommended to the ISRM procedure.

- The testing machine should have a travel greater than the amount of dilatation or consolidation expected, and should be able of maintaining normal load to within 2 % of a selected value throughout the test. Dilation should be measured during the test with the same accuracy as the shear displacements.

- The rate of shear displacement should be less than 0,1 mm/min in the 10 min period before taking a set of readings. If automatic data logging is used, there may be no need for a reduction of the rate of shear displacements to 0,1 mm/min.

- The specimen should be reconsolidated under each new normal stress, and shearing continued according to criteria given in ISRM. If sample surfaces are cleaned before beginning a new testing phase, or the samples are unloaded before repositioning, this should be noted in the test report. The appearance of the material removed by cleaning should be described.

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(3) The direct shear strength may also be determined by field tests. This requires a detailed assessment of the field characteristics of the discontinuities.

(4) The results are utilised in, for example, equilibrium analysis of slope stability problems or for the stability analysis of dam foundations, tunnels and underground openings.

(5) Rock specimens in the form of either cores or cut blocks may be used. The test plane should preferably have a minimum area of 2 500 mm^2. In case of unfilled joints, the diameter or the edge (in the case of a square cross-section) of test specimens should preferably be related to the size of the largest grain in the rock by a ratio of at least 10:1. The ratio between joint length and shear box size is recommended not to be less than about 0,5 to avoid possible instability problems of the shear apparatus.

(6) Equipment for cutting the specimen, for example a large-diameter core drill or rock saw should be used. Percussive drills, hammers and chisels should be avoided as the samples have to be as undisturbed as possible.

(7) The direction of the test specimen in the testing machine is usually selected such that the sheared plane coincides with a plane of weakness in the rock, for example a joint, plane of bedding, schistosity or cleavage, or with the interface between soil and rock or concrete and rock.

W.3.2 Number of tests

(1) A shear strength determination should preferably comprise at least five tests on the same test horizon or from the same joint family, with each specimen tested at a different but constant normal stress in the applicable stress range.

W.4 Brazil test

W.4.1 Test procedures

(1) It is recommended to follow the method of ISRM as an example for the Brazil test.

NOTE   See X.4.11.4.

(2) Test specimens should be cut with specimen diameters (D) not less than the core size, ($D \approx 54$ mm), with a thickness approximately equal to the specimen radius. The cylindrical surface should be free from obvious tool marks. Any irregularities across the thickness of the specimen should not exceed 0,025 mm. End faces should be flat within 0,25 mm and parallel to within 0,25°.

(3) For shale and other anisotropic rock, it is recommended to cut test specimens parallel to and perpendicular to the bedding. For specimens cut parallel to the direction of the bedding, the direction of the load should also be specified.

W.4.2 Number of tests

(1) Table W.1 gives a guideline for the minimum number of Brazil tests as a function of the variability of the rock and existing comparable experience. For rock characterisation and predictions of other strength parameters a higher number of tests is necessary.

W.5 Triaxial compression test

W.5.1 Test procedures

(1) It is recommended to follow the method of ISRM as an example for the triaxial compression test.

NOTE   See X.4.11.5.

(2) Test specimens should be cut with specimen diameter (D) not less than the core size, ($D \approx 54$ mm), and the height equal to 2 to 3 times the diameter as defined in 5.4 and with the specifications in accordance with X.4.8.

W.5.2 Number of tests

(1) Table W.1 gives a guideline for the minimum number of triaxial compression tests as a function of the variability of the rock and existing comparable experience. For rock characterisation and predictions of other strength parameters a higher number of tests is necessary.

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Annex X

(Informative)

Bibliography

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X.1 Abbreviations and notations

The following notations have been used in this annex.

ASTM	American Society for Testing and Materials,
BS	British Standard
DGF	Dansk Geoteknisk Forening (Danish Geotechnical Society),
DIN	Deutsche Industrienorm (German Industrial Standard)
ETC	European Technical Committee (of ISSMGE),
ISRM	International Society of Rock Mechanics,
ISSMGE	International Society of Soil Mechanics and Geotechnical Engineering.
NEN	Nederlandse norm, (Dutch Standard)
NF	Norme Francaise, (French Standard)
SN	Schweizer Norm, (Swiss Standard)
SS	Svensk Standard (Swedish Standard)

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AC₁⟩ **X.3 Field tests**[8] ⟨AC₁

X.3.1 Cone penetration test

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X.4 Documents related to laboratory testing

X.4.1 Tests for classification, identification and description of soil

X.4.1.1 Site investigations

BS 5930:1999, *Code of practice for site investigations*

X.4.1.2 Water content determination

CEN ISO/TS 17892-1, *Geotechnical investigation and testing – Laboratory testing of soil – Part 1: Determination of water content*

DIN 18 121:1998, *Subsoil; testing procedures and testing equipment, water content, determination by drying in oven*

NF P 94-050:1995, *Soils: Investigation and testing. Determination of moisture content. Oven drying method*

BS 1377-2:1990, *Methods of test for soils for civil engineering purposes — Part 2: Classification tests*

SN 670 340:1959, *Essais; Teneur en eau / Versuche; Wassergehalt.*

ASTM D2216:1998, *Test method for laboratory determination of water (moisture) content of soil, rock, and soil-aggregate mixtures*

ASTM D2974:2000, *Test methods for moisture, ash, and organic matter of peat and other organic soils*

ASTM D4542-95(2001), *Test methods for pore water extraction and determination of the soluble salt content of soils by refractometer*

SS 0271 16:1989, *Geotechnical tests – Water content and degree of saturation*

X.4.1.3 Bulk density determination

CEN ISO/TS 17892-2, *Geotechnical investigation and testing – Laboratory testing of soil – Part 2: Determination of density of fine soils*

DIN 18 125:1997, *Soil, investigation and testing – Determination of density of soil Part 1: Laboratory tests*

NF P 94-053:1991, *Soils. Investigation and testing. Determination of density of fine soils. Cutting curb, mould and water immersion methods. Sols : Reconnaissance et Essais – Détermination de la masse volumique des sols fins en laboratoire – Méthodes de la trousse coupante, du moule et de l'immersion dans l'eau.*

BS 1377-2:1990, *Methods of test for soil for civil engineering purposes — Part 2: Classification tests*

SN 670 335:1960, *Versuche; Raumgewicht; Sandersatz-Methode / Essais; Poids specifique apparent; Methode du sable*

SS 0271 14:1989, *Geotechnical tests – Bulk density*

X.4.1.4 Particle density determination

CEN ISO/TS 17892-3, *Geotechnical investigation and testing – Laboratory testing of soil – Part 3: Determination of density of soil particles*

DIN 18 124:1997 *Soil, investigation and testing – Determination of density of solid particles – Capillary pyknometer, wide mouth pyknometer*

NF P 94-054:1991, *Sols : Reconnaissance et Essais – Détermination de la masse volumique des particules solides des sols – Méthode du pycnomètre à eau*
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BS 1377-2:1990, *Methods of test for soil for civil engineering purposes — Part 2: Classification tests*

SN 670 335:1960, *Versuche; Raumgewicht; Sandersatz-Methode / Essais; Poids specifique apparent; Methode du sable*

ASTM D854-02, *Test Method for Specific Gravity of Soils*

ASTM D4404:84 (1998), *Determination of pore volume and pore volume distribution of soil and rock by mercury intrusion porosimetry*

SS 0271 15:1989, *Geotechnical tests – Grain density and specific gravity*

X.4.1.5 Particle size analysis

CEN ISO/TS 17892-4, *Geotechnical investigation and testing – Laboratory testing of soil – Part 4: Determination of particle size distribution*

DIN 18 123:1996, *Soil, investigation and testing – Determination of grain-size distribution*

NF P 94-056:1996, *Sols : Reconnaissance et Essais – Analyse granulométrique – Méthode par tamisage à sec après lavage* (in French)

XP P 94-041:1995, *Sols : Reconnaissance et Essais – Identification granulométrique – Méthode de tamisage par voie humide* (in French)

BS 1377-2:1990, *Methods of test for soil for civil engineering purposes — Part 2: Classification tests*; Subclause 9.2 Wet sieving method

BS 1377-2:1990, *Methods of test for soil for civil engineering purposes — Part 2: Classification tests*; Subclause 9.5 Sedimentation by the hydrometer method

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SN 670 810c:1986, *Granulats mineraux et sols; Analyse granulometrique par tamisage / Mineralische Baustoffe und Lockergesteine; Siebanalyse*

SN 670 816:1964, *Matériaux pierreux; Sédimentométrie par la méthode de l'aréomètre / Gesteinsmaterialien; Schlämmversuch nach der Araeometermethode*

ASTM D2217-85 (1998), *Wet preparation of s samples for particle size analysis and determination of soil constants*

ASTM D422-63 (1998), *Test method for particle size analysis of soil*

SS 0271 23:1992, *Geotechnical tests –Particle size distribution – sieving*

SS 0271 24:1992, *Geotechnical tests –Particle size distribution – sedimentation, hydrometer method*

X.4.1.6 Consistency (Atterberg) limits determination

CEN ISO/TS 17892-12, *Geotechnical investigation and testing – Laboratory testing of soil – Part 12: Atterberg limits*

DIN 18 122:1997, *Soil, investigation and testing Consistency limits Part 1: Determination of liquid limit and plastic limit*

NF P 94-051:1993, *Soils. Investigation and testing. Determination of Atterberg's limits. Liquid limit test using Casagrande apparatus. Plastic limit test on rolled thread*

NF P94-052-1:1995, *Sols: Reconnaissance et Essais – Détermination des limites d'Atterberg – Partie 1 : Limite de liquidité – Méthode du cône de pénétration*

BS 1377-2:1990, *Methods of test for soil for civil engineering purposes — Part 2: Classification tests*; Clause 4 Determination of the liquid limit

BS 1377-2:1990, *Methods of test for soil for civil engineering purposes — Part 2: Classification tests*; Clause 5 Determination of the plastic limit and plasticity index

SN 670 345:1959, *Essais; Limites de consistance / Versuche; Konsistenzgrenzen*

SS 0271 20: 1990, *Geotechnical tests – Cone liquid limit*

SS 0271 21: 1990, *Geotechnical tests – Plastic limit*

X.4.1.7 Determination of the density index of granular soil

BS 1377-4:1990, *Methods of test for soil for civil engineering purposes — Part 4: Compaction related tests*; Clause 4 Determination of maximum and minimum dry densities for granular soils

NF P 94-059:2000, *Sols : Reconnaissance et Essais – Détermination des masses volumiques minimale et maximale des sols non cohérents*

X.4.1.8 Soil dispersibility determination

BS 1377-5:1990, *Methods of test for soil for civil engineering purposes — Part 5: Compressibility, permeability and durability tests*; Clause 6 Determination of dispersibility

X.4.1.9 Frost susceptibility determination

SN 670 321:1994, *Essais sur les sols - Essai de gonflement au gel et essai CBR après dégel / Versuche an Böden - Frosthebungsversuch und CBR-Versuch nach dem Auftauen*

BS 1377-5:1990, *Methods of test for soil for civil engineering purposes — Part 5: Compressibility, permeability and durability tests*; Clause 7 Determination of frost heave

X.4.2 Chemical testing of soil and groundwater

X.4.2.1 General

BS 1377-3:1990, *Methods of test for soil for civil engineering purposes — Part 3: Chemical and electrochemical tests*

X.4.2.2 Organic content determination

BS 1377-3:1990, *Methods of test for soil for civil engineering purposes — Part 3: Chemical and electrochemical tests*; Clause 4 Determination of the mass loss on ignition or an equivalent method

ASTM D2974:1987, *Test methods for moisture, ash, and organic matter of peat and other organic soils*

NF P 94-055:1993, *Sols : Reconnaissance et Essais – Détermination de la teneur pondérale en matières organiques d'un sol – Méthode chimique*

XP P94-047:1998, *Sols : Reconnaissance et Essais – Détermination de la teneur pondérale en matière organique – Méthode par calcination*

SS 0271 05:1990, *Geotechnical tests – Organic content – Ignition loss method*

SS 0271 07:1990, *Geotechnical tests – Organic content – Colorimetric method*

X.4.2.3 Carbonate content determination

BS 1377-3:1990, *Methods of test for soil for civil engineering purposes — Part 3: Chemical and electrochemical tests*; Clause 6 Determination of the carbonate content

DIN 18129, *Soil, investigation and testing - Determination of lime content*

Head K.H., *Manual of Soil Laboratory Testing. Vol.1: Soil Classification and Compaction Tests*, 2nd ed;Vol 1:1992

NF P 94-048:1996, *Sols : Reconnaissance et Essais – Détermination de la teneur en carbonate – Méthode du calcimètre*

X.4.2.4 Sulfate content determination

BS 1377-3:1990, *Methods of test for soil for civil engineering purposes — Part 3: Chemical and electrochemical tests*; Clause 5 Determination of the sulfate content of soil and groundwater

X.4.2.5 pH Value determination (acidity and alkalinity)

BS 1377-3:1990, *Methods of test for soil for civil engineering purposes — Part 3: Chemical and electrochemical tests*; Clause 9 Determination of the pH value

X.4.2.6 Chloride content determination

BS 812-118:1988, *Testing aggregates. Methods for determination of sulfate content*

BS 1377-3:1990, *Methods of test for soil for civil engineering purposes — Part 3: Chemical and electrochemical tests*; Subclauses 7.2, 7.3

X.4.3 Strength index testing of soil

X.4.3.1 Laboratory vane

BS 1377-7:1990, *Methods of test for soils for civil engineering purposes — Part 7: Shear strength tests* (total stress)

NF P 94-072:1995, *Sols : Reconnaissance et Essais - Essai scissométrique en laboratoire*

X.4.3.2 Fall cone

CEN ISO/TS 17892-6, *Geotechnical investigation and testing – Laboratory testing of soil – Part 6: Fall cone test*

SS 02 7125:1991, *Geotechnical test methods. Undrained shear strength. Fall cone test Cohesive soil*

X.4.4 Strength testing of soil

X.4.4.1 Unconfined compression

CEN ISO/TS 17892-7, *Geotechnical investigation and testing – Laboratory testing of soil – Part 7: Unconfined compression test on fine grained soils*

NF P 94-077:1997, *Sols: Reconnaissance et Essais - Essai de compression uniaxiale*

X.4.4.2 Unconsolidated undrained compression

CEN ISO/TS 17892-8, *Geotechnical investigation and testing - Laboratory testing of soil - Part 8: Unconsolidated undrained triaxial test*

NF P 94-070:1994, *Sols: Reconnaissance et Essais - Essais à l'appareil triaxial de révolution - Généralités, définitions*
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NF P 94-074:1994, *Sols: Reconnaissance et Essais - Essai à l'appareil triaxial de révolution - Appareillage - Préparation des éprouvettes - Essais (UU) non consolidé non drainé - Essai (C_U + u) consolidé non drainé avec mesure de pression interstitielle - Essai (CD) consolidé drainé.*

X.4.4.3 Consolidated triaxial compression test

CEN ISO/TS 17892-9, *Geotechnical investigation and testing – Laboratory testing of soil – Part 9: Consolidated triaxial compression tests on water saturated soils*

BS 1377-8:1990, *Methods of test for soils for civil engineering purposes — Part 8 Shear strength tests* (effective stress)

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NF P 94-070:1994, *Sols: Reconnaissance et Essais - Essais à l'appareil triaxial de révolution - Généralités, définitions*

NF P 94-074:1994, *Sols : Reconnaissance et Essais - Essai à l'appareil triaxial de révolution - Appareillage - Préparation des éprouvettes - Essais (UU) non consolidé non drainé - Essai (C$_U$ + u) consolidé non drainé avec mesure de pression interstitielle - Essai (CD) consolidé drainé.*

X.4.4.4 Consolidated direct shear box tests

CEN ISO/TS 17892-10, *Geotechnical investigation and testing - Laboratory testing of soil - Part 10: Direct shear tests*

BS 1377-7:1990, *Methods of test for soils for civil engineering purposes — Part 7: Shear strength tests* (total stress)

ASTM D 3080-98, *Test method for direct shear test of soils under consolidated drained conditions*

SS027127, *Geotechnical tests - shear strength – Direct shear test, CU- and CD- tests – Cohesive soils*

NF P94-071-1:1994 Sols : Reconnaissance et Essais - Essai de cisaillement rectiligne à la boîte - Partie 1 : Cisaillement direct.

NF P94-071-2:1994, *Sols : Reconnaissance et Essais - Essai de cisaillement rectiligne à la boîte - Partie 2 : Cisaillement alterné*

X.4.5 Compressibility testing of soil

CEN ISO/TS 17892-5, *Geotechnical investigation and testing - Laboratory testing of soil - Part 5: Incremental loading oedometer test*

BS 1377-5:1990, *Methods of test for soils for civil engineering purposes — Part 5: Compressibility, permeability and durability tests*

NS 8017:1991, *Geotechnical testing - Laboratory methods - Determination of one-dimensional consolidation properties by oedometer testing - Method using incremental loading*

ASTM D2435-96, *Test method for One-Dimensional Consolidation Properties of Soils*

XP P94-090-1:1997, *Sols : Reconnaissance et Essais - Essai œdométrique -Partie 1 : Essai de compressibilité sur matériaux fins quasi saturés avec chargement par paliers*

XP P 94-091:1995, *Sols: Reconnaissance et Essais - Essai de gonflement à l'œdomètre - Détermination des déformations par chargement de plusieurs éprouvettes*

SS 0271 26:1991, *Geotechnical tests – Compression properties – Oedometer test, CRS-test – Cohesive soil*

SS 0271 29: 1992, *Geotechnical tests – Compression properties – Oedometer test, incremental loading – Cohesive soil*

X.4.6 Compaction testing of soil

BS 1377-4:1990, *Methods of test for soils for civil engineering purposes — Part 4: Compaction related tests*; Clause 3 Determination of dry density/moisture content relationship

BS 1377-4:1990, *Methods of test for soils for civil engineering purposes — Part 4: Compaction related tests*; Clause 7 Determination of California Bearing Ratio (CBR)

NOTE   ASTM D-698-78, D-1557-78 and AASHTO /99 and T180 might be used for compaction tests and ASTM D1883-94 and AASHTO T193 might be used for the California Bearing Ratio Determination. However, BS 1377:1990 has minor deviations from the specification in the US recommendations, which are used in most road laboratories.

SS027109, *Geotechnical tests – Compaction properties – Laboratory compaction*

NF P 94-078:1997, *Sols : Reconnaissance et Essais - Indice CBR après immersion - Indice CBR immédiat - Indice Portant Immédiat - Mesure sur échantillon compacté dans le moule CBR*

NF P 94-093:1999, *Sols : Reconnaissance et Essais - Détermination des références de compactage d'un matériau - Essai Proctor normal - Essai Proctor modifié*

X.4.7 Permeability testing of soil

CEN ISO/TS 17892-11, *Geotechnical investigation and testing - Laboratory testing of soil - Part 11: Permeability test*

BS 1377-5:1990, *Methods of test for soils for civil engineering purposes — Part 5: Compressibility, permeability and durability tests*

DIN 18130-1:1998, *Soil. Investigation and testing. Determination of the coefficient of water permeability. Part 1 Laboratory tests*

ISO/DIS 17313, *Soil quality – Determination of hydraulic conductivity of saturated porous materials using flexible wall permeameter*. ISO/TC 190/SC 5.

NOTE   ISO/DIS 17313 relates to environmental testing and includes some very strict normative clauses not necessary for normal geotechnical purposes.

X.4.8 Preparation of specimen for testing on rock materials

ASTM D4543-01, *Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances*

X.4.9 Classification testing of rock materials

X.4.9.1 General

BS 5930:1981, *Code of practice for site investigation Section 8 Description and classification of rock for engineering purposes*

ISRM Suggested Methods for Rock Characterization, Testing and Monitoring, Part I Site Characterization (1981).

X.4.9.2 Water content determination

ISRM Part 1, *Suggested methods for determining water content, porosity, density, absorption and related properties*; Section 1 Suggested method for determination of the water content of a rock sample.

X.4.9.3 Density and porosity

ISRM Part 1, *Suggested methods for determining water content, porosity, density, absorption and related properties*; Section 2 Suggested method for porosity/density determination using saturation and calliper techniques

ISRM Part 1, *Suggested methods for determining water content, porosity, density, absorption and related properties*; Section 3 Suggested method for porosity/density determination using saturation and buoyancy techniques

X.4.10 Swelling testing of rock material

X.4.10.1 Swelling pressure index under zero volume change

ISRM Suggested Methods For Determining Swelling and Slake-Durability Index Properties; Test 1 Suggested Method for Determination of the Swelling Pressure Index of Zero Volume Change

X.4.10.2 Swelling strain index for radially confined specimen with axial surcharge

ISRM Suggested Methods For Determining Swelling and Slake-Durability Index Properties; Test 2 Suggested Method for Determination of the Swelling Strain Index for a Radially Confined Specimen with Axial Surcharge

X.4.10.3 Swelling strain developed in unconfined rock specimen

ISRM Suggested Methods For Determining Swelling and Slake-Durability Index Properties; Test 3 Suggested Method for Determination of the Swelling Strain Developed in an Unconfined Rock Specimen

X.4.11 Strength testing of rock materials

X.4.11.1 Uniaxial compressive strength and deformability

ISRM Suggested Methods For Determining Unconfined Compressive Strength and Deformability

ASTM D 2938:1991, *Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens*

X.4.11.2 Point load test

ISRM Suggested Method for Determining Point Load Strength;
revised version has been published in International Journal for Rock Mechanics. Min. SCI. &
Geomech. Abstr. Vol 22, No. 2, pp. 51-60, 1985

X.4.11.3 Direct shear test

*ISRM Suggested Method for Determining Shear Strength, Part 2: Suggested Method For
Laboratory Determination of Direct Shear Strength*

X.4.11.4 Brazil test

*ISRM Suggested Method for Determining Tensile Strength of Rock Materials, Part 2: Suggested
Method for Determining Indirect Tensile Strength by the Brazil Test*

X.4.11.5 Triaxial compression test

ISRM Suggested Method for Determining the Strength of Rock Materials in Triaxial Compression

X.5 Books, articles and other publications related to laboratory testing

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Engineering Rock Mass Classification
New York: Wiley
251 p.

BRE Paper BR 279 (19--) "Sulfate and acid attack on concrete in the ground: recommended
procedures for soil analysis".
Watford, UK: Building Research Establishment

A guide to engineering geological description
DGF Bulletin 1, Rev. 1,
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London,Pentech Press, 1992

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London, Pentech Press, 1994

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Manual of Soil Laboratory Testing. Vol. 3: Effective Stress Tests.
London, Pentech Press, 1986

Suggested method for determining point load strength
Min. Sci & Geomech. Abstr. Vol 22, No. 2,
International Journal of Rock Mechanics
ISRM, 1985
pp. 51-60.

Sherard, K.L., Decker, R.S.and Ryker, N.L. (1972)
Piping in Earth Dams of Dispersive Clay Vol. 1, Part 1
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West Lafayette, Indiana, Purdue University, June 1972
pp 589–626

Sherard, K.L., Dunnigan, L.P., Decker, R.S. and Steel, E.F.
Pinhole test for identifying dispersive soil
K. Geotechn. Eng. Div., ASCE. Vol. 102, No. GT1 (January), 1976
pp 69–85

Slunga, E. & Saarelainen, S. (1989)
Determination of frost-susceptibility of soil, A.A. Balkema
Proc. of 12th ICSMFE, Vol. 2.Session 19
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pp 1465–1468

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