CEN/TC 250/SC 7
Eurocode 7 - Geotechnical design

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prEN 1997-2 MASTER v2021.11 Submission

Document type: Other committee document

Date of document: 2021-05-03

Expected action: INFO

Background:

Committee URL: https://cen.iso.org/livelink/livelink/open/centc250sc7

CEN/TC 250

Date: 2021-04

prEN 1997-2:202x

CEN/TC 250

Secretariat: NEN

Eurocode 7: Geotechnical design — Part 2: Ground properties

Eurocode 7 - Entwurf, Berechnung und Bemessung in der Geotechnik — Teil 2 Bodeneigenschaften

Eurocode 7 - Calcul géotechnique — Partie 2: Propriétés des terrains

ICS:

Descriptors:

Document type: European Standard
Document subtype:
Document stage:
Document language: E

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Drafting foreword by PT6

This document (prEN 1997-2:20xx) has been prepared by project team M515/SC7.T6.

This document is the Final PT Draft of prEN 1997-2 (as required under Phase 4 of Mandate M/515).

This document is a working document.

<Drafting note: Once revised the comments received from NSBs on October PT6 Draft of EN 1997-2 and taking into account there are contradictory comments, PT6 has decided to leave TG-A2 to discuss and vote on the following issues:

- Position of Table 5.4 There are three possibilities:

  - Option 1: To keep it as NDP in the main text where it is.

    (doing this, NSBs in their National Annex can change the content of the table to adjust it to their own national practice and even consider it as "Not applicable" in their countries).

  - Option 2: To transfer Table 5.4 + clauses 5.4.3 (2), (3), (4), (6), (7) the content to a new informative Annex

    (doing this, NSBs can declare the status and the potential use of the Annex in their National Annex)

  - Option 3. To delete Table 5.4 and adjacent clauses 5.4.3 (2), (3), (4), (6), (7) in accordance with comments 307 (FIN) / 317 (NO) / 323 (SE) / 328 (DE) / 331 (DK)

- Status of Annex A: informative or normative. There are three possibilities:

  - Option 1: To keep it normative as an Annex (doing this, Annex A (Part 2) and Annex C (Part 1) will be the only annexes which are normative in EN 1997).

  - Option 2B: To transfer the content to the main text (doing this, the content will be normative)

  - Option 3C: To change the status to "Informative" (If such change is done, note the status of Annex C in Part 1 should be also modified to informative, for coherence)

- National Standards in Tables of Clauses 7 to 12. There are two possibilities.

  - add new Table in the informative Annex B, as proposed in this draft. PT6 thinks it is a good idea to create this new table in the informative Annex with the National Standards for those tests mentioned in Tables of Clauses 7 to 12 that currently is not covered by a EN-Standard. Doing this, EC7 will provide very useful information for the designers. This table B4 is now a draft-version to be updated by TG A2, if considered appropriate.

  - Only have the references to EN, ISO standards and ISRM methods. As updated in the main text of this draft. >

European Foreword

[DRAFTING NOTE: this version of the foreword is relevant to EN Eurocode Parts for enquiry stage]

This document (EN 1997-2) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document will supersede EN 1997-2:2007.

The first generation of EN Eurocodes was published between 2002 and 2007. This document forms part of the second generation of the Eurocodes, which have been prepared under Mandate M/515 issued to CEN by the European Commission and the European Free Trade Association.

The Eurocodes have been drafted to be used in conjunction with relevant execution, material, product and test standards, and to identify requirements for execution, materials, products and testing that are relied upon by the Eurocodes.

The Eurocodes recognise the responsibility of each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level through the use of National Annexes.

0 Introduction

0.1 Introduction to the Eurocodes

The Structural Eurocodes comprise the following standards generally consisting of a number of Parts:

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

The Eurocodes are intended for use by designers, clients, manufacturers, constructors, relevant authorities (in exercising their duties in accordance with national or international regulations), educators, software developers, and committees drafting standards for related product, testing and execution standards.

NOTE Some aspects of design are most appropriately specified by relevant authorities or, where not specified, can be agreed on a project-specific basis between relevant parties such as designers and clients. The Eurocodes identify such aspects making explicit reference to relevant authorities and relevant parties.

0.2 Introduction to Eurocode 7

Eurocode 7 is intended to be used in conjunction with EN 1990, which establishes principles and requirements for the safety, serviceability, robustness, and durability of structures, including geotechnical structures, and other construction works.

Eurocode 7 establishes additional principles and requirements for the safety, serviceability, robustness, and durability of geotechnical structures.

Eurocode 7 is intended to be used in conjunction with the other Eurocodes for the design of geotechnical structures, including temporary geotechnical structures.

Design and verification in Eurocode 7 are based on the partial factor method or other reliability-based methods, prescriptive rules, testing, or the observational method.

Eurocode 7 consists of a number of parts:

- EN 1997-1, *Geotechnical design – Part 1: General rules*
- EN 1997-2, *Geotechnical design – Part 2: Ground properties*
- EN 1997-3, *Geotechnical design – Part 3: Geotechnical structures*

0.3 Introduction to EN 1997-2

Eurocode 7 Part 2 establishes principles and requirements for obtaining information about the ground at a site, as needed for the design and execution of geotechnical structures, including temporary geotechnical structures.

Eurocode 7 Part 2 gives guidance for planning ground investigations, collect information about ground properties and groundwater conditions, and preparation of the Ground Model.

Eurocode 7 Part 2 gives guidance for the selection of field investigation and laboratory test methods to obtain derived values of ground properties.

Eurocode 7 Part 2 gives guidance on the presentation of the results of ground investigation, including derived values of ground properties, in the Ground Investigation Report.

0.4 Verbal forms used in the Eurocodes

The verb “shall" expresses a requirement strictly to be followed and from which no deviation is permitted in order to comply with the Eurocodes.

The verb “should” expresses a highly recommended choice or course of action. Subject to national regulation and/or any relevant contractual provisions, alternative approaches could be used/adopted where technically justified.

The verb “may" expresses a course of action permissible within the limits of the Eurocodes.

The verb “can" expresses possibility and capability; it is used for statements of fact and clarification of concepts.

0.5 National annex for EN 1997-2

National choice is allowed in this standard where explicitly stated within notes. National choice includes the selection of values for Nationally Determined Parameters (NDPs).

The national standard implementing EN 1997-2 can have a National Annex containing all national choices to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

When no national choice is given, the default choice given in this standard is to be used.

When no national choice is made and no default is given in this standard, the choice can be specified by a relevant authority or, where not specified, agreed for a specific project by appropriate parties.

National choice is allowed in EN 1997-2 through the following clauses:

5.4.3(2)	5.4.3(3)

National choice is allowed in EN 1997-2 on the application of the following informative annexes:

Annex B	Annex C	Annex D
Annex E	Annex F	Annex G

The National Annex can contain, directly or by reference, non-contradictory complementary information for ease of implementation, provided it does not alter any provisions of the Eurocodes.

1 Scope

1.1 Scope of EN 1997-2

(1) EN 1997-2 provides requirements and recommendations for determining ground properties for the design and verification of geotechnical structures.

1.2 Assumptions

(1) The provisions in EN 1997-2 are based on the assumptions given in ENs 1990 and 1997-1.

(2) EN 1997-2 will be used in conjunction with EN 1997-1, which provides general rules for design and verification of all geotechnical structures.

(3) EN 1997-2 will be used in conjunction with EN 1997-3, which provides specific rules for design and verification of certain types of geotechnical structures.

(4) EN 1997-2 will be used in conjunction with EN 1998-1 which provides the requirements for the ground properties needed to define the seismic action.

(5) EN 1997-2 will be used in conjunction with EN 1998-5 which provides rules for the design of geotechnical structures in seismic regions.

(6) In addition to the assumptions given in ENs 1990 and 1997-1, the provisions in EN 1997-2 assume that:

– ground investigations are planned by personnel or enterprises knowledgeable about potential ground and groundwater conditions;
– ground investigations are executed by enterprises having appropriate skill and experience;
– evaluation of test results and derivation of ground properties from ground investigation are carried out by personnel with appropriate geotechnical experience and qualifications.

2 Normative references

<Drafting note: Status of the test standards given in tables of Clauses 7 to 12

The content of this Note is implicit in CEN rules about reference to standards of any kind or status but PT6 wants to highlight the following issues:

- In absence of relevant EN standards, national standards as prescribed by the national authorities are used.
- If and when a European test standard dealing with any of the tests given in any of the tables in Clauses 7 to 12 is approved, it shall take precedence over the standards given in such tables. >

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

NOTE 1. See the Bibliography for a list of other documents cited that are not normative references, including those referenced as recommendations (i.e. in 'should' clauses), permissions ('may' clauses), possibilities ('can' clauses), and in notes.

EN 1997-1, *Eurocode 7: Geotechnical design – Part 1: General rules*

EN 1997-3, *Eurocode 7: Geotechnical design – Part 3: Geotechnical structures*

3 Terms, definitions, and symbols

3.1 Terms and definitions

For purposes of this document, the definitions given in EN 1990 and EN 1997-1 and the following definitions apply.

3.1.1 Common terms used in EN 1997-2

3.1.1.1 site

surface area or underground space where construction work or other development is undertaken

3.1.1.2 anthropogenic ground

materials placed by human activity

3.1.1.3 rockhead

boundary between soil and rock

Note 1 to entry: rockhead can either be a geological boundary between in-situ (usually weathered) and transported materials or an engineering boundary between materials that behave as soil and those that behave as rock.

3.1.2 Terms relating to the Ground Model

3.1.2.1 state property

ground property that can change over time, such as mass density, water content and saturation, density index, or stress state

3.1.2.2 measured value of a ground property

value of a ground property recorded during a test

3.1.3 Terms relating to content of ground investigation

3.1.3.1 ground investigation

use of non-intrusive and intrusive methods to investigate the ground and groundwater conditions beneath or around the site or zone of influence

3.1.3.2 ground investigation location

location (point, line, or area) on the site where the ground is examined and investigated by intrusive or non-intrusive methods

3.1.3.3 low-rise structure

warehouse sheds, factory buildings, or residential buildings up to three storeys high

3.1.3.4 high-rise structure

buildings and structures greater than three storeys high, including chimneys and towers

3.1.3.5 site inspection

observation and recording of features relevant to the surface and sub-surface conditions and any exposures of the ground, existing infrastructure or environment

Note 1 to entry: the inspection normally extends beyond the site boundaries

3.1.3.6 sample

defined amount of rock, soil or groundwater recovered from recorded depth

[SOURCE: EN ISO 22475-1]

3.1.3.7 specimen

part of the sample taken for laboratory testing

3.1.3.8 sample quality class

quality class of soil sample based on its degree of disturbance according to sampling technique

[SOURCE: EN ISO 22475]

3.1.3.9 disturbance factor

disturbance of the rock mass

3.1.3.10 mapping

the process of physically going out into the field and recording information from the ground at the surface or from excavations and exposures

3.1.3.11 geological mapping

mapping to record and describe geological information and features observed in the field

Note 1 to entry: Description covers features such as morphology, lithology, hydrogeology, weathering, and any visible geological structure.

3.1.3.12 geotechnical mapping

geological mapping with the addition of ground classification in terms of quality indexes and of geometrical features of discontinuities

Note 1 to entry: Classification covers parameters such as rock quality designation, rock mass rating, joint sets, alteration and weathering numbers, joint wall roughness, technical ground behaviour.

3.1.4 Terms relating to chemical, physical, and state properties

3.1.4.1 classification

definition of material groups and classes and assigning of materials to groups and classes with similar properties

[SOURCE: modified from EN ISO 16907-2]

3.1.4.2 coarse soil

soil with particle sizes between 0.063 and 63 mm

[SOURCE: EN ISO 14688-1]

3.1.4.3 fine soil

soil with particle sizes smaller than 0.063 mm

[SOURCE: EN ISO 14688-1]

3.1.4.4 very coarse soil

soil with particle sizes larger than 63 mm

[SOURCE: EN ISO 14688-1]

3.1.4.5 density index

ratio of the difference between the maximum void ratio and the observed void ratio to the difference between maximum and minimum void ratios, expressed as a percentage

3.1.4.6 relative density

synonym for 'density index'

3.1.4.7 consistency (Atterberg) limits

collective name for liquid, plastic, and shrinkage limits of soil

3.1.4.8 liquid limit

water content of soil at which a fine soil passes from the liquid to the plastic condition, as determined by the liquid limit test

[SOURCE: EN ISO 14688-2]

3.1.4.9 plastic limit

water content of soil at which a fine soil passes from the plastic to the semi-solid condition, as determined by the plastic limit test

[SOURCE: EN ISO 14688-2]

3.1.4.10 shrinkage limit

water content of soil below which loss of water does not result in volume reduction

3.1.4.11 activity index

ratio of the plasticity index and the clay fraction that is finer than two microns (expressed as a percentage)

3.1.5 Terms relating to strength

3.1.5.1 shear strength envelope

expression that identifies stress combinations that produce material failure

3.1.5.2 shear strength parameters

material parameters appearing in the expression of shear strength envelopes

3.1.5.3 shear strength in effective stresses

shear strength obtained from an envelope defined in terms of effective stress

3.1.5.4 peak shear strength

upper limit of the shear strength observed in a test

3.1.5.5 critical state shear strength

shear strength observed when shearing continues without change in either volume or pore water pressure

3.1.5.6 residual shear strength

lower limit of the shear strength of a fine soil reached after extensive shearing and particle re-orientation or lower limit of the shear strength reached after extensive shearing of discontinuities

3.1.5.7 undrained shear strength

shear strength of water saturated soils obtained from an envelope defined in terms of total stress

3.1.5.8 peak undrained shear strength

upper limit of the undrained shear strength for undisturbed soil

3.1.5.9 remoulded undrained shear strength

undrained shear strength for totally remoulded soil

3.1.5.10 sensitivity

ratio between peak and remoulded undrained shear strengths

3.1.5.11 crack initiation stress

stress level at which pre-existing cracks (rock material) or discontinuities (rock mass) initiate growth

3.1.5.12 crack damage stress

stress level at which unstable growth of cracks (rock material) or discontinuities (rock mass) occurs

3.1.5.13 Geological Strength Index

index used to estimate rock mass strength and rock mass deformation modulus

3.1.5.14 Joint Roughness Coefficient

number characterizing the roughness of discontinuities

3.1.5.15 joint wall compressive strength

compressive strength of a discontinuity adjusted for weathering, size, width, infill and scale

3.1.5.16 rock mass strength

strength resulting from the combination of the structural and material properties of the rock mass

3.1.5.17 flexural strength

strength of the rock material from a flexure test

[SOURCE: ASTM C880-98]

3.1.6 Terms relating to stiffness and consolidation

3.1.6.1 elastic modulus

ratio of stress increase to the corresponding increase in strain in the stress-strain relationship as shown in Figure 3.1

3.1.6.2 bulk modulus

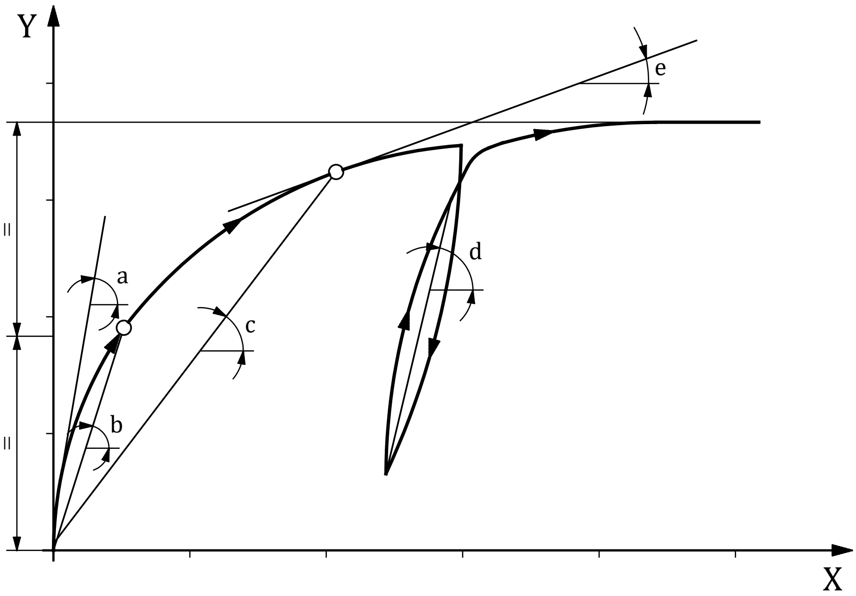
ratio between mean stress increase to a corresponding decrease in volumetric strain

3.1.6.3 shear modulus

ratio of shear stress increase to a corresponding increase in shear strain, as shown in Figure 3.1

3.1.6.4 secant modulus

ratio between stress and the corresponding strain accumulated from an initial reference state, as defined by Figure 3.1



Key

X Strain

Y Shear stress

a E_0 / G_0

b E_{50} / G_{50}

c E_{sec} / G_{sec}

d E_{cyc} / G_{cyc}

e E_{tan} / G_{tan}

Figure 3.1 — Definition of modulus on stress-strain curve (Key: X = strain, Y = shear stress)

3.1.6.5 tangent modulus

ratio between small increments of stress and strain from a given reference state, as shown in Figure 3.1.

3.1.6.6 very small strain elastic modulus

value of the elastic modulus at strains < 10^{-5}

3.1.6.7 very small strain Poisson's ratio

value of Poisson's ratio at strains < 10^{-5} (see Annex F)

3.1.6.8 oedometer (one dimensional) modulus

ratio of the variation of a principal stress by the linear strain obtained in the same direction, with the other principal strains equal to zero

Note 1 to entry: Also known as the 'constrained modulus'.

3.1.6.9 swelling

ground volume expansion caused by physicochemical processes or by the ingress of water

3.1.6.10 undrained modulus

elastic modulus for undrained conditions

3.1.7 Terms relating to cyclic, dynamic, and seismic properties

3.1.7.1 compressional wave velocity

velocity of propagation of a compressional (primary) wave in a medium

3.1.7.2 cyclic liquefaction

transition of soil behaviour from solid-like to liquid-like due to cyclic or seismic actions

3.1.7.3 cyclic modulus

slope of the line connecting the two points of reversal in cyclic loading as shown in Figure 3.1

3.1.7.4 cyclic shear strength

maximum value of cyclic shear stress that can be sustained for a given number of cycles without exceeding a given strain threshold

3.1.7.5 cyclic strain

maximum strain attained or imposed during the application of cyclic actions

3.1.7.6 cyclic stress

maximum stress attained or imposed during to the application of cyclic actions

3.1.7.7 damping ratio

ratio between the energy dissipated in a cyclically loaded system and the corresponding elastic energy of deformation based on hysteresis loops of stress vs strain

3.1.7.8 cyclic degradation

deterioration of ground properties due to repeated load cycles (similar to fatigue in structural members)

3.1.7.9 fundamental frequency

lowest value of the frequency associated with relative maximum amplification of the seismic ground motion

3.1.7.10 post-cyclic strength

available strength after the application of a given number of stress cycles

3.1.7.11 post-cyclic creep

deformation associated with average constant loads after the application of a given number of stress cycles

3.1.7.12 seismic bedrock

reference formation identified by a shear wave velocity greater than 800 m/s

[SOURCE: EN 1998-1]

3.1.7.13 shear wave velocity

velocity of propagation of a shear wave in a medium

3.1.8 Terms relating to groundwater and geohydraulic properties

3.1.8.1 aquitard

confining layer that retards, but does not prevent, the flow of water to or from an adjacent aquifer

[SOURCE: EN ISO 22475-1]

3.1.8.2 joint water pressure

pressure of the water in the joints or discontinuities of ground

3.1.8.3 pressure head

ratio of the pore water/joint water pressure and the weight density of water above a point

[SOURCE: EN ISO 18674-4]

3.1.8.4 piezometer

field instrument system for measuring pore or joint water pressure or piezometric level, at the measuring point

Note 1 to entry: The system is either an open piezometer system or a closed piezometer system.

[SOURCE: modified from EN ISO 18674-4]

3.1.8.5 open system

field instrument system in which the fluid is in direct contact with the atmosphere and the piezometric level at the measuring point is measured

Note 1 to entry: Also known as an 'open piezometric system'.

[SOURCE: EN ISO 18674-4]

3.1.8.6 closed system

measuring system in which the reservoir is not in direct contact with the atmosphere and in which the pressure in the fluid is measured by a pressure measuring device

Note 1 to entry: Also known as a 'closed piezometric system'.

[SOURCE: EN ISO 18674-4]

3.1.8.7 hydraulic conductivity

ratio of the average velocity of a fluid through a cross-sectional area (Darcy's velocity) to the applied hydraulic gradient

Note 1 to entry: Hydraulic conductivity can be anisotropic.

3.1.8.8 absolute permeability

property that quantifies the ability of porous ground to permit the flow of fluids through its pore spaces

Note 1 to entry: Also known as intrinsic permeability or specific permeability. The terms 'hydraulic conductivity' and 'absolute permeability' are interchangeable if the ground is fully saturated

3.1.8.9 transmissivity

the rate at which water passes through a unit width of an aquifer under unit hydraulic gradient

3.1.9 Terms relating to geothermal properties

3.1.9.1 thermal conductivity

ratio of the thermal flux through a cross-sectional area (Fourier's law) to the applied thermal gradient

3.1.9.2 heat capacity or specific heat capacity

capacity of a material to store thermal energy

3.1.9.3 thermal diffusivity

ratio of the thermal conductivity to the specific heat capacity

3.2 Symbols and abbreviations

The symbols in EN 1997-1 and the following apply to this document.

<Drafting note: this table will be updated before the final PT6 submission. Kept for the drafting process, the table will be deleted in final version depending on the drafting rules.>

3.2.1 Latin upper case letters

B	pore water pressure coefficient
C	thermal capacity per unit volume
C_{c}	compression index
$C_{\mathrm{C,PSD}}$	coefficient of curvature
C_{i}	non-dimensional coefficient of correlation
$C_{\mathrm{U,PSD}}$	coefficient of uniformity

C_α coefficient of secondary compression

D disturbance factor for rock mass

D_n particle size that n % by weight are smaller than

E_{BJT} Young's modulus from a borehole jack test according to EN ISO 22476-7

E_{cyc} cyclic Young's modulus

E_{DMT} Young's modulus from a flat dilatometer test according to EN ISO 22476-11

E_{FDP} Young's modulus from a full displacement pressuremeter test according to EN ISO 22476-8

E_i Young's modulus of intact rock

E_M Young's modulus from a Ménard pressuremeter test according to EN ISO 22476-4

E_{OED} Young's modulus from an oedometer test according to EN ISO 17892-5

E_{PBP} Young's modulus from a pre-bored pressuremeter test according to EN ISO 22476-5

E_{PLT} Young's modulus from a plate loading test according to EN ISO 22476-13

E_{rm} Young's modulus of rock mass

E_s Young's modulus of soil

E_{SBP} Young's modulus from a self-boring pressuremeter test according to EN ISO 22476-6

E_{sec} secant Young's modulus

E_{tan} tangent Young's modulus

E_u undrained Young's modulus

G_0 shear modulus at very small strain

G_M shear modulus from a Ménard pre-bored pressuremeter test according to EN ISO 22476-4

G_{PBP} shear modulus from a pre-bored pressuremeter test according to EN ISO 22476-5

G_{SBP} shear modulus from a self-boring pressuremeter test according to EN ISO 22476-6

H_{800} depth of the bedrock formation identified by a shear wave velocity vs greater than 800 m/s

I_A activity index

I_D density index of coarse soil

I_f fracture spacing

I_L liquidity index of fine soil according to EN ISO 17892-12

I_P plasticity index of fine soil according to EN ISO 17892-12

K_{bulk} bulk modulus

K_D DMT horizontal stress index as per EN ISO 22476-11

K_0 at-rest earth pressure coefficient

K_{PMT} calibration factor for PMT test results

K_r rock creep index

L length of test section in the thickness of aquifer

N_{60} SPT blow count normalized for energy as per EN ISO 22476-3

$(N_1)_{60}$ SPT blow count normalized for overburden pressure and energy as per EN ISO 22476-3

Q coefficient that depends on the crushability of the material

T transmissivity

$V_{S,H800}$ equivalent value of the shear wave velocity of the soil column above the depth of the bedrock formation

3.2.2 Latin lower case letters

a non-dimensional material parameter in Hoek-Brown envelope

$c'_{X,Y}$ effective cohesion measured at a condition X (p for peak, cs for critical state, r for residual) by a specific test Y (UCT, UU, TX, FVT, etc.)

c_h coefficient of horizontal consolidation

$c_{u,p}$ peak undrained shear strength

$c_{u,rmd}$ remoulded undrained shear strength

$c_{u,X,Y}$ undrained shear strength measured at a condition X (p for peak, cs for critical state, r for residual) by a specific test Y (UCT, UU, TX, FVT, etc.)

e void ratio of soil

e_{max} maximum void ratio of soil

e_{min} minimum void ratio of soil

f_0 fundamental frequency of a soil deposit

f_1 soil property function defining the relationship between shear strength and soil suction

h_w pressure head

k absolute permeability

m coefficient that depends on the relevant shear mode to failure

m_b non-dimensional material parameter for rock mass in the Hoek-Brown envelope

m_i non-dimensional material parameter for rock material in the Hoek-Brown envelope

m_v one dimensional compressibility

p' mean principal effective stress

p_1 corrected pressure at the origin of the pressuremeter modulus pressure range (see EN ISO 22476-4)

p_a atmospheric air pressure

p_{LM} limit pressure from Ménard pressuremeter test according to EN ISO 22476-4

p_{ref} reference pressure usually equal to 100 kPa

q_c cone tip resistance measured as per EN ISO 22476-1

q_n net cone resistance (= $q_t - \sigma_{v0}$)

q_t corrected cone resistance as per EN ISO 22476-1

s non-dimensional material parameter for Hoek-Brown envelope

v_P compressional wave velocity

v_S shear wave velocity

w water content

w_L liquid limit of soil

w_P plastic limit of soil

w_S shrinkage limit of soil

3.2.3 Greek upper case letters

Δu_2 excess pore water pressure measured at the gap between cone tip and friction sleeve as per EN ISO 22476-1

3.2.4 Greek lower case letters

γ shear strain

γ_w weight density of groundwater

δ_x horizontal incremental displacement of a specimen in direct shear

δ_z vertical incremental displacement of a specimen in direct shear

ε strain

η dynamic viscosity of a fluid

κ thermal diffusivity

λ thermal conductivity

ν Poisson's ratio

ν_0 Poisson's ratio at very small strain

ρ bulk mass density

σ normal stress

σ' effective normal stress

σ_{ci} uniaxial compressive strength of intact rock

σ_{fl} flexural strength of rock material

σ'_{h0} in-situ horizontal effective stress

σ_n normal stress acting on a discontinuity

σ'_p preconsolidation pressure

σ_t tensile strength of soil or rock

σ_v Total vertical stress

σ_{v0} in-situ vertical total stress

σ'_{v0} in-situ vertical effective stress

σ_1 major principal stress

σ_3 minor principal stress

τ shear stress

τ_p peak shear strength of discontinuity

φ angle of friction

φ' angle of effective friction

φ_b base angle of friction of a rock surface

φ'_p angle of peak effective friction

φ'_X angle of effective friction measured at a condition X (p for peak, cs for critical state, r for residual)

$\varphi'_{X,Y}$ effective stress angle of internal friction measured at a condition X (p for peak, cs for critical state, r for residual) by a specific test Y (UCT, UU, TX, FVT, etc.)

3.2.5 Abbreviations

BDP Borehole Dynamic Penetration (test)

BE bender element

BJT Borehole Jack Test

BST Borehole Shear Test

CD crack damage (stress level)

CDSS Cyclic Direct Simple Shear

CI crack initiation (stress level)

CPT Cone Penetration Test

CPTU Cone Penetration Test with pore water pressure measurement (piezocone test)

CRS constant rate of strain

CTS cyclic torsional shear

CTxT cyclic triaxial test

DMT Flat Dilatometer Test (also known as Marchetti Dilatometer Test)

DP Dynamic Penetration (Test)

DSS direct simple shear

DST Direct Shear Test

FDP full displacement pressuremeter

FDT Flexible Dilatometer Test

FVT Field Vane Test

GIR Ground Investigation Report

GSI Geological Strength Index

IL incremental loading oedometer test

IST interface shear test

JCS joint compressive strength of a discontinuity

JRC Joint Roughness Coefficient

MPM Ménard pressuremeter

MQC minimum quality class of sample suitable for a test (see Annex F)

MR modulus ratio

MWD measuring while drilling

NDP Nationally Determined Parameter

OCR over-consolidation ratio

OMR	Organic matter content
OED	oedometer test
PLT	Plate Loading Test
PBP	pre-bored pressuremeter
PMT	Pressuremeter Test
RC	resonant column
RQD	Rock Quality Designation
SBP	self-boring pressuremeter
SCR	Solid Core Recovery
SPT	Standard Penetration Test
TCR	Total Core Recovery
TxT	triaxial test
UCS	Unconfined Compressive Strength
UCT	Unconfined Compression Test

4 Ground Model

4.1 General

(1) <REQ> A Ground Model shall comprise the geological, hydrogeological, and geotechnical conditions at the site, based on the ground investigation results.

NOTE 1. Geological conditions include, but are not limited to: the description of the site geomorphology, the lithology of the geotechnical units, the potential presence and level of a rockhead, geometrical and geotechnical properties of discontinuities and weathered zones;

NOTE 2. Hydrogeological conditions address surface, groundwater and piezometric levels, including its potential variations with time, potential water flows and the presence of other fluids or gases affecting the site;

NOTE 3. Geotechnical conditions include, but are not limited to: the disposition of the geotechnical units and the mechanical behaviour of the ground described through the geotechnical properties of the geotechnical units.

(2) <REQ> The detail and the extent of the Ground Model shall be consistent with the Geotechnical Category and the zone of influence of the structure.

(3) <REQ> The Ground Model shall be progressively developed and updated based on potential new information.

(4) <REQ> The Ground Model shall include the derived values of the relevant ground properties for all the geotechnical units encountered in the zone of influence.

(5) <REQ> Variability and uncertainty of geological, hydrogeological and geotechnical conditions and properties shall be included in the Ground Model.

(6) <REQ> The Ground Model shall be documented in the Ground Investigation Report.

NOTE 1. The Ground Model is the main output of the Ground Investigation.

NOTE 2. The Ground Model forms the basis for development of the Geotechnical Design Model (see EN 1997-1, 4.2.3)

4.2 Derived values

(1) <REQ> Derived values of properties of a geotechnical unit shall be established from data gathered during the desk study, site inspection, preliminary and design investigation, and monitoring of the ground and structures.

(2) <REQ> Empirical correlations and theories used to obtain the derived values shall be documented in the Ground Investigation Report.

(3) <RCM> The Ground Investigation Report shall record whether empirical correlations and theories used in parameter derivation are intended to provide average, superior, or inferior values.

(4) <RCM> The information given for each correlation should specify either directly or through reference:

- the materials to which they apply, specified by their classification according to EN ISO 14688-2 and 14689, or their physical and chemical properties;
- the database that supports the correlation;
- the estimated transformation errors.

(5) <RCM> Site-specific data should be used to support generic correlations.

NOTE 1. Site specific data generally results in smaller correlation errors.

(6) <RCM> Derived values of ground mass properties that are determined from test results on samples should be adjusted for scale effects.

5 Ground investigation

5.1 General

(1) <REQ> The Ground Investigation shall be planned so that it collects all the information needed to define the geotechnical units that influence the anticipated design situations.

NOTE 1. Guidance on suitable ground investigation techniques is given in Annex B.

NOTE 2. The geotechnical units include soil layers, rock masses and any fill in place prior to the investigation works.

(2) <REQ> The scope, level of detail, and accuracy of the ground properties to be obtained during the Ground Investigation shall be defined before the start of the investigation.

NOTE 1. The extent of the ground investigation and the required sample quality classes affect the precision of the determined ground properties.

NOTE 2. Guidance on the confidence levels of the results from different tests is given in Annex B.

(3) <RCM> The Ground Investigation should be carried out in phases to progressively increase knowledge, improve reliability, and reduce uncertainty of the information about the ground.

(4) <REQ> The Ground Investigation shall identify the ground materials and groundwater conditions within the zone of influence.

(5) <RCM> The Ground Investigation should identify rockhead, transition zones between geotechnical units and weathered zones where present.

NOTE 1. A distinct rockhead can be difficult to define in cases with transition or weathered zones going from soil to rock.

(6) <RCM> The minimum amount of Ground Investigation should be specified according to the Geotechnical Category, as given in Table 5.1.

Table 5.1. Minimum amount of Ground Investigation for different Geotechnical Categories

Geotechnical Category	Minimum amount of Ground Investigation
GC3	All items given below for GC1, GC2 and, in addition: – sufficient investigations to capture the variability of the ground; – sufficient investigations to capture the relevant properties for all geotechnical units using more than one ground investigation method; – sufficient investigations to capture the scatter of the properties of each geotechnical unit
GC2	All items given below for GC1 and, in addition: – sufficient investigations to identify all geotechnical units in the zone of influence; – determination of relevant ground properties by field and laboratory testing and by monitoring.
GC1	All items given below: – desk study of the site, review of comparable experience; – site inspection.

(7) <REQ> When sufficiently reliable ground properties for GC1 cannot be determined from desk study and site inspection only, additional ground investigation shall be performed.

(8) <PER> Ground investigations may also include other laboratory or field investigation tests than are specified in EN 1997-2, using methods adapted to the local conditions.

(9) <RCM> The result of the Ground Investigation should be presented together with statements on any limitations, discrepancies, uncertainties, or gaps in the data, as well as if deviation has been made from standard procedures for the investigation.

5.2 Content of Ground Investigation

5.2.1 Desk study

(1) <REQ> A desk study shall be carried out.

NOTE 1. Guidance on the Desk Study is given in Annex C.

(2) <RCM> The desk study should be carried out at an early stage of the ground investigation.

(3) <REQ> The desk study should identify potential hazards in the ground.

NOTE 1. Potential hazards include, among others: aggressive ground, aggressive groundwater, high-sensitivity clays (quick clays), or presence of gases, cavities and unexploded ordnance, potential seismic activity.

(4) <RCM> The desk study should identify the presence of any existing structure that can influence or can be affected by the new structure.

NOTE 1. This includes hidden structures, e.g. foundations, cables, pipes, tunnels, and potential archaeological finds.

(5) <REQ> The desk study shall determine a preliminary zone of influence.

(6) <REQ> Until the Geotechnical Complexity Class has been established by the desk study, GCC3 shall be assumed.

5.2.2 Site inspection

(1) <REQ> The site shall be visited and inspected before testing for design or execution is performed.

NOTE 1. Guidance on site inspection is given in Annex C.

NOTE 2. See EN 1997-1, 10.3, for inspection during execution and EN 1997-1, 10.6, for inspection as part of the Observational Method.

(2) <RCM> The findings of the site inspection should be cross-checked against the information gathered by the desk study.

(3) <RCM> The site inspection should cover the entire surface area of the zone of influence.

(4) <RCM> Inspection and geotechnical mapping of visible rock surfaces should be included into the site inspection.

(5) <REQ> The findings of the site inspection shall be recorded.

5.2.3 Preliminary investigation

(1) <RCM> Preliminary investigation should be performed to identify key issues that need to be addressed by the testing for design and execution.

(2) <RCM> Preliminary investigation should enable assessment of:

- groundwater conditions, including aquifers, aquitards, or aquicludes;
- geotechnical hazards present at the site, including landslide and seismic hazards;
- valuable or historical constructions;
- suitable positioning of the structure;
- preliminary design of the geotechnical and related structures;
- stability of any excavations or underground openings;
- potential impacts of the proposed construction on the surroundings, including neighbouring buildings, structures, and sites;
- potential need for and suitability of different ground improvement methods;
- sources of construction materials.

(3) <RCM> Preliminary investigation should provide information concerning:

- material types and their disposition;
- rockhead within the zone of influence;
- discontinuities and their geometry;
- piezometric levels and groundwater pressures;
- preliminary values of the strength and stiffness properties of the geotechnical units;
- potential occurrence of natural or anthropogenic contamination in the ground or groundwater.

5.2.4 Design investigation

(1) <REQ> Design investigation shall provide ground properties to address all relevant design issues.

(2) <RCM> Design investigation should identify ground and groundwater conditions that could influence the behaviour and execution of the structure or adversely affect its durability.

(3) <RCM> In addition to **Fel! Hittar inte referenskälla.** (2) and (3), design investigation should provide:

- data for a description of the geotechnical units;
- values of physical and chemical ground and groundwater properties;
- values of strength and stiffness properties of the ground;
- groundwater conditions and ground hydraulic properties; and
- values of thermal properties of the ground.

(4) <RCM> The information on groundwater conditions should include:

- the depth, thickness, and extent of water-bearing geotechnical units in the ground;
- the groundwater pressure distribution;
- piezometric levels and their variation over time;
- the hydraulic conductivity and its possible anisotropy for each geotechnical unit;
- the chemical composition and temperature of groundwater.

5.2.5 Monitoring

(1) <RCM> Monitoring should be used to obtain information on ground behaviour and on groundwater conditions needed for design.

NOTE 1. See EN 1997-1, 10.4.

5.3 Ground investigation techniques

5.3.1 Site inspection techniques

(1) <RCM> Site inspection should be performed using one or more of the following techniques:

- visual observation,
- topographic mapping
- photogrammetric mapping,
- airborne and on-ground video tools,
- geological mapping
- geotechnical mapping.

5.3.2 Preliminary or design investigation techniques

5.3.2.1 Exploratory holes and openings

(1) <RCM> Ground and groundwater conditions should be determined using one or more of the following techniques:

- test pits, shafts, and exploratory headings;
- percussive and rotary boreholes;

- logging of borehole walls, excavations, and exposures;
- probing.

(2) <RCM> During exploratory activities, sampling and groundwater measurements should be performed.

5.3.2.2 Field investigation techniques

(1) <RCM> Field investigation tests should be selected from the standards given in Table 5.2.

Table 5.2. Field investigation tests and corresponding standards

Type of test	Test standard
Field testing	EN ISO 22476 (all parts)
Geophysical testing	EN ISO xxx (in preparation)
Geohydraulic testing	EN ISO 22282 (all parts)
Geothermal testing	EN ISO 17628 (all parts)
Testing of geotechnical structures	EN ISO 22477 (all parts)
In-situ stress measurements	ISRM Suggested Methods (all parts)

(2) <PER> Field investigation tests other than those given in Table 5.2 may also be carried out, provided the test standard used is recorded in the Ground Investigation Report.

(3) <RCM> Geophysical tests should be used to identify:

- ground conditions (stratigraphy, lithology and any lateral variations, weathered or fractured zones, presence of cavities);
- buried objects (utilities, services, artefacts, archaeological structures) that could interfere with the construction works;
- groundwater conditions;
- parameters for the estimation of porosity, hydraulic conductivity and stiffness.

(4) <REQ> Disposition of geotechnical units using geophysical testing shall be compared with the results of direct ground investigation techniques and adjusted accordingly.

5.3.2.3 Laboratory testing

(1) <RCM> Laboratory tests should be selected from the standards given in Table 5.3.

Table 5.3. Laboratory test standards

Type of test	Test standard
Laboratory testing of soil	EN ISO 17892 (all parts)

Laboratory testing of rock	ISRM Suggested Methods

(2) <PER> Laboratory tests other than those given in Table 5.3 may also be carried out, provided the test standard used is recorded in the Ground Investigation Report.

(3) <RCM> Laboratory testing should be carried out shortly after sampling.

NOTE 1. This is particularly relevant when stiffness properties are to be determined.

(4) <RCM> Preservation and storage of samples for laboratory testing should comply with EN ISO 22475-1.

5.3.3 Instrumentation for monitoring

(1) <REQ> Geotechnical monitoring by field instrumentation shall comply with EN ISO 18674.

(2) <REQ> The instruments shall be maintained throughout the period identified in the Monitoring Plan specified in EN 1997-1, 10.4.

5.3.4 Back analysis

(1) <PER> Monitoring results may be obtained during execution or from existing, trial, or failed geotechnical structures.

(2) <PER> Information on the behaviour of the ground may be gathered from back-analysis of monitoring results.

(3) <PER> Parameters for calculation models may be determined from back-analysis of monitoring results.

5.4 Planning of preliminary or design investigations

5.4.1 General

(1) <REQ> The planning of investigation shall be appropriate for the purpose of the investigation and the anticipated geotechnical units.

(2) <REQ> The planning of investigation shall provide:

- positioning and depth of the field investigation and samplings locations;
- field investigation techniques to be used at each location;
- required sample quality and therefore the samplers to be used (see 7, 8, 9, 10 and EN 22475-1);
- laboratory testing to be carried out;
- measurements of groundwater pressures and piezometric levels to be made;
- details of instrumentation to be installed;
- standards to be applied to all aspects of the works.

(3) <REQ> The type and extent of the techniques to be used in the investigation shall be based on the results of the desk study, the site inspection, previous knowledge of the geotechnical structure to be designed, the Geotechnical Category, and the zone of influence.

(4) <REQ> The extent of the investigation shall cover the:

- zone of influence of the structure;
- zone of influence of temporary works elements;
- depth of effect of any dewatering works on groundwater conditions;
- presence of any destabilising features in the ground on or around the site.

NOTE 1. The zone of influence under cyclic, dynamic or seismic actions can extend significantly beyond the zone of influence under static loading.

5.4.2 Number of investigation locations and laboratory tests

(1) <REQ> The number of investigation locations and laboratory tests in a geotechnical unit shall be determined from:

- the variability of the ground;
- previous documented experience in ground with the same variability;
- the Geotechnical Category.

(2) <RCM> The number of investigation locations and laboratory tests should be appropriate for the ground investigation techniques used and typical uncertainty levels of the test results.

NOTE 1. The use of more than one ground investigation technique can reduce uncertainty in derived values.

NOTE 2. Guidance on the suitability and applicability of different ground investigation techniques is given in Annex B.

(3) <RCM> The number of samples for laboratory tests should be appropriate for the material to be sampled and potential sample disturbance.

5.4.3 Spacing of investigation locations

(1) <REQ> The spacing of investigation locations shall be appropriate for the:

- variation of the ground conditions;
- variation of the geotechnical units;
- variation of the groundwater conditions; and
- critical elements of the structure.

(2) <RCM> Investigation locations should be spaced no greater than X_{max} apart on plan.

NOTE 1. Values of X_{max} for structures in Geotechnical Category 2 are given in Table 5.4(NDP), unless the National Annex gives different values.

NOTE 2. For structures not in Geotechnical Category 2, the value of X_{max} can be specified by a relevant authority or, where not specified, agreed for a specific project by appropriate parties.

(3) <RCM> The number of investigation locations should be no less than N_{min}.

NOTE 1. Values of N_{min} for structures in Geotechnical Category 2 are given in Table 5.4(NDP), unless the National Annex gives different values.

NOTE 2. For structures not in Geotechnical Category 2, the value of N_{min} can be specified by a relevant authority or, where not specified, agreed for a specific project by appropriate parties.

(4) <REQ> For structures in Geotechnical Category 3, the maximum spacing X_{max} shall be not greater than and the minimum number of investigation locations N_{min} shall be not less than, the values given for structures in Geotechnical Category 2.

(5) <REQ> In cases where more than one type of investigation technique is planned, the investigation locations shall be separated by sufficient distance to avoid any interference.

Table 5.4(NDP). Maximum spacing and minimum number of investigation locations for structures in Geotechnical Category 2

Structures		**Maximum spacing** X_{max}	**Minimum number**[a] N_{min}
Low-rise structures		30 m	3
High-rise structures	4-10 storeys	25 m	3-4[b]
	11-20 storeys	20 m	3-5[b]
	>20 storeys	15 m	3-6[b]
Estate roads, parking areas and pavements		40 m	2
Silos and tanks		15 m	3
Bridges piers and abutments		1 per pier/base	
Power lines		1 per pylon	
Wind turbines		2 per turbine	
Retaining structures		150 m	-
Slopes and cuttings	< 3 m high	100 m	-
	≥ 3 m high	50 m	-
Embankments and reinforced fill structures	< 3 m high	200 m	-
	≥ 3 m high	100 m	-
Excavations in urban areas > 5 m deep from ground surface		25 m	3

[a] Where no spacing or number of locations is given this should be assessed on a project-specific basis.
[b] Underlined numbers are more appropriate for difficult structures

(6) <PER> Where documented previous knowledge, local experience or the results of preliminary investigations indicate that the ground is uniform or that the strength and stiffness properties are sufficient for the proposed structure, a wider spacing or fewer investigation locations may be used, provided the reduced intensity is justified in the Ground Investigation Report.

(7) <REQ> Where documented previous knowledge, local experience or the results of preliminary investigations indicate that the ground is highly variable, closer spacing and more investigation locations shall be used.

NOTE 1. Further guidance on the extent of ground investigation for specific geotechnical structures is given in EN 1997-3.

5.4.4 Positioning of investigation locations

(1) <REQ> The depth and positioning of investigation locations shall be sufficient to identify the disposition of all geotechnical units and their properties within the zone of influence of the structure.

NOTE 1. The zone of influence for the different geotechnical structures is specified in EN 1997-3.

(2) <RCM> The positioning of investigation locations should be based on the results of the Desk Study and Site Inspection and be selected according to the:

– presence of critical locations relative to the shape, structural behaviour, and expected load distribution of the structure;

- stability of slopes or cuttings and steps in the ground;
- necessity of investigation locations outside the site so that the stability of the slopes or cuttings can be assessed;
- preventing hazards to the structure, its execution, or the surroundings;
- requirements for groundwater and monitoring instruments; and
- potential monitoring during and after execution.

(3) <REQ> The position and elevation of investigation locations shall be recorded in the Ground Investigation Report, together with details of the grid reference system and geodetic reference datum used.

5.4.5 Sampling and laboratory testing

(1) <RCM> Samples should be taken from all geotechnical units.

(2) <REQ> Sampling and groundwater measurements shall comply with EN ISO 22475-1.

(3) <REQ> Planning of the sampling and selection of the equipment for taking each sample shall be appropriate for the:

- parameters to be measured;
- tests to be carried out;
- minimum sample quality class;
- material to be sampled;
- required diameter and mass of the sample; and
- appropriate sampler.

(4) <RCM> Before taking specimens for testing, the quality of recovered samples should be assessed and recorded in the Ground Investigation Report.

NOTE 1. See Table F.1 for guidance on verifying sample quality.

(5) <REQ> Soil samples obtained using Category A samplers (as defined in EN ISO 22475-1) shall be handled to avoid deformation, desaturation, or swelling of samples during transport and storage.

(6) <PER> Reconstituted or reconsolidated specimens may be used to determine ground properties.

(7) <RCM> Reconstituted specimens of coarse soils should have approximately the same composition, bulk mass density, and water content as the in-situ material.

(8) <REQ> The procedure used to reconstitute soil specimens shall be recorded in the Ground Investigation Report.

(9) <REQ> Planning of laboratory testing shall consider:

- the selection of test samples;
- the conditions of storage before testing;
- maximum allowed time between sampling and laboratory testing.
- whether desiccated samples are to be re-saturated and by which technique;
- the number of tests required per geotechnical unit;
- whether parallel tests are to be run on the same geotechnical unit.

6 Description and classification of the ground

6.1 General

(1) <REQ> Natural exposures, artificial exposures created as part of the investigation and all samples recovered in the investigation shall be inspected and described.

(2) <RCM> The description and classification of soils should comply with EN ISO 14688 (all parts).

(3) <RCM> The description and classification of rock and its weathered profile should comply with EN ISO 14689.

NOTE 1. Particular care is necessary in describing the transition between soil and rock at rockhead.

(4) <PER> Rock mass classification systems may be used for placing a rock mass into groups or classes and assigning a unique description (or value) to it on the basis of similar properties or characteristics.

(5) <PER> Rock mass classification systems may be used to determine the strength parameters in accordance with 8.1.

(6) <REQ> Classification of a site for seismic purposes shall comply with EN 1998-1.

(7) <RCM> Classification of materials and fill for earthworks should comply with EN 16907-2.

(8) <RCM> The classification of materials should be adjusted in accordance with results from different test types and comparable experience.

(9) <RCM> Anisotropy of the ground and corresponding properties should be described.

6.2 Discontinuities and weathered zones

(1) <RCM> Potential foliation of rock and of hard soils should be identified.

(2) <REQ> Discontinuities and weathered zones shall be defined by geometrical and geotechnical properties.

(3) <REQ> Geotechnical properties of discontinuities and weathered zones shall include, but not limited to state and physical properties, such as apertures, interlocking, infill, thickness, roughness, smoothness, weathering, alteration.

(4) <REQ> State and physical properties affecting strength of the discontinuities and weathered zones shall be defined and described as specified in 7 and 8.

(5) <PER> Discontinuities may be grouped by sets with similar properties on dip and direction.

(6) <REQ> The presence of groundwater in discontinuities and possible freeze-thaw conditions shall be determined.

(7) <REQ> The determination of groundwater pressures in the discontinuities should comply with 11 and EN 1997-1, 6.

(8) <REQ> The properties of any infill and hydraulic conductivity in discontinuities and weathered zones shall be determined according to 7 and 11.

NOTE 1. Typical infill properties of discontinuities are organic matter content, swelling potential and presence of crushed material.

NOTE 2. See Annex E.5 for guidance on determining the Geological Strength Index.

7 State, physical, and chemical properties

7.1 State properties

7.1.1 General

(1) <RCM> Determination of state properties should comply with one or more of the standards given in Table 7.1.

(2) <RCM> State properties should be determined on specimens representing each soil or rock type in the sample.

Table 7.1 — Field and laboratory tests to determine state properties

Property	Test	Test standard	MQC	Comments on suitability and interpretation
Bulk mass density (ρ)	Linear measurement method; immersion in fluid method; fluid displacement method	EN ISO 17892-2	2	Testing method used considering soil type and possible sample disturbance Density of a coarse soil is generally only approximate
	Water absorption coefficient by capillarity	EN 1925	2	-
	Immersion in fluid method; fluid displacement method	EN 1936	2	Determination of real density and apparent density, and of total and open porosity
	Sand replacement method	ISO 11272:2017	-	In situ test
	Loose bulk density and voids	EN 1097-3	3	Suitable for coarse soils and aggregates
	Nuclear gauge	See Table B.4	-	Presence of nuclear source as a hazard
	Electrical density method	See Table B.4	-	-
	Dynamic cone penetration	EN ISO 22476-2	-	Use of correlations
Water content (w)	Oven drying	EN ISO 17892-1	3	Check storage method of samples Standard oven-drying method not appropriate for gypsum, organic soil and soil with closed pores filled with water; precautions may be needed Report presence of gypsum, organic soil
	Water content	See Table B.4	2	For rock
	Oven drying in a ventilated oven	EN 1097-5	3	Suitable for aggregates
	Neutron depth probe method	ISO 10573	-	Determination of water content in the unsaturated zone
Porosity	Mercury intrusion porosimetry for soil	See Table B.4	2	Use of mercury in the laboratory depends on local regulation
	Porosity of rock by saturation and caliper	See Table B.4	-	Determination of porosity and density of rock
	Porosity of rock by saturation and buoyancy	See Table B.4	-	Determination of porosity and density of rock
	Water method	EN 1936	3	Determination of real density and apparent density, and of total and open porosity
Saturation				Determination of saturation is based on water content, bulk mass density and particle density values
Density index	-	EN 14688-2	4	Applicable to coarse soils
	Cone Penetration Test	EN ISO 22476-1	-	-
	Dynamic Penetration Test	EN ISO 22476-2	-	-
	Standard Penetration Test	EN ISO 22476-3	-	-
	Ménard Pressuremeter Test	EN ISO 22476-4	-	-
	Weight Sounding Test	EN ISO 22476-10	-	-
	Borehole dynamic probing	EN ISO 22476-14	-	-

7.1.2 Bulk mass density

(1) <RCM> Determination of the bulk mass density of soil and rock should comply with one or more of the standards given in Table 7.1.

(2) <REQ> A distinction shall be made between the bulk mass density of rock in-place and the bulk mass density of excavated or blasted rock.

NOTE 1. A given mass of rock in place will occupy up to 40% greater volume after excavation or blasting.

(3) <REQ> Test specimens used to determine bulk mass density of soil shall be at least Quality Class 2 as defined in EN 22475-1.

(4) <REQ> The evaluation of the test results shall be adjusted for potential sample disturbance.

(5) <REQ> The standard procedure used for such adjustment shall be defined and reported in the Ground Investigation Report.

(6) <RCM> Additional tests should be performed if test results fall outside the typical range of values, taking into account mineralogy and organic matter content.

7.1.3 Water content

(1) <RCM> Determination of water content of soil and rock should comply with one or more of the standards given in Table 7.1.

(2) <REQ> Determination of water content of soil specimens shall comply with EN ISO 17892-1.

(3) <RCM> The water content of rock material should be determined according to the ISRM suggested method.

NOTE 1. See ISRM (2007), Suggested Methods for Determining Water Content, Porosity, Density, Absorption and Related Properties and Swelling and Slake-Durability Index Properties.

(4) <REQ> Soil specimens for measuring the water content shall be at least Quality Class 3 as defined in EN 22475-1.

(5) <RCM> The extent to which the water content measured in the laboratory on a soil sample is representative of the in-situ value should be recorded in the Ground Investigation Report.

(6) <PER> Water content may be determined indirectly using field tests provided the testing, reporting, and interpretation procedures are recorded in the Ground Investigation Report.

NOTE 1. See ASTM D6780/D6780M–12 for the Time Domain Reflectometry technique.

7.1.4 Porosity

(1) <RCM> Determination of the porosity and pore size distribution of soil samples should comply with one or more of the standards given in Table 7.1.

(2) <RCM> Determination of the porosity of rock samples should comply with the appropriate ISRM Suggested Method.

7.1.5 Saturation

(1) <RCM> Determination of the degree of saturation of an unsaturated specimen should comply with one or more of the standards given in Table 7.1.

(2) <REQ> When specimens are not fully saturated, suctions present in the specimen shall be measured when relevant for the design situation.

NOTE 1. Further information on the determination of the volume of water is given in EN ISO 17892-1 and of the volume of void in the EN ISO 17892-2 and EN ISO 17892-3.

NOTE 2. Further information on the determination of the volume of water is given in ASTM D-5298.

NOTE 3. Further information on the determination of water suction height in aggregates is given in EN 1097-10.

7.1.6 Density index

(1) <RCM> Determination of the density index of coarse soils should comply with one or more of the standards given in Table 7.1.

7.1.7 In-situ stress state

(1) <RCM> Determination of the in-situ stress state of soils and rock should comply with one or more of the standards given in Table 7.2.

Table 7.2 — Field and laboratory tests to determine in-situ stress parameters

Parameter	Test	Test standard	MQC	Comments on suitability and interpretation
In-situ stress state: horizontal total stress (σ_0, K_0)	Self-boring pressuremeter	EN ISO 22476-6	-	
	Ménard pressuremeter	EN ISO 22476-4	-	-
	Pre-bored pressuremeter	EN ISO 22476-5	-	Pre-bored expansion test Specific procedure
	Full displacement pressuremeter	EN ISO 22476-8	-	Insertion by full displacement Specific procedure is used
	Marchetti dilatometer	EN ISO 22476-11	-	Insertion by full displacement Choice of correlation depending of soil type
	Total pressure cells	EN ISO 18674-5	-	Insertion by full displacement
At-rest earth pressure coefficient (K_0)	Triaxial test	EN 17892-9	1	Specific procedure and local measurement needed
	Oedometer test	EN 17892-5	1	Specific procedure and local measurement needed
In-situ stress state component	Flat jack	See Table B.4	-	Measured stress component in a rock surface
In-situ stress state: minimum/maximum horizontal stresses and orientation/components of the stress tensor	Hydraulic fracturing in a borehole/ hydraulic tests on pre-existing joints	See Table B.4	-	Vertical axis often considered as one principal direction and vertical stress magnitude equals the weight of the overburden
In-situ stress state in rock: independent components of the stress tensor	Over coring in a borehole	See Table B.4	-	Elastic parameters of the rock required
Initial groundwater pressure	Piezometers	EN ISO 18674-4	-	-
Pre-consolidation pressures (σ'_p), over-consolidation ratio (OCR)	Incremental loading oedometer test	EN 17892-5	1	For unsaturated soils, specific procedures are used
	Constant rate of strain oedometer test	See Table B.4	1	-

(2) <PER> In-situ stress state may be assessed by geophysical testing.

(3) <REQ> Determination of in-situ vertical total stress shall be based on the bulk mass densities of the geotechnical units.

(4) <RCM> When determining the in-situ stress state of soils, results from self-boring pressuremeter tests should be considered more reliable than those from displacement and pre-bored pressuremeters and from laboratory tests.

NOTE 1. The self-boring pressuremeter method gives more reliable results, particularly for clayey soil.

(5) <PER> The stress in the ground may be determined using tests other than those given in Table 7.2 provided the testing, reporting, and interpretation procedures are recorded in the Ground Investigation Report.

(6) <PER> In the absence of reliable test results for soils, the coefficient of earth pressure at rest K_0 may be estimated from Formula (7.1):

$$K_0 = [1 - \sin(\varphi')] \cdot OCR^{0,5} \tag{7.1}$$

where:

K_0 is the at-rest earth pressure coefficient;

φ' is the angle of effective friction;

OCR is the over-consolidation ratio.

NOTE 1. See Mayne and Kulhawy (JGE ASCE, 1982) for further information.

7.2 Physical properties

7.2.1 Particle size distribution

(1) <REQ> The particle size distribution of soil shall be determined using particle size analysis.

(2) <RCM> Determination of the particle size distribution of soils should comply with one or more of the standards given in Table 7.3.

Table 7.3 — Laboratory tests to determine particle size properties

Property	**Test**	**Test standard**	**MQC**	**Comments on suitability and interpretation**
Grain size distribution curve Coefficient of uniformity ($C_{U,PSD}$) Coefficient of curvature (C_C)	Sieve method	EN ISO 17892-04	4	For particles larger than 0.063 mm (or closest sieve size available)
	Sedimentation method	EN ISO 17892-04	4	For particles smaller than 0.063 mm (or closest sieve size available) Carbonates and organic matter influence test results
	Laser diffraction	EN ISO 13320	4	For particle sizes from about 0,1 µm to 3 mm May be extended with modification
	X-Ray gravitational	ISO 13317-3		For particle sizes from about 0,5 to 100 µm Influence by the chemical composition of particles.

(3) <PER> In addition to (2), other tests than those given in Table 7.3 may be used to measure particle size provided they incorporate detection systems using density measurements or particle counters or they are calibrated against the sieve or sedimentation methods given in Table 7.3.

7.2.2 Consistency (Atterberg) limits

(1) <RCM> Determination of the consistency (liquid and plastic) limits of fine soils should comply with one or more of the standards given in Table 7.4.

(2) <RCM> Determination of the shrinkage limit of fine soils should comply with one or more of the standards given in Table 7.4.

NOTE 1. The shrinkage limit can be useful for determining swelling behaviour or to evaluate the volume change of soils in unsaturated conditions.

Table 7.4 — Laboratory tests to determine consistency limits

Property	Test	Test standard	MQC	Comments on suitability and interpretation
Plastic limit (w_P)	Thread method	EN ISO 17892-12	4	Organic matter influences test results
Liquid limit (w_L)	Fall cone	EN ISO 17892-12	4	Strongly sensitive to pore fluid salinity for w_L > 100 %
	Casagrande method	EN ISO 17892-12	4	Strongly sensitive to pore fluid salinity for w_L > 100 % Results may not be reliable for thixotropic soil
Methylene blue value (MBV)	Methylene blue test	EN 933-9+A1	4	For fine soils
Shrinkage limit (w_S)	Volumetric or linear method	See Table B.4	2	For fine soils

NOTE 1. When significant clay content is present in an organic sample, the classification is assisted by plotting a Casagrande diagram complying with ISO 14688-2 and determining soil particle density.

(3) <RCM> The specimens used to determine consistency limits should be at least Quality Class 4 as defined in EN ISO 22475.

7.2.3 Particle density

(1) <RCM> Determination of the density of solid soil particles should comply with EN ISO 17892-3 for soils or EN 1097-6 for aggregates.

(2) <RCM> Soil specimens for determining particle density should be at least Quality Class 4 as defined in EN ISO 22475.

(3) <RCM> The mineralogy of the soil, its organic matter, and its geological origin should be confirmed by further testing if, for a particular geotechnical unit, the measured values of the particle density are outside the range 2 500 to 2 800 kg/m^3.

7.2.4 Maximum and minimum void ratios

(1) <RCM> Determination of maximum and minimum void ratios and density at loosest and densest packing of coarse soils should be carried out as specified by the relevant authority or agreed for a specific project by the relevant parties.

NOTE 1. If the values of minimum and maximum void ratios are decisive for the design, a series of methods can be used to map them as the values vary with the applied method.

(2) <RCM> If the minimum or maximum void ratios of a coarse soil are not within the range 0.35 to 0.9, the particle size distribution should be checked.

(3) <PER> Maximum and minimum void ratios may be determined indirectly using field tests provided the testing, reporting, and interpretation procedures are recorded in the Ground Investigation Report.

7.2.5 Particle and rock block shape

(1) <RCM> Descriptive and quantitative representation of particle shape and morphology should comply with EN ISO 14688-1 and ISO 9276-6 for coarse and very coarse soils.

(2) <RCM> Particle shape should be characterized by three dimensionless ratios: sphericity (cf. eccentricity or plainness), roundness (cf. angularity) and smoothness (cf. roughness).

(3) <PER> Digital image analysis may be used to facilitate the evaluation of mathematical descriptors of particle shape including Fourier analysis, fractal analysis and other hybrid techniques.

(4) <RCM> Descriptive and quantitative representation of rock block shape and morphology should comply with EN ISO 14689.

7.2.6 Rock weathering and alteration, abrasivity, and degradation

(1) <RCM> Rock weathering and alteration, abrasivity, and degradation should be determined in accordance with one or more of the standards given in Table 7.5.

Table 7.5 – Laboratory tests to determine rock physical properties

Property	Test standard
Weathering and alteration	EN ISO 14689 See Table B.4
Abrasivity	See Table B.4
Degradation	EN ISO 14689

(2) <REQ> The state of weathering and alteration of discontinuities shall be recorded in the Ground Investigation Report.

7.2.7 Water density

(1) <PER> Determination of the density of water samples may be performed according to ASTM D1429-13

7.2.8 Soil dispersibility, erosion, and rock degradation

(1) <RCM> The dispersive and erodible characteristics of clayey soil and stability of rock when immersed in water should be identified according to one of standards given in Table 7.6.

NOTE 1. Usual tests for classifying soil for engineering purposes do not identify the dispersive characteristics of a soil.

NOTE 2. 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.

Table 7.6 — Field and laboratory tests to determine stability, dispersibility and erodibility properties

Property	Test	Test standard	MQC	Comments on suitability and interpretation
Stability	Immersion in water	ISO 14689:2018	1	Compares the disintegration of a rock specimen in plain water with a conventional description
Dispersibility	Double Hydrometer Test	See Table B.4	4	Compares the dispersion of clay particles in plain water without mechanical stirring with that obtained using a dispersant solution and mechanical stirring Qualitative evaluation
	Crumb Test	See Table B.4	2	Stability of soil aggregates subjected to the action of water Qualitative evaluation
	Pinhole test	See Table B.4	2	Need to consider specifying different compaction conditions for specimens Avoid drying of the specimen before testing Qualitative evaluation of internal erosion
Critical stress and erosion coefficient	Hole erosion test		2	Internal erosion on undisturbed or reconstituted specimens Hydraulic gradient should be specified according to the structure
	jet erosion test	See Table B.4	2	In-situ or laboratory on small surface Representativeness External erosion

(2) <REQ> The following shall be specified:

- the storage of samples such that their water content is preserved;
- testing procedures to be applied;
- the specimen preparation method.

(3) <REQ> Particle size distribution and consistency limits of the sample shall be reported.

(4) <RCM> Dispersibility and erodibility tests should not be used for soil with clay content of less than 10 % and with a plasticity index less than or equal to 4 %.

(5) <RCM> For the hole erosion and pinhole test, the compaction conditions of the soil specimens, wet or dry of optimum, and the mixing water (distilled versus reservoir water) should be specified.

(6) <RCM> For the double hydrometer test, a third hydrometer test should be specified if it appears necessary to study the effect of reservoir water on the soil in suspension.

(7) <RCM> The sensitivity of rocks that contain magnesium or calcium carbonate to dissolution and chemical weathering should be analysed.

(8) <RCM> The sensitivity to disintegration of rocks that are liable to break up or crumble away when subject to moisture, heat, frost, air, or internal chemical reaction of the component parts of rocks should be analysed.

7.2.9 Compactability

(1) <REQ> Soil compaction tests (Proctor and vibration tests) shall be used to determine the relationship between dry density and water content for a given compactive effort.

NOTE 1. In civil engineering works, compaction is widely used on soils intended to support mobile (roads, aerodromes, etc.) or static (building foundations, embankments, etc.) actions and on structures in earth (earth dams, embankments, etc.).

NOTE 2. EN 16907 covers the design of earthworks materials, execution, monitoring, and checking of earthworks construction processes to ensure that the completed earth-structure satisfies the geotechnical design.

NOTE 3. The laboratory determination of compaction can miss the macro structure due to scale effects.

(2) <RCM> The California Bearing Ratio (CBR) test should be used to evaluate the subgrade strength of roads and pavements.

(3) <RCM> The degree of compaction of soils should be determined using one or more of the tests given in Table 7.7.

Table 7.7 — Laboratory tests to determine compaction properties

Property	**Test**	**Test standard**	**MQC**	**Comments on suitability and interpretation**
Reference density and water content ($\rho_{d;max}$, w_{opt})	Proctor compaction	EN 13286-2	4	Unbound and hydraulically bound mixtures Limited in particle dimension to 20 mm
	Vibrating hammer Vibrating table	EN 13286-4 EN 13286-5	4	Suitable for coarse soils and aggregates
California bearing ratio (CBR) immediate bearing index and linear swelling	CBR test	EN 13286-47	4	Unbound and hydraulically bound mixtures Limited in particle dimension to 20 mm
Fragmentability and degradability	Evolution of particle size distribution after dynamic compaction or humidification drying of soil	See Table B.4	4	for aggregates

(4) <PER> Proctor compaction and CBR tests may be systematically combined.

(5) <PER> Proctor, modified Proctor, and vibration tests may be used to define the optimal water content to obtain the higher dry density during the compaction process.

(6) <PER> Fragmentability and degradability index may be determined to characterize the evolution of gravels during compaction, as specified by the relevant authority or agreed for a specific project by the relevant parties.

7.3 Chemical properties

7.3.1 General

(1) <RCM> Determination of the chemical properties of ground should comply with one or more of the standards given in Table 7.8.

NOTE 1. There are other chemical components that can cause an environment to be very aggressive to steel and concrete, for example magnesium and ammonium. The corresponding chemical testing is not covered in this standard.

Table 7.8 — Laboratory tests to determine chemical properties of ground

Property	Test	Test standard	MQC	Comments on suitability and interpretation
Mineralogy	X-Ray diffraction	EN 13925	3	
	Petrographic description (for natural stones)	EN 12407	3	For natural stones
	Simplified petrographic description (for aggregates)	EN 932-3	3	For aggregates
	Normal and UV light microscopy	-	3	For μm thin sections of rock specimens
Carbonate content	Loss in dry weight after reaction with hydrochloric acid	See Table B.4	3	-
	Volumetric method	EN ISO 10693	3	
Organic matter content (OCM)	Hydrogen peroxide reagents and dry weight after reaction	EN ISO 10694	4	Commonly used in geotechnical laboratories
	Determination of loss on ignition	EN 15935	4	Only suitable for estimating organic matter content for peats and organic sands
	Sulfochromatic oxidation	ISO 14235	4	Test not usually achievable in a geotechnical laboratory
	Dry combustion	EN 15963 ISO 10694	4	Methods given are combustion or acid dissolution. Test not usually achievable in a geotechnical laboratory
	Chemical analysis	EN 1744-1	4	For aggregates Unsuitable for soil due to high oven temperatures
Sulphate and sulphide content	Determination of sulphide content of rock	ISO 11048	3	
Hydrogen potential pH	Electrometric methods (acidity and alkalinity)	EN ISO 10390	3	
Chloride and other salt content	Mohr's method for water-soluble chlorides; Volhard's method for acid-soluble or water-soluble chlorides; electrochemical procedures.	EN 1744-5	3	For aggregates; similar test methods are also suitable for soils
Radioactivity	Geiger counter/measurement of radioactivity Gamma emitting radionuclides	ISO 19581	-	For radon content in rock

(2) <RCM> The corrosiveness of steel constructions in ground should be determined according to EN 12501-1.

(3) <RCM> Chemical tests should be used to classify the ground and to assess the detrimental effect of chemicals in the ground and groundwater on concrete, steel, timber, and on the ground itself.

NOTE 1. The tests are not intended for environmentally related purposes.

(4) <RCM> Chemical characterization of water should include, as a minimum, carbonate and carbon dioxide content, sulphate content, pH value, and magnesium content.

(5) <RCM> Determination of the chemical properties of water should comply with one or more of the standards given in given in Table 7.9.

Table 7.9 — Laboratory tests to determine chemical properties of groundwater

Property	Test	Test standard	MQC	Comments on suitability and interpretation
Carbonate content	-	See Table B.4	-	-
Chloride content	-	EN ISO 10304-1		-
Carbon dioxide content	-	EN 13577	-	Aggressive CO2 content Total CO2 content
Sulphate and sulphide content	-	EN 196-2	-	-
Dissolved magnesium content	Flame atomic absorption spectrometry	EN ISO 7980	-	Magnesium content of up to 5 mg/l
Calcium content		EN ISO 7980 EN ISO 14911	-	-
Conductivity	Specific electrical conductivity	EN 27888 ISO 7888	-	-
Dissolved oxygen		EN 25813 ISO 5813	-	-
Hydrogen potential (pH)	Electrometric methods (acidity and alkalinity)	EN ISO 10523 EN ISO 9963-1	-	Within the range pH 2 to pH 12 with an ionic strength below I = 0,3 mol/kg (conductivity: γ25 °C < 2 000 mS/m) solvent and in the temperature range 0 °C to 50 °C

(6) <PER> Disturbed soil samples may be used for the chemical tests, provided particle size and water content are representative of field conditions (Quality Classes 1 to 3).

7.3.2 Mineralogy

(1) <REQ> Determination of the mineralogical composition and petrographic description should comply with one or more of the standards given in Table 7.8.

(2) <RCM> Determination of the mineral type should comply with one or more of the standards given in Table 7.8.

(3) <RCM> Mineralogical 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.

(4) <RCM> In case of swelling soils or rocks, the clay content and dominant clay minerals should be determined according to 9.2.4.

7.3.3 Carbonate content

(1) <RCM> Determination of carbonate content should comply with one or more of the standards given in Table 7.8.

(2) <REQ> Carbonate content shall be calculated from the content of carbon dioxide measured on treatment of the soil with HCl to classify natural carbonate ground or as an index to indicate the degree of cementation.

NOTE 1. 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 ($CaCO3$).

(3) <PER> In soils and rocks with non-homogenous carbonate distribution, large samples may be riffled and crushed to provide representative test specimens.

(4) <RCM> In the presence of carbonates that do not dissolve using the standard solution of hydrochloric acid or during the specified time, alternative test methods should be used.

7.3.4 Organic matter content

(1) <RCM> Determination of the organic matter content of soils should comply with one or more of the standards given in Table 7.8.

(2) <REQ> In organic soil with less than 2% of clay particles and carbonate content, the organic matter content shall be determined from the loss on ignition at a controlled temperature.

(3) <RCM> In addition to (4), if the carbonate content is significant the temperature should not exceed 600 °C.

(4) <REQ> In organic soils with more than 2% of clay content, the organic content shall be derived from the loss on ignition at a controlled temperature as low as possible e.g. 550°C or even lower, but not less than 480°.

(5) <REQ> The organic matter content shall be reported as a percentage of original dry matter, also giving the method of determination.

7.3.5 Sulphate and sulphide content

(1) <REQ> The sulphate and sulphide content of ground and groundwater shall be determined and classified to define suitable precautionary measures against possible detrimental effects on concrete, steel, and timber and the swelling potential of the ground.

(2) <RCM> Determination sulphate content should comply with one or more of the standards given in Table 7.8.

(3) <REQ> The acid-soluble sulphate content shall be reported as the total sulphate content, where appropriate.

(4) <REQ> In non-homogeneous ground containing visible crystals of anhydrite or gypsum, large samples shall be crushed, mixed, and riffled to provide representative test specimens with the method of preparation selected by visual assessment.

7.3.6 Acidity and alkalinity

(1) <REQ> The pH value of groundwater and solutions of ground in water shall be determined to assess the possibility of excessive acidity or alkalinity.

(2) <RCM> Determination of acidity and alkalinity should comply with EN 16502.

(3) <REQ> 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.

(4) <REQ> The pH value of the ground suspensions and groundwater and the test method used shall be recorded in the Ground Investigation Report.

(5) <RCM> The evaluation should consider that, in some soils, the measured values can be influenced by oxidation.

7.3.7 Chloride content

(1) <REQ> The salinity of ground and groundwater shall be assessed to determine the water-soluble or acid-soluble chloride content.

NOTE 1. Salinity potentially adversely affects concrete, steel, other materials, and soil.

(2) <RCM> The source of the chloride content should be determined, whether from sea water or other sources.

(3) <REQ> The following shall be specified for each test or group of tests:

- whether water-soluble or acid-soluble chlorides shall be determined;
- whether or not the soil shall be dried.

7.3.8 Radioactivity

(4) <RCM> The presence of radioactivity should be determined according to ISO 19581.

7.3.9 Other chemical content

(1) <RCM> The presence of ammonium should be determined according ISO 7150-1.

(2) <RCM> The presence of carbon dioxide should be determined according EN 13577.

(3) <RCM> The presence of magnesium cation should be determined according EN ISO 7980.

8 Strength

8.1 Strength envelopes and parameters for soils and rocks

8.1.1 General

(1) <PER> Ground strength may be described in terms of total or effective stress using strength envelopes.

(2) <RCM> The applicable stress and strain range of each strength envelope shall be recorded in the Ground Investigation Report.

(3) <REQ> The applicable strength condition (peak, critical state, or residual) of each strength envelope shall be recorded in the Ground Investigation Report.

8.1.2 Strength envelopes for saturated soils and rock

(1) <PER> The shear stress at failure τ_f of saturated soils and rock may be determined from the Mohr-Coulomb envelope given in terms of effective stresses by Formula (8.1):

$$\tau_{\mathrm{f}} = c' + (\sigma - u)\tan\varphi' \tag{8.1}$$

where:

c' is the effective cohesion;

σ is the normal total stress on the failure plane;

u is the groundwater pressure;

φ' is the angle of effective friction.

NOTE 1. The effective cohesion and the angle of effective friction are mutual dependent parameters and cannot be determined independently.

(2) <REQ> In drained strength envelopes describing critical state conditions, effective cohesion shall be assumed to be zero ($c' = 0$).

(3) <PER> As an alternative to (1), the shear stress at failure τ_f may be determined from a Mohr-Coulomb envelope given in terms of total stresses by Formula (8.2):

$$\tau_{\mathrm{f}} = \frac{(\sigma_1 - \sigma_3)_f}{2} \tag{8.2}$$

where:

σ_1 and σ_3 are the major and minor total principal stresses, respectively.

(4) <PER> Undrained strength may be determined from different types of tests considering:

- the deformation modes applied in the tests;
- the rates of shearing in the tests;
- potential anisotropic ground behaviour.

8.1.3 Strength envelopes for unsaturated soils

(1) <PER> The shear stress at failure τ_f of unsaturated soils may be determined from the Mohr-Coulomb envelope given in terms of effective stresses by Formula (8.3):

$$\tau_{\mathrm{f}} = c' + (\sigma - u)\tan\varphi' + (u_{\mathrm{a}} - u)f_1 \tag{8.3}$$

where:

c' is the effective cohesion;

σ' is the normal total stress on the failure plane;

u_a is the pore air pressure;

φ' is the angle of effective friction;

u is the pore water pressure;

f_1 is a function defining the relationship between shear strength and soil suction.

NOTE 1. Methods to evaluate the parameters in Formula (8.3) are given in Fredlund (2006).

(2) <PER> Total strength envelopes may be defined for unsaturated conditions with friction angle and cohesion depending directly or indirectly on the degree of saturation.

8.1.4 Strength envelopes for rock material and rock mass

(1) <PER> For both rock material and rock mass, shear strength may be described using the Hoek-Brown strength envelope given by Formula (8.4):

$$\sigma_1 = \sigma_3 + \sigma_{\mathrm{ci}}\left(m_{\mathrm{b}}\frac{\sigma_3}{\sigma_{\mathrm{ci}}} + s\right)^a \tag{8.4}$$

where:

σ_1 and σ_3 are the major and minor principal stresses, respectively;

σ_{ci} is the uniaxial compressive strength of the intact rock;

m_b, s, and a are non-dimensional material parameters.

NOTE 1. For intact rock, the values in Formula (8.4) are a = 0.5, and s = 1.

NOTE 2. For intact rock, m_i replaces m_b = m_i in Formula (8.4).

NOTE 3. The value of m_b is determined as given in 8.3.3.

NOTE 4. See Hoek, E. & Brown, E.T. (2019) for further information.

(2) <PER> For both rock material and rock mass, shear strength may be described also using Mohr-Coulomb envelopes.

(3) <REQ> When the Hoek-Brown strength envelope is approximated by a linear relationship, the procedure used, and its applicable stress range shall be recorded in the Ground Investigation Report.

8.1.5 Strength envelopes for rock discontinuities

8.1.5.1 Closed rock discontinuities

(1) <RCM> The shear stress at failure τ_f along closed rock discontinuities should be determined using non-linear strength envelopes.

(2) <PER> The value of τ_f along closed rock discontinuities may be described by the Barton-Bandis strength envelope given by Formula (8.5):

$$\tau_{\mathrm{f}} = \sigma_{\mathrm{n}} \tan\left(JRC \log_{10}\frac{JCS}{\sigma_{\mathrm{n}}} + \varphi_{\mathrm{r}}\right) \tag{8.5}$$

where

τ_f is the shear stress at failure along a discontinuity;

σ_n Is the normal stress acting on the discontinuity;

JRC is the joint roughness coefficient;

JCS is the joint wall compressive strength;

φ_r is the residual friction angle of the discontinuity.

NOTE 1. Methods to evaluate the parameters in Formula (8.5) are given in ISRM (2004).

(3) <REQ> The Barton-Bandis strength envelope shall only be used for closed discontinuities.

(4) <PER> The value of τ_f along closed rock discontinuities may be also described by Mohr-Coulomb envelopes.

(5) <REQ> When the Barton-Bandis strength envelope is approximated by a linear relationship, the procedure used and its applicable stress range shall be recorded in the Ground Investigation Report.

8.1.5.2 Open and infilled rock discontinuities

(1) <RCM> Aperture and filling should be considered when assessing the shear strength of discontinuities.

(2) <PER> The value of the shear stress at failure τ_f along open or infilled rock discontinuities may be described by the Mohr-Coulomb strength envelope using Formula (8.1).

NOTE 1. For open and infilled discontinuities, cohesion is normally taken as $c' = 0$.

NOTE 2. For infilled discontinuities, the value of φ' is normally taken as the angle of friction of the infill or as the angle of interface friction between the infill and the rock surface.

8.1.6 Other strength envelopes

(1) <PER> As an alternative to 8.1.2, 8.1.4, and 8.1.5, other strength envelopes may be used.

NOTE 1. More elaborate descriptions of the effect of intermediate principal stress on shear strength than those provided by Mohr-Coulomb and Hoek-Brown models are sometimes necessary.

(2) <REQ> Strength envelopes shall be considered as calculation models and validated according to 1997-1, 7.1.1.

(3) <REQ> If models including implicitly defined strength envelopes are used, it shall be shown that these models can reproduce the strength of the material under simple stress conditions relevant for design.

NOTE 1. Simple stress conditions that are typically relevant include triaxial compression, triaxial extension and simple shear.

8.2 Soil strength

8.2.1 Direct determination of soil strength

(1) <RCM> The shear strength of soils should be determined directly using one or more of the tests given in Table 8.1.

Table 8.1 – Direct determination of soil strength properties

Property	**Test**	**Standard**	**MQC**	**Comments on suitability and interpretation**
Peak effective cohesion and friction (c'_{p}, φ'_{p})	Consolidated triaxial compression	EN ISO 17892-9	1	See 8.2.1 (4) to (10)
	Direct shear	EN ISO 17892-10	1	
	Direct simple shear	See Table B.5	1	
Angle of friction critical state (φ'_{cs})	Consolidated triaxial compression	EN ISO 17892-9	1-4	See 8.2.1 (5) to (9)
	Direct shear	EN ISO 17892-10	1-4	See 8.2.1 (4) to (9)
	Direct simple shear	See Table B.5	1-4	
Residual effective cohesion and friction (c'_{r}, φ'_{r})	Direct shear	EN ISO 17892-10	1	
	Ring shear	EN ISO 17892-10	1	
Peak undrained cohesion ($c_{u,p}$)	Unconfined compression test	EN ISO 17892-7	1	See 8.2.1 (4), (8), and (11) In the triaxial tests, compression is undrained
	Unconsolidated undrained triaxial compression	EN ISO 17892-8	1	
	Consolidated triaxial compression	EN ISO 17892-9	1	
	Laboratory vane	See Table B.5	1	
	Field vane	EN ISO 22476-9	-	See 8.2.1 (11)
	Direct simple shear	See Table B.5	1	
Remoulded undrained cohesion $c_{u,rmd}$	Laboratory vane	See Table B.5	1	
	Field vane	EN ISO 22476-9	-	

(2) <PER> The shear strength of soils may be determined directly using tests not given in Table 8.1 provided that the test procedure and reporting requirements have been specified by the relevant authority or agreed for a specific project by the relevant parties.

(3) <REQ> For laboratory tests not listed in Table 8.1, the following shall be specified and reported:

- specimen preparation method;
- orientation of specimen;
- type of test;
- classification tests that need to be done;
- consolidation stresses;
- time for consolidation increments;
- criteria applied to end consolidation;
- shearing rate;
- strain at which parameters are determined
- criteria for terminating tests;
- acceptability criteria;
- accuracy of measurements.

NOTE 1. Examples of acceptability criteria is degree of saturation during test and scatter in results.

(4) <REQ> The peak shear strength of clays, silts, and organic soils shall be determined using specimens prepared from samples in Quality Class 1.

(5) <PER> The critical state and residual shear strengths of clays, silts, and organic soils may be determined using reconstituted specimens from samples in Quality Class 4 or above.

(6) <RCM> The shear strength of coarse soils should be determined from reconstituted specimens from samples in Quality Class 4 or above.

(7) <REQ> If reconstituted specimens are employed to determine shear strength, the method employed for specimen formation as well as the composition and state (stress, density, saturation) of the specimen shall be specified before testing and reported with the test results.

NOTE 1. It is generally desirable to reconstitute specimens at state conditions close to those in-situ.

NOTE 2. Methods of preparing reconstituted specimens are given in EN ISO 17892-9.

(8) <RCM> Differences in saturation between specimens at testing and conditions in-situ at appropriate design situations should be taken into account when deriving strength parameters.

NOTE 1. Effective cohesion can arise from fitting a linear envelope to a non-linear response that is also relevant in the field, but it also may arise from specimen conditions not relevant in design, e.g. partial saturation.

(9) <RCM> Non-uniform failure modes in triaxial specimens should be avoided when determining critical state shear strength.

NOTE 1. Special procedures not covered by EN ISO 17892-9 (e.g. lubricated end platens) are sometimes needed to avoid non-uniformities in triaxial specimens.

(10) <RCM> Dilatancy effects on shear strength should be considered when determining strength parameters.

(11) <RCM> Undrained shear strength values derived from any field or laboratory vane test should not be employed in design if partial drainage is suspected during testing.

8.2.2 Indirect determination of soil strength

(1) <PER> The shear strength of soils may be determined indirectly using any of the tests listed in Table 8.2.

Table 8.2 – Indirect determination of soil strength properties

Property	**Test**	**Standard**	**MQC**	**Comments on suitability and interpretation**
Angle of peak effective friction (φ'_p)	Cone Penetration Test	EN ISO 22476-1	-	See Annex E for correlations (with I_D) for coarse soils correlations
	Standard Penetration Test	EN ISO 22476-3	-	
	Menard Pressuremeter Test	EN ISO 22476-4	-	-
	Flexible Dilatometer Test	EN ISO 22476-5	-	-
	Flat Dilatometer Test	EN ISO 22476-11	-	-
Angle of friction critical state (φ'_{cs})	Consistency limits	EN ISO 17892-12	4	-
Residual shear strength (φ'_r)	Consistency limits	EN ISO 17892-12	4	-
Peak undrained cohesion ($c_{u,p}$)	Cone Penetration Test	EN ISO 22476-1	-	See Annex E for correlations for fine soils
	Standard Penetration Test	EN ISO 22476-3	-	
	Menard Pressuremeter Test	EN ISO 22476-4	-	
	Flexible Dilatometer Test	EN ISO 22476-5	-	
	Flat Dilatometer Test	EN ISO 22476-11	-	-
	Fall cone	EN ISO 17892-6	1	
Remoulded undrained cohesion $c_{u,rmd}$	Cone Penetration Test	EN ISO 22476-1	-	Correlations (with sensitivity) exist for fine soils

(2) <PER> The shear strength of soils may be determined indirectly using tests not given in Table 8.2 provided that the test procedure and reporting requirements have been specified by the relevant authority or agreed for a specific project by the relevant parties.

(3) <REQ> For tests not listed in Table 8.2, the following shall be specified and reported:

- the testing procedure (reference shall be given to relevant standard, if available);
- the test equipment (reference shall be given to relevant standard, if available);
- the test results to be employed in interpretation;
- an estimate of measurement error.

(4) <REQ> All correlations used for indirect determination of shear strength shall comply with 4.2.

(5) <RCM> Peak undrained cohesion may be derived from DMT results using the relationship between DMT results and overconsolidation ratio.

NOTE 1. See Annex E.3 for an example relationship.

8.3 Rock strength

8.3.1 Rock material strength

(1) <RCM> Strength parameters for rock material (intact rock) should be determined using one or more of the laboratory tests given in Table 8.3.

Table 8.3 – Determination of rock material strength properties

Property	**Test**	**Standard**	**MQC**	**Comments on suitability and interpretation**
Compressive strength (σ_{ci})	Unconfined compression test (UCT)	See Table B.5	-	See (2) and (3) See ISRM (2007a)
	Triaxial test (TX)	See Table B.5	-	See (2) and (3) See ISRM (2007b)
	Point load test	See Table B.5	-	See (2) and (3)
	Schmidt hammer test	See Table B.5	-	See (2) and (3)
Parameter (m_i)	Triaxial test (TX)	See Table B.5	-	See (2) and (3) See ISRM (2007b)
Tensile strength (σ_t)	Direct tensile tests	See Table B.5	-	See (2) and (3) See ISRM (2007b)
	Point load test	See Table B.5	-	See (2) and (3)
Flexural strength (σ_{fl})	3 and 4-point bend tests for flexural strength	See Table B.5	-	See (2) and (3)

(2) <RCM> The effect of specimen size should be taken into account.

NOTE 1. Small specimens normally give less representative results than large ones.

8.3.2 Strength of rock discontinuities

(1) <REQ> 6.2 and EN 1997-1, 4.3.3, shall apply.

(2) <REQ> The properties affecting the strength of the discontinuities shall be recorded in the Ground Investigation Report.

(3) <RCM> The shear strength of rock discontinuities should be determined using one or more of the laboratory tests given in Table 8.4.

Table 8.4 – Determination of strength properties for rock discontinuities

Property	Test	Standard	MQC	Comments on suitability and interpretation
Peak shear strength along discontinuity (c'_p, φ'_p)	Direct shear of rock discontinuities	See Table B.5	-	See (6)-(8) Only derived for a certain range at a particular value of σ_3 or σ_n
	Triaxial test (TX)	See Table B.5	-	See (6)-(8)
Basic shear strength along discontinuity (φ_b)	Basic friction angle of rock discontinuities	See Table B.5	-	See (6)-(8). The tilt test defines under which dip angle the upper block on a discontinuity starts sliding
Residual shear strength of discontinuity (φ_r)	Residual friction angle of rock discontinuities	See Table B.5	-	See (6)-(8)

(4) <RCM> Scale effects should be accounted for when extrapolating the results of laboratory discontinuity strength measurements to larger scales.

(5) <PER> Inputs for the strength evaluation of rock discontinuities may be obtained from descriptive procedures given in EN ISO 14689.

8.3.3 Rock mass strength

(1) <PER> The strength of the rock mass may be determined using:

- rock mass classifications;
- extrapolation from laboratory results in combination with rock mass characteristics;
- back-analysis;
- a combination of the above.

(2) <REQ> The strength of rock mass shall be determined accounting for the influence of:

- rock material strength;
- rock mass structure;
- discontinuity geometry;
- surface conditions of discontinuities;
- groundwater presence;
- depth of stress level;
- the scale of project in relation to the scale of rock properties, and
- type and applicability of investigation methods.

(3) <PER> Crack Initiation (CI) and Crack Damage (CD) stress levels may be determined from rock mass classification results.

(4) <PER> The strength of a rock mass may be described by one or more of the following:

- Hoek-Brown strength parameters;
- Mohr-Coulomb strength parameters;
- the Geological Strength Index (GSI).

(5) <RCM> Rock mass strength values should be reduced in the presence of weak discontinuities unfavourably oriented with respect to the excavation face.

(6) <RCM> Rock mass strength values should be reduced in the case of water and frost-induced weathering of discontinuity surfaces.

(7) <PER> The values of the Hoek-Brown strength parameters for a rock mass (m_b, a, and s) may be determined using empirical formulations that have as inputs:

- the values of Hoek-Brown strength parameters for the rock material (intact rock); and
- the values obtained from GSI or from other rock mass classification systems.

(8) <PER> In addition to 8.1.4, the values of the Hoek-Brown strength parameters for a rock mass may be determined using Formulae (8.6) to (8.8):

$$m_{\mathrm{b}} = m_{\mathrm{i}} e^{\left(\frac{GSI-100}{28-14D}\right)} \tag{8.6}$$

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \tag{8.7}$$

$$a = \frac{1}{2} + \frac{1}{6} e^{\left(\frac{-GSI}{15}\right)} - e^{\left(\frac{-100}{15}\right)} \tag{8.8}$$

m_b, s, and a	are as defined for Formula (8.4);
m_i	is a non-dimensional material parameter for the intact rock;
GSI	is the Geological Strength Index;
D	is the disturbance factor of the rock mass (0 < D < 1).

(9) <RCM> The value of the GSI should be derived from the lithology, rock structure, interlocking, discontinuities, joint sets, and joint surface conditions.

NOTE 1. See Appendix E.5 for an example procedure for determining GSI.

(10) <RCM> An average value of GSI should be determined for each distinct rock mass unit to consider the spatial variability of the parameter.

(11) <RCM> Hoek - Brown envelope for rock mass strength should not be used in cases when failure is controlled by discontinuities or other geological features.

(12) <RCM> Values of GSI should be applied carefully with respect to the methods limitations.

8.4 Interface strengths

(1) <PER> Interface strengths between the ground and other materials (steel, concrete and plastics) may be determined by suitably adapted laboratory direct or ring shear tests or by field pull-out or direct shear tests.

(2) <REQ> When measuring interface strengths, the roughness of the material surface shall be recorded in the Ground Investigation Report.

9 Stiffness, compressibility and consolidation

9.1 Ground stiffness

9.1.1 General

(1) <RCM> Ground stiffness should be described by a stress-strain curve over the expected stress and strain ranges for the anticipated design situation.

(2) <PER> Ground stiffness may be approximated by one or more values of elastic moduli, provided each modulus is limited to a particular stress or strain range (see Annex F).

(3) <RCM> Ground stiffness properties should be determined directly (from test results), according to 9.1.2.

(4) <RCM> Tests carried out to measure ground stiffness should follow the anticipated stress path for the relevant design situation.

NOTE 1. Ground zones around a geotechnical structure can follow different stress paths.

(5) <REQ> The following factors shall be considered when selecting the test method and the procedure to determine stiffness:

– scale of specimens according to ground mass;
– discontinuity pattern in the ground, especially in rock masses and stiff clays;
– strain level and strain rate compared to the ones expected in the ground;
– in-situ stress state;
– stress history;
– foliation;
– anisotropy of the ground.

(6) <REQ> The loading rate and drainage conditions shall be chosen accordingly when determining undrained or drained moduli.

(7) <RCM> Time effects should be determined for swelling (9.2.4), creep (9.2.2), and crushing behaviour of rock mass

(8) <REQ> The effect of cyclic and dynamic actions on ground stiffness shall be taken into account according to 10.

(9) <PER> Techniques based on propagation of shear waves or other dynamic methods may be used to determine the very small strain modulus of ground.

(10)<RCM > The stiffness decay curve should be assembled using the results from a range of tests including seismic, laboratory, and field investigation tests.

NOTE 1. See Annex F.

(11)<REQ> The minimum resolution of the test procedure shall be recorded in the Ground Investigation Report.

9.1.2 Direct determination of ground stiffness

9.1.2.1 By field investigation

(1) <RCM> Field measurements of stiffness should be obtained using one or more of the tests given in Table 9.1.

Table 9.1 – Direct determination of ground stiffness properties from field investigation

Strain level	Property	Test	Standard	Comments on suitability and interpretation
Very small ($< 10^{-5}$)	G_0	Seismic DMT/CPT Geophysical seismic tests	ISRM Methods*	One-
	G_{FDT}, E_{FDT}	Flexible Dilatometer Test	EN ISO 22476-5	Several-
	E_{BJT}	Borehole Jacking Test	EN ISO 22476-7	Several
	E_{rm}	Rigid Plate Loading	See Table B.6	-
		Flexible plate loading method	See Table B.6	-
		Radial jacking test	See Table B.6	-
		Large flat jack tests	See Table B.6	-
Small (10^{-5}-10^{-2})	G_{SBP}, E_{SBP}	Self-boring pressuremeter test	EN ISO 22476-6	Full curve-
	E_{PLT}	Plate Loading Test	EN ISO 22476-13	Several-
	E_{DMT}	Flat Dilatometer Test	EN ISO 22476-11	One-
Medium (10^{-2}-10^{-1})	G_M, E_M	Ménard Pressuremeter Test	EN ISO 22476-4	Several/full curve-
	G_{FDT}, E_{FDT}	Flexible Dilatometer Test	EN ISO 22476-5	Full curve-
	G_{SBP}, E_{SBP}	Self-boring Pressuremeter Test	EN ISO 22476-6	Full curve-
	E_{FDP}	Full Displacement Dilatometer Test	EN ISO 22476-8	Full curve-
	-	Drill hole deformation gauges	See Table B.6	Full curve-
Large ($> 10^{-1}$)	G_M, E_M	Ménard Pressuremeter Test	EN ISO 22476-4	Several/full curve-
	E_{PLT}	Plate Loading Test	EN ISO 22476-13	One value-

(2) <PER> Field measurements of stiffness may be obtained using tests not listed in in Table 9.1 provided that the test procedure and reporting requirements have been specified by the relevant authority or agreed for a specific project by the relevant parties.

NOTE 1. The geophysical seismic tests from which very small strain modulus can be derived are given in 10.4.

NOTE 2. Other geophysical methods or specific logging technics can provide information on stiffness parameters: micro-seismic in borehole; full wave sounding in borehole and seismic refraction.

(3) <PER> In rock mass, optical or acoustic images of the borehole walls may be requested for the choice of the interval to be tested (especially when cores are not available).

(4) <RCM> Rock creep should be determined by using Unconfined Compression Tests (UCTs) or triaxial tests.

9.1.2.2 By laboratory testing

(1) <RCM> Laboratory measurements of stiffness should be obtained using one or more of the tests given in Table 9.2.

Table 9.2 – Direct determination of ground stiffness properties from laboratory tests

Strain level	Property	Test	Standard	Comments on suitability and interpretation
Very small small (< 10^{-5})	G_0	Bender elements	-	One
	G_0	Resonant column tests	See Table B.6	Several/full curve
	K_p	P-wave pulsar elements	EN 14579 EN 14146	One
Small (10^{-5}-10^{-2})	G, G_{cyc}	Consolidated Undrained Direct Simple Shear Testing	See Table B.6	Several/full curve
	G, E	Consolidated triaxial compression tests on water saturated soils (with measurement of local strains)	EN ISO 17892-9	(Partially applicable) Several/full curve
		Triaxial tests for rock specimens (with global or local strain measurement)	See Table B.6	-
		Direct shear test for discontinuities (for normal and tangential stiffness)	See Table B.6	-
	E	Unconfined compression test (UCT)	EN ISO 17892-7 EN 14580 See Table B.6	(Partially applicable) Several
	G_{cyc}, E_{cyc}	Determination of the Modulus and Damping Properties of Soils Using the Cyclic Triaxial Apparatus (CTxT)	See Table B.6	Several/full curve
	$G_{0,RC}$	Modulus and Damping of Soils by Fixed-Base Resonant Column Devices (RC)	See Table B.6	Several/full curve
Medium (10^{-2}-10^{-1})	E_{OED}	Incremental loading oedometer test	EN ISO 17892-5	Several
	E_{OED}	Constant Rate of strain test (CRS)	See Table B.6	Full curve
	G, G_{sec}	Consolidated Undrained Direct Simple Shear Testing (DSS)	See Table B.6	Several/full curve
	G, E	Consolidated triaxial compression tests on water saturated soils (with measurement of local strains)	EN ISO 17892-9	Several/full curve
		Triaxial tests for rock specimens (with global or local strain measurement)	See Table B.6	-
		Direct shear test for discontinuities (for normal and tangential stiffness)	See Table B.6	-
	E	Unconfined compression test (UCT)	EN ISO 17892-7 EN 14580 See Table B.6	Several
	K_s, K_n	Discontinuity shear test	See Table B.6	-
Large (> 10^{-1})	G, G_{sec}	Consolidated Undrained Direct Simple Shear Testing (DSS)	See Table B.6	Several/full curve
	G, E	Consolidated triaxial compression tests on water saturated soils(TxT with measurement of local strains)	EN ISO 17892-9	Several/full curve
		Triaxial tests for rock specimens (with global or local strain measurement)	See Table B.6	-
		Direct shear test for discontinuities (for normal and tangential stiffness)	See Table B.6	-
	G, E	Unconfined Compression Test (for rocks)	See Table B.6	-

(2) <REQ> The choice of laboratory test shall be consistent with the strain level expected in the anticipated design situations.

(3) <PER> Laboratory measurements of stiffness may be obtained using tests not listed in Table 9.2 provided that the test procedure and reporting requirements have been specified by the relevant authority or agreed for a specific project by the relevant parties.

(4) <PER> Bulk modulus may be determined by following appropriate stress paths in triaxial tests.

(5) <PER> The Poisson's ratio (υ) of the ground may be determined in uniaxial or triaxial compression tests in the elastic range.

(6) <PER> In the absence of direct measurement, the Poisson's ratio of soil may be assumed to be υ = 0,3 in drained conditions and υ_u = 0,5 in undrained conditions.

(7) <REQ> Soil specimens used for laboratory measurement of stiffness shall be obtained from Sample Quality Class 1.

NOTE 1. Annex F gives indicators of specimen quality that can be used to ensure a minimum quality class.

NOTE 2. 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. Specific sampling equipment and methods can 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.

9.1.3 Indirect determination of ground stiffness

(1) <PER> Stiffness parameters of ground may be determined indirectly from one or more of the tests or procedures given in Table 9.3.

Table 9.3 – Indirect determination of ground stiffness

Property	**Test or procedure**	**Standard**	**Comments on suitability and interpretation**
Shear modulus (G)	Cone Penetration Test	EN ISO 22476-1	Correlations are strain level dependent
	Standard Penetration Test	EN ISO 22476-3	For coarse grained soils only
	Back analysis	EN 1997-1 4.3.2 and 4.8 EN ISO 18674 (all parts)	-
Drained Young's modulus (E') and undrained Young's modulus (E_u)	Cone Penetration Test	EN ISO 22476-1	Correlations are strain level dependent
	Dynamic Penetration Test	EN ISO 22476-2	-
	Ménard pressuremeter test and Flexible dilatometer test PMT	EN ISO 22476-4 and -5	-
	Back analysis	EN 1997-1 4.3.2 and 4.8 EN ISO 18674 (all parts)	-
Oedometer modulus (E_{OED})	Ménard pressuremeter test and Flexible dilatometer test PMT	EN ISO 22476-4 and -5	
	Cone Penetration Test	EN ISO 22476-1	Correlations are strain level dependent
	Standard Penetration Test	EN ISO 22476-3	For coarse grained soils only
	Back analysis	EN 1997-1 4.3.2 and 4.8 EN ISO 18674 (all parts)	-

(2) <PER> Indirect measurements of stiffness may be obtained using tests or procedures not listed in Table 9.3 provided that the test procedure and reporting requirements are as specified by the relevant authority or agreed for a specific project by the relevant parties.

(3) <PER> Back analysis may be used to determine ground stiffness according to 5.3.4.

9.2 Ground compressibility and consolidation

9.2.1 General

(1) <RCM> Ground compressibility should be described by a load-compression curve over the expected stress and strain ranges, including loading-unloading conditions, for the anticipated design situation.

(2) <RCM> Ground compressibility should be approximated by one or more compression parameters, each parameter limited to a particular stress or strain range and time period.

NOTE 1. Relevant parameters include the compression index (C_c), recompression index (C_r), coefficient of secondary compression (C_α) and pre-consolidation pressure (σ'_p).

(3) <RCM> Compressibility parameters and the coefficient of consolidation (c_v) should be determined directly according to 9.2.2.

(4) <RCM> When swelling or viscous (time dependent) behaviour is encountered in the ground, the testing program should be adapted with longer test duration (see 9.2.4).

(5) <PER> Compressibility parameters for soils in an unsaturated state may be determined to evaluate the additional compression upon inundation due to structural collapse of the soil as specified by the relevant authority or agreed for a specific project by the relevant parties.

9.2.2 Direct determination of compression and consolidation properties

(1) <RCM> Laboratory measurements of ground compressibility and consolidation parameters should be obtained using one or more of the tests given in Table 9.4.

NOTE 1. Additional loading procedure stages to those reported in the standards listed in Table 9.4 can be used.

Table 9.4 – Direct determination of compression and consolidation properties

Property	Test	Standard	MQC	Comments on suitability and interpretation
Compression index (C_c)	Incremental loading oedometer	EN ISO 17892-5	1	1-dimensional value
	Constant rate of strain	See Table B.7	1	1-dimensional value
	Consolidated triaxial compression on water saturated soils	EN ISO 17892-9	1	Isotropic value
Recompression index (C_r)	Incremental loading oedometer	EN ISO 17892-5	1	For any loading cycle
	Constant rate of strain	See Table B.7	1	Single value
One-dimensional compressibility (m_v)	See Table 9.2 ($m_v = 1/E_{OED}$)			
Pre-consolidation pressure (σ'_p)	Incremental loading oedometer	EN ISO 17892-5	1	1-dimensional value
	Constant rate of strain	See Table B.7	1	1-dimensional value

Property	Test	Standard	MQC	Comments on suitability and interpretation
	Consolidated triaxial compression on water saturated soils	EN ISO 17892-9	1	Isotropic value
Coefficient of vertical consolidation (c_v)	Incremental loading oedometer	EN ISO 17892-5	1	At any loading or unloading step
	Constant rate of strain (CRS)	See Table B.7	1	At any set of readings
Coefficient of horizontal consolidation (c_h)	Cone Penetration Test (CPTU)	EN ISO 22476-1	-	-
	Flexible Dilatometer Test	EN ISO 22476-5	-	-
	Self-boring Pressuremeter Test	EN ISO 22476-6	-	-
	Flat Dilatometer Test	EN ISO 22476-11	-	-
Coefficient of secondary compression (C_α)	Incremental loading oedometer	EN ISO 17892-5	1	Several values

9.2.3 Indirect determination of compression and consolidation properties

(1) <PER> Compression and consolidation properties for soils may be determined indirectly from one or more of the tests given in Table 9.5.

Table 9.5 – Indirect determination of compression and consolidation properties

Property	Test	Standard
Compression index (C_c)	Liquid limit	EN ISO 17892-12
Recompression index (C_r)	Compression index	EN ISO 17892-5
Coefficient of consolidation (c_v)	Liquid limit	EN ISO 17892-12
Coefficient of secondary compression (C_α)	Compression index	EN ISO 17892-5

(2) <PER>Indirect measurements of ground compression or recompression parameters may be obtained using tests not listed in Table 9.5 provided that the test procedure and reporting requirements have been specified by the relevant authority or agreed for a specific project by the relevant parties.

(3) <REQ> The formulas used to obtain the ground compression or consolidation parameters shall be documented in the Ground Investigation Report, together with all parameters used.

9.2.4 Swelling properties

(1) <RCM> Swelling parameters of ground should be determined directly from one or more of the tests given in Table 9.6.

Table 9.6 — Direct determination of swelling properties from laboratory tests

Property	Test	Test standard	MQC	Comments on suitability and interpretation
Swelling pressure (σ_g)	One specimen with axial surcharge Huder Amberg method	ISRM Methods*	n/a	σ'_{vo} deduced from the Ground Model should be provided
	under zero volume change	See Table B.7	1	Specific to stress path
	Several specimens with axial surcharge	See Table B.7	-	-
Swelling amplitude (ε_g)	Free swelling	See Table B.7	1	Specific to stress path
	Linear swelling	EN 13286-47	-	Unbound and hydraulically bound mixtures
Swelling coefficient (C_g)	Several specimens with axial surcharge; one-Dimensional Swell or Settlement Potential	See Table B.7	1	Pressures should be specified
	Huder Amberg method	See Table B.7		
Swelling index (C_{sw})	Incremental loading oedometer test (unloading)	EN ISO 17892-5	1	Several conventional values
	Constant rate of strain test	See Table B.7		-

(2) <PER> Laboratory measurements of swelling may be obtained using tests not listed in Table 9.6 provided that the test procedure and reporting requirements have been specified by the relevant authority or agreed for a specific project by the relevant parties.

(3) <RCM> Undisturbed specimens of Quality Class 1 should be tested where possible, since material fabric has an important effect on swelling characteristics.

(4) <PER> Where the sample is too weak or too broken to allow preparation, such as joint fill material, swelling index tests may be carried out on remoulded and re-compacted specimens provided the procedures used are documented in the Ground Investigation Report.

10 Cyclic, dynamic, and seismic properties

10.1 General

(1) <REQ> Ground investigations of the mechanical response to cyclic and dynamic action shall provide the relevant information for:

– Design for cyclic and dynamic actions; and
– seismic design (see ENs 1998-1 and 1998-5).

(2) <RCM> Ground investigations should provide relevant information on:

– stress-strain response to cyclic actions, including small strain stiffness;
– energy dissipation properties;
– development of excess pore water pressures under cyclic actions;
– shear strength under cyclic actions;
– post cyclic behaviour in terms of post-cyclic shear strength, dissipation of cyclic-induced pore water pressure, and other associated deformations;
– cyclic undrained shear strength for liquefaction assessment (see EN 1998-5).

(3) <REQ> The following factors, which affect the measurement of mechanical response of the ground to cyclic and dynamic actions, shall be considered when selecting a test method:

– intrinsic and state properties;
– scale of specimens according to ground mass;
– discontinuity pattern in the ground specially in rock masses;
– strain level compared to the one expected in the ground for the specific design situation;
– expected cyclic and average stresses for the specific design situation;
– anisotropy;
– foliation; and
– stress history.

(4) <PER> The pre-failure stress-strain response to cyclic actions may be described in terms of variation of the secant elastic modulus and damping ratio versus cyclic shear strain.

10.2 Measurement of cyclic response

(1) <RCM> The response to cyclic and dynamic actions should be investigated in the laboratory using one or more of the laboratory tests given in Table 10.1.

Table 10.1 – Laboratory tests for measuring response to cyclic and dynamic actions

	Laboratory test (and associated test standards)					
Test	**Cyclic torsional shear**	**Cyclic direct simple shear**	**Cyclic triaxial**	**Resonant column**	**Bender elements**	**Cyclic triaxial for rock**
Standard	See Table B.8					
Strain level						
Very small (< 10^{-5})	(full)	-	-	full	one	(full)
Small (10^{-5} -10^{-2})	full	full	(full)	(full)	-	full
Medium (10^{-2} -10^{-1})	-	(full)	full	-	-	-
- = not applicable; 'one' = one conventional value; 'full' = full curve; () = partially applicable						

(2) <REQ> In laboratory tests, the range of cyclic strains investigated shall be consistent with the expected level of strains for the specific design situation.

(3) <REQ> Cyclic shearing applied in the tests shall be initiated from the effective stress state relevant for the design situation.

(4) <RCM> The response to cyclic actions should be investigated on specimens obtained from samples of Quality Class 1.

(5) <PER> When undisturbed sampling is not feasible, the tests may be performed on reconstituted samples that reproduce the state properties of the in-situ ground

(6) <REQ> The method for reconstituted specimen formation shall be specified before testing and reported with the test results.

NOTE 1. Methods of preparing reconstituted specimens are given in EN ISO 17892-9.

(7) (<RCM> The reconstitution process of the sample should mimic as far as possible the stress history of the soil before application of the cyclic test stresses.

(8) <REQ> Specimens of soil to be used for fill shall be reconstructed from bulk or disturbed samples by simulating the expected compaction process to be employed on site.

10.3 Secant modulus and damping ratio curves

10.3.1 General

(1) <RCM> The variation of secant shear modulus and of damping ratio against cyclic shear strain should be investigated with laboratory tests.

NOTE 1. The variation of shear modulus versus cyclic shear strain is usually normalized by the value of shear modulus at very small strains (γ < 10-5) as measured during tests.

(2) <RCM> The variation of shear modulus versus cyclic shear strain should be normalized by the value of shear modulus at very small strains ($\gamma < 10^{-5}$) as measured during the test.

(3) <PER> When laboratory tests are not feasible, the normalised secant shear modulus curve and the damping ratio curve may be determined indirectly using empirical relationships that take into account physical parameters and soil classification indices.

NOTE 1. Examples of indirect methods are reported in Annex G.

(4) <RCM> The indirect methods should take explicit account of the influence of:

- grain size distribution;
- plasticity index;
- in-situ state of stress;
- density index for coarse soils or void ratio for fine soils;
- over-consolidation ratio; and
- number of equivalent cycles.

10.3.2 Measured values

(1) <REQ> The measurements shall cover the relevant stress or strain regime being identified for the design situation.

(2) <RCM> The degradation of soil response over repeated cycles should be quantified in cyclic tests by assessing the degradation of the normalised shear modulus and the increase of damping ratio as a function of the number of applied cycles.

10.4 Very small strain moduli and wave velocities

10.4.1 General

(1) <PER> The very small strain shear modulus G_0 may be estimated using field geophysical measurements of the velocity of propagation of shear waves (see EN 1998-5, 6.1(8)), using Formula (10.1):

$$G_0 = \rho v_S^2 \tag{10.1}$$

where

ρ is the bulk mass density; and

v_S is the shear wave velocity.

(2) <PER> Wave propagation velocities determined on laboratory specimens may be used to assess disturbance of the material with respect to its in-situ state.

(3) <PER> Measurement of vertically and horizontally polarised shear waves may be used to investigate the anisotropy of ground response.

10.4.2 Direct determination of wave velocities

(1) <RCM> Shear and compressional wave velocities should be determined directly using any of the geophysical tests given in Table 10.2.

Table 10.2 – Geophysical tests to determine shear and compressional wave velocities

Parameter	**Test**	**Standard**
Shear wave velocity (v_S)	Cross-Hole Test	See Table B.9
	Down-Hole Test	See Table B.9
	P-S suspension logging test	-
	Seismic Refraction	See Table B.9
	Seismic Cone Penetration Test	-
	Seismic Flat Dilatometer Test	-
	Surface Wave Methods	-
Compressional wave velocity (v_P)	Cross-Hole Test	See Table B.9
	Down-Hole Test	See Table B.9
	P-S suspension logging test	-
	Seismic Refraction	See Table B.9
	Seismic Reflection	See Table B.9
Note: Surface wave methods include all the geophysical methods which are based on the spectral analysis of the propagation of surface waves (Rayleigh, Love, or Stoneley) such as Spectral Analysis of Surface Waves (SASW), Multichannel Analysis of Surface Waves (MASW), Continuous Source Surface Waves (CSSW), and Ambient Vibration Analysis (AVA).		

(2) <PER> Other geophysical methods may be used provided that they guarantee an adequate spatial resolution and accuracy with respect to the design situation and that the test procedure and reporting requirements have been specified by the relevant authority or agreed for a specific project by the relevant parties.

(3) <RCM> Cross-hole tests should be used whenever a very high resolution and accuracy is necessary for the specific design situation.

(4) <REQ> The interpretation of cross-hole tests shall account for critical refraction at the interface between different layers especially when a sequence of thin layers with a marked change of velocity is expected.

(5) <REQ> P-S suspension logging shall only be used to obtain measurement more than 5 m deep.

(6) <RCM> When using surface wave methods or seismic refraction surveys, uncertainties associated with solution non-uniqueness should be quantified.

(7) <REQ> The seismic refraction survey shall only be used whenever the stratigraphic conditions are such that a reduction of velocity with depth is excluded.

(8) <RCM> Surface wave methods and seismic refraction surveys should not be used whenever the identification of thin layers (few metres) at large depth (more than 20m) is relevant for the design situation.

10.4.3 Indirect determination of shear wave velocity

(1) <PER> The shear wave velocity may be determined indirectly using correlations with any of the tests listed in Table 10.3.

NOTE 1. Example correlations are given in Annex G.

Table 10.3 – Tests for indirect determination of shear wave velocity

Parameter	Test	Test Standard
Shear wave velocity (v_s)	Cone Penetration Test	EN ISO 22476-1
	Standard Penetration Test	EN ISO 22476-3
	Ménard Pressuremeter Test	EN ISO 22476-4
	Flexible Dilatometer Test	EN ISO 22476-5
	Self-Boring Pressuremeter Test	EN ISO 22476-6
	Flat Dilatometer Test	EN ISO 22476-11

(2) <PER> Indirect determination using tests other than those listed in Table 10.3 may be used provided that the test procedure and reporting requirements have been specified by the relevant authority or agreed for a specific project by the relevant parties.

(3) <REQ> Correlations used for the indirect derivation of the shear wave velocity shall comply with 4.2.

(4) <REQ> Correlations used for standard and cone penetration tests (SPTs and CPTs) shall include the influence of:

– type of soil or grain size distribution; and
– in-situ state of stress or depth at which the measurement is taken.

(5) <RCM> Correlations used for SPTs and CPTs should also include the influence of the geological history of the soil deposit (age, stress history, and diagenesis).

NOTE 1. The uncertainty in the estimate of shear wave velocity is usually evaluated taking into account the uncertainty associated with the measured value in the test and random error and bias inherent in the correlation used.

10.5 Excess pore water pressure

(1) <RCM> The development of excess pore water pressure during cyclic loading should be investigated in the laboratory, according to 10.2 using any of the following tests:

– Cyclic Torsional Shear Test (CTS); or
– Cyclic Direct Simple Shear Test (CDSS); or
– Cyclic Triaxial Test (CTxT).

(2) <PER> When laboratory tests are not feasible, the excess pore water pressure may be determined indirectly by empirical correlations.

(3) <REQ> Empirical correlations used to determine excess pore water pressure shall account for:

– type of material;
– plasticity index and over-consolidation ratio for clays or the relative density for sands;

– effective confining pressure;
– expected level of shear strains in the soil;
– expected number of equivalent cycles.

10.6 Cyclic shear strength

10.6.1 General

(1) <REQ> Cyclic undrained shear strength shall be expressed as the number of cycles required to attain a cyclic strength limit for a given combination of average shear stress τ_a and cyclic stress τ_{cyc}.

(2) <REQ> Cyclic strength limits shall be associated with either a maximum threshold strain level or an excess pore water pressure equal to the effective stress.

(3) <PER> Threshold strain levels may be defined in terms of accumulated average shear strains (permanent) or cyclic shear strain.

(4) <RCM> The cyclic undrained strength should be determined using medium strain level cyclic tests given in Table 10.1.

(5) <RCM> Potential effects of the load frequency should be considered when selecting the testing frequency and interpreting the results.

10.6.2 Cyclic undrained shear strength of coarse soils

(1) <RCM> For assessing the response of foundations subject to cyclic actions, the effects of installation and preloading should be taken into account by applying appropriate static or cyclic preloading sequences.

NOTE 1. As an example, installation effects for a driven pile may be simulated by DSS interface shear tests where monotonic anisotropic loading is applied to mimic the effects of soil displacement followed by small amplitude cyclic pre-shearing to mimic the effects of driving blows.

(2) <REQ> Cyclic tests for testing the undrained strength shall be conducted in undrained conditions.

(3) <RCM> Each cyclic test should be defined by a value of the average (permanent) shear stress, a cyclic shear stress amplitude and shear stress frequency.

(4) <REQ> The cyclic test program should include a sufficient number of tests to reproduce the full response of the soil under cyclic loading.

NOTE 1. Depending of the nature of the cyclic event and of the stress paths in the ground, both one-way and two-way cyclic tests may be required.

(5) <PER> For Geotechnical Category 1 structures and when cyclic liquefaction cannot be determined by laboratory tests, empirical correlations with the results of field tests (EN 1998-5, 7.3.3) may be used.

(6) <REQ> Empirical correlations based on results from field investigation shall account for effective confining pressure and fine content and comply with 4.2.

10.6.3 Cyclic undrained shear strength for fine soils

(1) <RCM> The potential degradation of the shear strength during cyclic loading should be investigated with cyclic laboratory tests on specimens obtained from samples of Quality Class 1.

(2) <RCM> The influence of the strain rate should be investigated.

(3) <PER> The potential increase of undrained shear strength with rate of loading may be taken into account as specified in EN 1998-5, 6.3(4).

(4) <RCM> In total stress analysis, for assessing the response of foundations subject to cyclic loading, the effects of installation and preloading on the undrained shear strength should be taken into account by applying appropriate static or cyclic preloading sequences.

(5) <RCM> The mode of deformation imposed during the test should reproduce as close as possible the expected cyclic loading conditions, either in compression, extension, or simple shear.

(6) <RCM> Each cyclic test should be defined by a value of the average shear stress and a cyclic shear stress amplitude.

NOTE 1. Excess pore pressure criteria should not be applied to fine soils because excess pore pressures cannot be considered homogeneous within the sample.

(7) <REQ> The cyclic test program should include a sufficient number of tests to reproduce the full response of the soil under cyclic loading.

NOTE 1. Depending on the nature of the cyclic event and of the stress paths in the ground, both one-way and two-way cyclic tests can be relevant.

(8) <PER> The value of undrained shear strength derived from monotonic conditions may be used for dynamic or cyclic analyses when shear strength degradation due to number of cycles and shear strength increase due to rate of loading compensate each other.

10.6.4 Cyclic shear strength on discontinuities

(1) <RCM> Laboratory cyclic direct shear tests should be carried out on natural discontinuities to estimate the shear strength decrease.

10.7 Additional parameters for seismic site response evaluation

10.7.1 Depth to seismic bedrock

(1) <RCM> The position of the seismic bedrock should be determined with the geophysical tests given in 10.4.2 for the measurement of shear wave velocity profile.

(2) <PER> Tests for the indirect determination of the shear wave velocity may be used to evaluate the position of the seismic bedrock if they are accompanied by a direct inspection of soil samples retrieved from the bedrock layer.

(3) The position of the seismic bedrock may be determined from the measurement of compressional wave velocity, with a seismic refraction survey or a seismic reflection survey, if a suitable contrast in compressional wave velocity is observed between the bedrock and the overburden soil.

10.7.2 Fundamental frequency of soil deposits

(1) <PER> When used as an additional parameter for site categorization according to EN 1998-1, the fundamental frequency of soil deposits may be determined directly from a single-station horizontal-to-vertical spectral ratio (HVSR) survey, accounting for uncertainties and limitations of the method.

11 Groundwater and geohydraulic properties

11.1 General

(1) <RCM> Geohydraulic measurements and testing should comply with EN ISO 18674-4, EN ISO 22282 (all parts) and EN 17892-11.

(2) <REQ> The evaluation of groundwater measurements shall consider the influence of:

- geological and geotechnical conditions of the site;
- accuracy of individual measurements;
- natural fluctuations of pore and joint water pressures with time;
- duration of the observation period;
- frequency of readings;
- nearby surface water;
- the density and temperature of groundwater;
- weather and precipitation before and during the period of measurements.

(3) <RCM> The level of surface water within the zone of influence should be recorded during the period of groundwater measurements.

(4) <RCM> Measurement of the fluctuations in groundwater pressure should also be made outside the zone of influence.

11.2 Groundwater pressure and pressure head

11.2.1 General

(1) <REQ> The determination of groundwater pressure shall comply with EN ISO 18674-4.

(2) <REQ> The selection of equipment for piezometric measurements shall be based on:

- the method of installation;
- the anticipated hydraulic conductivity of the ground;
- the purpose of the measurements;
- the required observation period;
- the expected groundwater fluctuations;
- the required accuracy.

(3) <RCM> In aquicludes and ground with low hydraulic conductivity, measurements should be made using a closed system.

NOTE 1. Ground with low hydraulic conductivity includes, for example, fine soils and lightly jointed rock.

(4) <RCM> Artesian groundwater pressure should be measured using a closed system.

(5) <REQ> The number and frequency of readings and the length of the measuring period shall be planned according to the purpose of the measurements and the period needed for groundwater pressures to come into equilibrium.

(6) <PER> Piezometric level $h_{w,z}$ at elevation z may be determined from Formula (11.1):

$$h_{\mathrm{w,z}} = \frac{u}{\gamma_{\mathrm{w}}} + z \qquad (11.1)$$

where:

u is the groundwater pressure;

γ_{w} is the weight density of pore water;

u/γ_{w} is the pressure head;

z is the elevation where u is measured (positive upwards).

(7) <RCM> The weight density of groundwater should be measured according to 7.2.6.

(8) <PER> In the absence of a measured value, the weight density of fresh groundwater, γ_{w}, may be assumed to be 10 kN/m^3.

NOTE 1. The weight density of groundwater depends on its mineral content, salinity, and temperature.

(9) <RCM> The response time of the measuring system should be less than anticipated rate of variation of the groundwater pressure.

(10)<RCM> When the rate of variation is high, continuous recording systems should be used.

NOTE 1. The continuous recording includes the use of transducers and data loggers.

(11)<RCM> The reading interval should be adjusted after an initial period to accord with the actual rate of variations of the readings.

11.2.2 Test results

(1) <REQ> Correction due to atmospheric pressure shall be made when deriving groundwater pressure from measurements.

(2) <RCM> To assess groundwater pressure fluctuations, measurements should be taken at intervals appropriate to the frequency of the fluctuations.

(3) <RCM> Measurements should be performed through at least two cycles of variation in groundwater pressure.

(4) <REQ> The water level in wells, the occurrence of springs, and artesian groundwater shall be reported.

(5) <REQ> At seaside and offshore locations, the tidal water level shall be reported.

(6) <REQ> The accuracy of measurements shall be evaluated, based upon the method of reading, the system components, and accuracy of determined water density.

11.2.3 Direct determination

11.2.3.1 Open systems

(1) <RCM> Installation, measurements, and monitoring in open standpipes should comply with EN ISO 18674-4 and EN ISO 22282 (all parts).

(2) <RCM> Open standpipes installed in pre-drilled boreholes should be sealed off from layers above and below their filters.

11.2.3.2 Closed system

(1) <REQ> Installation, measurements and monitoring in closed systems tests shall comply with EN ISO 18674-4.

11.2.3.3 Cone penetration tests with pore water pressure measurement

(1) <RCM> Cone penetration tests with pore water pressure measurement should comply with EN ISO 22476-1.

NOTE 1. Cone penetration tests with pore water pressure measurement are also known as piezocone tests.

(2) <RCM> Measurements in a geotechnical unit during sounding should be performed over a length longer than 2 m.

NOTE 1. Derivation of pressure head over a length shorter than 2 m is only indicative.

(3) <PER> Measurements may also be made by pausing the sounding and allowing equilibration (see 11.2.3.3).

(4) <REQ> For equilibration, it shall be verified that the hydraulic response is drained and the pore water pressure hydrostatic over the measured length.

(5) <PER> During soundings, the pressure head in a geotechnical unit may be evaluated assuming a hydrostatic pressure gradient of 10 kPa/m.

NOTE 1. A more accurate method is a cone penetration test with pore water pressure monitoring.

(6) <RCM> Derived values should be reported together with the depth over which the hydrostatic pore water pressure is recorded.

11.3 Geohydraulic properties

11.3.1 General

(1) <RCM> Determination of hydraulic conductivity should comply with EN ISO 22282 (all parts).

(2) <REQ> The following items shall be considered when determining the hydraulic conductivity of a geotechnical unit through field tests in boreholes or laboratory testing:

- the preferred test type for conductivity determination;
- the orientation of the test and the specimen;
- evaluation of the representativeness of a soil specimen for a geotechnical unit, considering heterogeneities and changes in soil-structure during sampling;

- anisotropy;
- the need for additional classification tests.

NOTE 1. Further information on the procedure for, presentation of, and determination of hydraulic conductivity can be found in EN ISO 17892-11.

(1) <REQ> The following shall be specified for deriving hydraulic conductivity in the laboratory, depending on the conditions where the test results will be used:

- in fine 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 and the need for back pressure, under which the specimen is to be tested;
  - the required degree of saturation;
  - the chemistry of percolating water;
- in coarse soil:
  - the density index to which the specimen is to be prepared;
  - the hydraulic gradient under which the specimen is to be tested;
  - the direction of flow;
  - the need for back pressure;
  - the required degree of saturation.

(3) <RCM> The relationship between transmissivity (T), hydraulic conductivity (K), and absolute permeability (k) should be determined from Formulae (11.2)-(11.4):

$$T = KL = kL\frac{\gamma}{\eta} \tag{11.2}$$

$$K = \frac{T}{L} = k\frac{\gamma}{\eta} \tag{11.3}$$

$$k = \frac{T}{L}\frac{\eta}{\gamma} = K\frac{\eta}{\gamma} \tag{11.4}$$

where:

L length of test section in the thickness of aquifer;

η dynamic viscosity of the fluid; and

γ weight density of the fluid.

(4) <RCM> The weight density of groundwater should be measured according to 7.2.6.

11.3.2 Test results

(1) <REQ> When deriving the hydraulic conductivity from test results, it shall be verified that the flow is laminar and obeys Darcy's law.

(2) <RCM> The determination of hydraulic conductivity should take into account:

- the extent to which the boundary conditions (degree of saturation, the direction of flow, hydraulic gradient, stress conditions, density and layering, side leakage and head loss in filter and tubing) affect the test results;
- effect of scale (field tests and specimen vs the size of geotechnical unit);
- how well these conditions match the situation in the field, especially temperature and dynamic viscosity.

11.3.3 Applicability

(1) <RCM> The specimens used for laboratory hydraulic conductivity tests on fine or organic soil should be Quality Class 1.

(2) <PER> The specimens used for laboratory hydraulic conductivity tests on coarse soils may be Quality Class 2 or higher or reconstituted soil.

11.3.4 Direct determination of hydraulic conductivity

11.3.4.1 Hydraulic conductivity tests by constant and falling head

(1) <RCM> Determination of hydraulic conductivity by constant and falling head tests in the laboratory should comply with EN ISO 17892-11.

11.3.4.2 Hydraulic conductivity tests in a borehole using open systems

(1) <RCM> Determination of hydraulic conductivity in a borehole using open systems should comply with EN ISO 22282-2.

11.3.4.3 Water pressure tests in rock mass

(1) <RCM> The water intake capacity of the rock mass should be measured with Lugeon tests in boreholes.

NOTE 1. Lugeon tests indicate the water absorption capacity of rock mass and indirectly the hydraulic conductivity of the rock mass.

(2) <RCM> Field water tests should comply with EN ISO 22282-3.

(3) <RCM> Before conducting a Lugeon test, leakage from the borehole should be measured.

(4) <PER> Hydraulic apertures of joint openings may be measured in case of tight or closed joints.

11.3.4.4 Pumping tests

(1) <REQ> Field pumping tests should comply with EN ISO 22282-4.

11.3.4.5 Infiltrometer tests

(1) <RCM> Field ring infiltrometer tests should comply with EN ISO 22282-5.

11.3.4.6 Hydraulic conductivity tests in a borehole using closed systems

(1) <RCM> Determination of hydraulic conductivity in a borehole using closed systems should comply with EN ISO 22282-6.

11.3.5 Indirect determination

11.3.5.1 Cone penetration tests with pore water pressure dissipation

(1) <RCM> Indirect determination of hydraulic conductivity using cone penetration tests with pore water pressure dissipation should comply with EN ISO 22476-1.

(2) <REQ> The accuracy of the test results shall be evaluated depending on the cone used according to EN ISO 22476-1.

(3) <REQ> Dissipation test results shall be reported together with the results of CPTU soundings.

11.3.6 Empirical rules

(1) <PER> For coarse and very coarse soils, derivation of hydraulic conductivity may be based on the grain size distribution and relative density.

(2) <PER> For fine soils, derivation of hydraulic conductivity may be based on the overall content of fines.

12 Geothermal properties

12.1 General

(1) <REQ> The method to be used for measurements of geothermal properties shall be selected according to:

– the type and expected thermal conductivity of the ground;
– the purpose of the measurements;
– the required observation period;
– the expected temperature fluctuations;
– the response time of the equipment and ground.

(2) <REQ> The determination of geothermal properties shall consider the influence of:

– geological and geotechnical conditions of the site;
– accuracy of individual measurements;
– initial and anticipated natural variation of temperature in the tested volume;
– natural fluctuations of pore water pressures with time;
– chemistry of groundwater;
– permeability and groundwater flow;
– duration of the observation period;
– season of measurements;
– climatic conditions during and before the testing.

(3) <RCM> Thermal expansion and contraction should be considered when determining the strength and stiffness properties of ground that is sensitive to thermomechanical changes.

12.2 Frost susceptibility

(1) <PER> The susceptibility of a soil to frost heave may be determined directly from laboratory tests on natural, recompacted and reconsolidated, specimens or on reconstituted specimens.

(2) <PER> As an alternative to (1), susceptibility to frost heave may be determined indirectly from correlation with soil classification properties (particle size distribution, the height of capillary rise, or fines content).

NOTE 1. The height of capillary rise is defined in EN 1097-10.

NOTE 2. The frost susceptibility of soil materials plays an essential role in the design of foundations placed above the freezing front in frost susceptible soil.

NOTE 3. 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) <RCM> For soil in which (2) does not clearly indicate the absence of frost heave susceptibility, tests in the laboratory should be conducted.

NOTE 1. Examples of soil types indicating the need for laboratory tests in addition to correlations to classification properties include organic soils, peat, saline soils, artificial soils, and coarse soils with a wide range of grain size.

(4) <RCM> Frost susceptibility of soil in its natural state should be determined from intact samples of Quality Class 2 or higher.

(5) <PER> Frost susceptibility of a constructed fill may be determined by frost heave tests carried out on recompacted and reconsolidated specimens or on reconstituted specimens.

(6) <RCM> If the risk of thaw weakening is to be tested, a California Bearing Ratio (or equivalent) test should be carried out after subjecting the recompacted or reconstituted specimen to one or more freeze-thaw cycles.

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

12.3 Thermal conductivity

(1) <RCM> The determination of thermal conductivity by a Geothermal Response Test in a borehole in ground should use a borehole heat exchanger complying with EN ISO 17628.

(2) <PER> Thermal conductivity may be determined in soil and soft rock by thermal needle probe method according to ASTM D5334.

NOTE 1. ASTM D5334 presents a procedure for determining the thermal conductivity of soil and soft rock using a transient heat method.

NOTE 2. ASTM D5334 is applicable for both intact and reconstituted soil specimens and soft rock specimens and only suitable for homogeneous materials.

12.4 Heat capacity

(1) <PER> The specific heat capacity of the ground may be determined according to ASTM D4611.

NOTE 1. The value of specific heat depends upon chemical or mineralogical composition and temperature.

NOTE 2. The rate of temperature diffusion through a material, thermal diffusivity, is a function of specific heat; therefore, specific heat is an essential property of rock and soil when these materials are used under conditions of unsteady or transient heat flow.

12.5 Thermal diffusivity

(1) <PER> The thermal diffusivity of the ground may be determined according to ASTM D4612, provided the bulk mass density, thermal conductivity, and specific heat are determined under as near identical specimen conditions as possible.

12.6 Thermal linear expansion

(1) <PER> The thermal linear expansion coefficient of rock may be determined using laboratory tests complying with ASTM D4535.

12.7 Direct determination of geothermal properties

(1) <RCM> Geothermal properties should be determined by any of the methods given in Table 12.1.

Table 12.1 — Direct determination of geothermal properties

Property	Method	Standard	Applicable to			Comment
			Soil	Rock	Fluid	
Thermal conductivity	Multi-probe method	ASTM D5334	Yes	Yes		Transient field and lab. method
	Single-probe method (needle-probe)		Yes	Yes	Yes	Transient field and lab. method
	Divided-bar method			Yes		Stationary laboratory method
	Transient plane source (TPS)		Yes	Yes	Yes	Transient laboratory method
Thermal diffusivity	Multi-probe method	ASTM D4612	Yes	Yes		Transient field and lab. method
	Single-probe method (needle-probe)		Yes	Yes	Yes	Transient field and lab. method
	Transient plane source (TPS)		Yes	Yes	Yes	Transient laboratory method

(2) <PER> Geothermal properties may be determined by theoretical calculation from the knowledge of rock and soil mineral content, porosity and water content.

13 Reporting

13.1 Ground Investigation Report

(1) <REQ> The results of a ground investigation shall be compiled in a Ground Investigation Report.

NOTE 1. Guidance on the content of the Ground Investigation Report is given in Annex A.

(2) <REQ> EN 1997-1, 12, shall apply.

(3) <REQ> The Ground Investigation Report shall document the Ground Model.

Annex A

(normative)

Ground Investigation Report

A.1 Use of this Annex

(1) This Normative Annex contains additional provisions to Clause 13 for preparing the Ground Investigation Report.

A.2 Scope and field of application

(1) This Normative Annex covers contents of the Ground Investigation Report.

A.3 Contents of the Ground Investigation Report

(1) <RCM> The Ground Investigation Report should include, but is not limited to, the following information:

1. Project name
2. Proposed structure stage of execution relevant for GIR, scope of investigation
3. Normative references
4. List of information used to plan the ground investigation
5. Geotechnical Category (selection for ground investigation purposes)
6. Site overview
    a. For land-based projects: topography, existing structures, vegetation, nearby open water
    b. For near shore projects: current tidal levels and bathymetry
7. Location (coordinates)
8. Desk study
9. Site inspection
10. Geological and hydrogeological studies
11. Geophysical surveys or measurements
12. Field investigations
    a. Dates of fieldwork
    b. Names and qualifications of field personnel
    c. Type of equipment
    d. Calibration certificates and documents
    e. List of performed investigations and locations
    f. Environmental conditions during field investigation;
    g. Geodetic position and level of samples and tests performed;
    h. Geodetic position and level of the monitoring probes;
    i. Handling of samples
    j. Main site observations during the investigation
13. Laboratory testing
    a. List of investigations performed and on which samples
    b. Dates tests performed
    c. Names and qualifications of laboratory personnel
    d. Calibration certificates and documents

    e. Main observations during testing (quality, sample content)
14. Groundwater investigations
    a. List of field investigation performed and their locations (short and long term)
    b. Time period of investigation
    c. Names and qualification of field personnel
    d. Calibration certificates and documents
    e. Handling of samples
    f. Main observations during the investigation
15. Presentation and review of monitoring results
16. Derived values of ground properties
    a. State, physical, and chemical properties
    b. Strength properties
    c. Stiffness and compressibility properties
    d. Cyclic, dynamic, and seismic properties
    e. Groundwater and geohydraulic properties
    f. Geothermal properties
    g. Other relevant properties
    h. Information referred in 4.2 (4)
17. Ground Model
18. Review of results
    a. Any limitations, discrepancies, uncertainties, or gaps in the data
    b. Any deviation from the standard procedures for field and laboratory testing

(2) <RCM> Additional information than given in (1) should be included as necessary:

a. Significant variations of consistency of the ground (weaker or stronger)
b. Apparently anomalous or outlier results for a ground property
c. Geometrical irregularities including cavities and zones of discontinuous material
d. Important observations from the field and laboratory testing and from the monitoring

(3) <RCM> The following should be added to the GIR as referenced reports:

– Field reports;
– laboratory test reports;
– field investigation and monitoring reports;
– desk studies; and
– geological and hydrogeological studies.

(4) <RCM> The following should be added to the GIR as separate annexes:

– Tabulation and graphical presentation of the field investigation and laboratory test results;
– field reconnaissance reports;
– evaluated soundings with derived values;
– graphical presentations of derived value;
– estimates of the coefficients of variation of ground properties.

(4) <RCM> Plans, sections, and profiles with investigation locations should be added to the GIR.

(5) <PER> A Building Information Model may be included in the GIR.

Annex B

(informative)

Suitability and applicability of test methods

B.1 Use of this Informative Annex

(1) This Informative Annex provides supplementary guidance to Clauses 4-6 for suitable methods of test in investigation.

NOTE 1. National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

B.2 Scope and field of application

(1) This informative Annex covers identification of the suitability and applicability of test methods given in EN 1997-2.

B.3 Suitability of test methods

(1) <RCM> Designers of ground investigations should consider the relationship between the proposed structure, the necessary geotechnical information and the appropriate methods of ground investigation that can be deployed.

NOTE 1. An indication of the suitability of the test methods covered by EN 1997-2 is given in Table B.1.

NOTE 2. Guidance for use of Table B.1: for a proposed geotechnical structure or works (left hand column in top half of table) the ground information needed is identified and ranked in the top row. To obtain the information follow the ground information needed column down to the lower half of the table and thereby identify appropriate methods of investigation (left hand column in lower half of table).

NOTE 3. An illustration is highlighted and arrowed where for Spread Foundations, information on the disposition and nature of geotechnical units is of High Relevance and, following down, this might be found by, amongst other methods, where Sampling and Laboratory testing is of High Applicability.

>

Table B.1 — Guidance on appropriate methods of ground investigation

Ground information needed (EN 1997-2 clauses) H = High relevance M = Medium relevance L = Low relevance			5.2.1 Desk Study - history and past uses of site	5.2.2 Site Inspection - ground	5.2.3 Disposition and nature of geotechnical units	5.2.4 Groundwater conditions	5.2.4 Geohydraulic properties	5.2.5 Geotechnical monitoring	6 Description and classification of ground	7.1 and 7.2 Physical properties	7.3 Chemical properties	8 Strength propertiers	9 Stiffness properties	10 Cyclic response and seismic properties	11 Groundwater and geohydraulic properties	12 Geothermal properties	Presence of voids (natural or man-made)	Properties of material for reuse	Contaminated ground	Aggressive ground
Proposed structures and engineering works	Structures (EN 1997-3)	4 Excavations, cuttings	C	C	H	H	H	H	H	H	L	H	M	M	H	L	M	M	L	L
		4 Embankments	C	C	H	H	M	H	H	H	L	H	H	M	H	L	M	M	M	L
		5 Spread foundations	C	C	H	H	H	M	H	H	M	H	H	M	H	L	H	L	H	H
		6 Piled foundations	C	C	H	H	H	M	H	H	H	H	H	M	H	M	H	L	H	H
		7 Retaining structures	C	C	H	H	H	H	H	H	H	H	H	M	H	L	M	H	M	M
		8 Anchors	C	C	H	H	M	M	H	H	H	H	H	M	H	L	M	L	M	H
		9 Reinforced fill structures	C	C	H	H	H	M	M	H	H	H	H	M	H	L	M	M	M	H
		10 Ground reinforcing elements	C	C		H	M	M	H	H	H	H	H	M	H		M	L	M	H
		11Ground improvement	C	C	H	H	M	M	M	H	M	H	M	M	M	L	L	L	M	H
		12 Groundwater control	C	C	H	H	H	H	H	H	M	M	M	M	H	L	H	L	H	M
	Miscellaneous works	Linear - roads	C	C	H	H	M	M	H	H	M	H	H	M	H	L	H	H	H	H
		Linear - pipelines	C	C	H	H	H	M	H	H	H	H	H	M	H	L	M	H	H	H
		Linear – tunnels	C	C	H	H	H	H	H	H	L	H	H	M	H	L	H	H	M	M
		Underground openings	C	C	H	H	H	M	H	H	L	H	H	M	H	L	H	M	M	M
		Dams and weirs	C	C	H	H	H	H	H	H	H	H	H	H	H	L	H	H	L	M
		Construction materials	C	C	H	H	L	M	H	H	H	H	H	L	H	L	L	H	H	H
		Ground source heat installations	C	C	H	H	H	L	H	H	H	L	L	L	H	H	L	L	M	H

Appropriate methods of investigation (Clause 5.3)
H = High, M = Medium , L = Low applicability
" " = not applicable

Mapping and remote sensing	H	H	M	H	L		M						L		H		M	M
Probing		M	H	M	L	L	H	L	M	H	H	M	M	L	H	L	M	L
Boreholes		L	H	H	H	M	H	H	M	M	M	L	H	M	M	M	H	M
Test pits		M	H	H	M	L	H	H	M	M	L	L	H	L	L	H	H	H
Geophysical tests		H	H	M	L	L	M	H	L	H	H	H	M	M	H	L	L	L
Field testing		M	H	H	H	H	M	H	H	H	H	H	H	H	L	M	L	M
Sampling and laboratory testing		M	H		H	H	H	H	H	H	H	H	H	H		H	H	H
Description and classification of ground	H	H	H			H	H	H	M	H	M	M	M	M	H	M	H	H
Groundwater conditions	H	H		H	H	H			M			M	H	H	M		H	H
Geohydraulic testing				H	H	H						M	H	H	H		H	H
Geothermal testing														H				
Monitoring						H								H	M		M	
Large scale tests of prototypes								H		H	H	M	M			H		
Back analysis of structures				H				H		H	H	H	M			M		
Back analysis of slopes				H				H		H	H		H			M		

B.4 Applicability of field investigation and laboratory tests

(1) <RCM> Designers of ground investigations should consider the applicability of field investigation techniques covered by Clause 5 when planning field investigations, as indicated in 5.4.

NOTE 1. An indication of the applicability of field investigation and laboratory tests covered by EN 1997-2 is given in are given in Table B.2 and Table B.4.

NOTE 2. The confidence levels that appear in this Annex refer only to the confidence derived from the intrinsic characteristics of the tests.

<Drafting note: Layout of Table B2 and B3 to be improved.

Table B.2 — Simplified overview of the applicability of laboratory tests covered by Clauses 7 to 10 – part 1

	Laboratory Tests																										
Property	Atterberg - Casagrande	Atterberg - Fall cone	Atterberg - Tread Method	CBR-test	Chemical teststables 7.8 and 7.9	Crumb test	Double Hydrometer test	Drying in ventilated oven	Hole ersion test	Immersion in water	ISRM – Water content	Jet erosion test	Laser diffraction	Linear measurement Immersion in fluid Fluid displacement	Mercury Intrusion porosity	Methylene blue test	Oven drying at 105^0	Pinhole test	Part. Size distribution after compaction	Poctor compactionr	Sedimentation method	Shrinkage Limit	Sieve method	Vibrating Hammer/Table	Water absorption Coefficient by capillary	Water method	X-ray gravitational
7.1.2 Mass bulk density											R2-3			FC3											FC3		
7.1.3 Water content								FC3			R3						FC3										
7.1.4 Macropores porosity															FCR3											FCR2-3	
7.2.1 Grain size distribution													FC2								F2-3		C3				FC2
7.2.2 Plastic Limit			F2																								
7.2.2 Liquid Limit	F2	F3																									
7.2.2 Methylene blue value																F3											
7.2.2 Shrinkage Limit																						F2-3					
7.2.8 Stability										R3																	
7.2.8 Dispersibility						F1	F2											F2									
7.2.8 Critical stress and erosion coefficient									F3			F2															
7.2.9 Ref density and water content																				CF3				CF3			
7.2.9 CBR				FC 2-3																							
7.2.9 Fragmentability and degradability (aggregates)																			C3								
7.3 Chemical properties*1					FCR3																						
F = Fine Soils, C = Coarse Soils, R = Rock, 1 = Low Applicability, 2 = Medium Applicability, 3 = High Applicability																											
*1 Mineralogy, Carbonate content, Organic content, Sulphate/Sulphide, Acidity and Alkalinity, Chloride, Others																											

Table B.3 — Simplified overview of the applicability of laboratory tests covered by Clauses 7 to 10 – part 2

	Laboratory Tests																								
Property	Atterberg - Casagrande	Atterberg - Fall cone	Atterberg - Tread Method	BE – Bender Element test	CDSS -Cyclic Direct Simple Shear	CTS-CyclicTorsional Shear	CTX – Cyclic Triaxial Test	Direct shear test	DSS – Direct Simple shear	ISRM- creep characteristics of Rock Methods	ISRM – Huder Amberg method	ISRM - TX – Consolidated triaxial compression test-	ISRM - UCT-Unconfined Compression test	IST – Interface Shear Test	OED CRS Oedometer – CRS	OED – IL Oedometer – incremental	Point Load test	P-Wave	RC – Resonant Column Test	Ring shear test	Schmidt Hammer test	Swelling tests - Other methods – see Table 9.6	TX – Consolidated triaxial compression test	UCT -Unconfined Compression test	UUTX -Unconsolidated Undrained triaxial test
7.1.7 At rest coefficient K0															F3	F3							FC2-3		
7.1.7 Pre-consolidation, OCR															F3	F3									
8.2 Soil Strength	F1	F1	F1					FC2	FC3											FC3			FC3	F2	F2
8.3 Rock Strength												R3	R3				R2				R2				
9.1 Oedometer Modulus															F3	F3									
9.1 E-modulus							FC3					R3	R3	R2				FC3					FC3	F3	
9.1 Shear modulus				FC2-3			FC3		FC3					R3					FC3				FC3		
9.2 Compression, Consolidation and Creep Properties	F1	F1								R3					F3	F3							F3		
9.2.4 Swelling properties											R2-3				F3	F3						F2-3			
10.3 Secant shear modulus and damping ratio curves					FC3	R3	FC1												FC3						
10.4 Very small strain shear modulus				FC2	FC1	R2	FC1												FC2						
10.5 Excess pore pressure					FC3	R2	FC3																		
10.6 Cyclic shear strength					FC3	R2	FC3																		
F = Fine Soils, C = Coarse Soils, R = Rock, 1 = Low Applicability, 2 = Medium Applicability, 3 = High Applicability																									

Table B.4 — Simplified overview of the applicability of field investigation tests covered by Clause 5

	Field investigation tests																									
Property	BDP Borehole Dynamic penetration test	BJT Borehole Jack Test	BST Borehole Shear test	CPT/CPTU Cone penetration test	DMT Flat Marchetti dilatometer test	DPT Dynamic Penetration test	Electrical density method	FDP Full displacement poressiometer	FDT Flexible Dilatometer test	FVT Field Vane Test	ISRM-Flat Jack	ISRM – Geophysical Methods	ISRM – Hydraulic Fracturing	ISRM – Overcoring in borehole	ISRM – Total pressure Cells	Loose bulk density and voids	MPM Ménard Pressiometer	Nuclear methods	PBP Pre-bored pressiometer	PLT Plate Loading Test	Sand replacement method	SBP Self-boring pressiometer	SCPT/SDMT/SPBP Seismic tests	SPT Standard penetration Test	Total Pressure Cells	WST Weight Sounding test
7.1.2 Bulk mass density						C2	FCR 2–3									C2		FCR2 – 3			C2					
7.1.3 Water content																		FC2								
7.1.6 Density Index	C2			C2		C1											C2							C1		C1
7.1.7 Horizontal stress,					FC3			FC 1-2			R3						FC2		FCR 2-3			FC3			FC3	
7.1.7 Hor stress state / orientation													R2-3													
7.1.7 insitu stress state (stress tensor)														R3												
8.2 (Undrained) strength			CR2	FC3	FC3	C2		FC3		F3							FC3		FCR3			FC3		C2		
9.1 Oedometer modulus				FC2					FC3															FC1		
9.1 E-Modulus		R2		FC1	FC3	FC1		FC3	FC3								FC3		FC3	C2		FC3		FC1		
9.1. Shear Modulus				FC2				FC3				R3					FC3		FC3			FC3	FCR3	FC1		
9.2 Horizontal consolidation c_h				F2-3	F2-3				F2-3													F3				
10.4 Shear wave velocity				FC1	FC1			FC1									FC1		FCR1			FC1	FC3	FC1		

F = Fine Soils, C = Coarse Soils, R = Rock, 1 = Low Confidence/Applicability, 2 = Medium Confidence/Applicability, 3 = High Confidence/Applicability

B.5 National standard for investigation and laboratory tests

(1) <PER> In absence of published European standards for field investigation and laboratory testing, national standards may be applied.

NOTE 1. Guidance on available national standards for state, physical and chemical properties is given in Table B.5, for strength properties in Table B.6, for stiffness properties in Table B.7, for compressibility, consolidation and swelling in Table B.8, for response to cyclic and dynamic actions in Table B.9, for shear and compressional wave velocities in Table B.10 and for geothermal properties in Table B.11, unless the National Annex give different references.

Table B.5 — List of national test standards for state, physical and chemical properties

Property	**Method**	**Standard**	**MQC**	**Comments on suitability and interpretation**
Table 7.1 Field and laboratory test to determine state properties				
Bulk mass density (ρ)	Nuclear gauge	NF P 94-061-1 ASTM D6938 - 17a	-	Presence of nuclear source as a hazard
	Electrical density method	ASTM D7698 - 11a	-	-
Water content (w)	Water content	ISRM Suggested Methods	2	For rock
Porosity	Mercury intrusion porosimetry for soil	ASTM D4044		
	Porosity of rock by saturation and caliper	ISRM Suggested Methods	-	Determination of porosity and density of rock
	Porosity of rock by saturation and buoyancy	ISRM Suggested Methods	-	Determination of porosity and density of rock
Table 7.2 Field and laboratory test to determine in-situ stress parameters				
In-situ stress state component	Flat jack	ISRM suggested method	-	Measured stress component in a rock surface
In-situ stress state: minimum/maximum horizontal stresses and orientation/components of the stress tensor	Hydraulic fracturing in a borehole/ hydraulic tests on pre-existing joints	ISRM suggested methods	-	Vertical axis often considered as one principal direction and vertical stress magnitude equals the weight of the overburden
In-situ stress state in rock: independent components of the stress tensor	Over coring in a borehole	ISRM suggested methods	-	Elastic parameters of the rock required
Pre-consolidation pressures (σ'_p), over-consolidation ratio (OCR)	Constant rate of strain oedometer test	ASTM D4186	1	-

Property	Method	Standard	MQC	Comments on suitability and interpretation
Table 7.4 Laboratory tests to determine consistency limits				
Shrinkage limit (w_S)	Volumetric or linear method	NF P94-060.1 NF P94-060.2 DIN 18122-2 ASTM D427	2	For fine soils
Table 7.5 Laboratory tests to determine rock physical properties				
Weathering and alteration		EN ISO 14689 ISRM suggested method	-	-
Abrasivity		ISRM suggested method NF P94-430-1,2 ASTM D7625-10	-	-
Table 7.6 Field and laboratory tests to determine stability, dispersibility and erodibility properties				
Dispersibility	Double Hydrometer Test	BS 1377-5 ASTM D4221-99	4	Compares the dispersion of clay particles in plain water without mechanical stirring with that obtained using a dispersant solution and mechanical stirring Qualitative evaluation
	Crumb Test	BS 1377-5 ASTM D6572-00	2	Stability of soil aggregates subjected to the action of water Qualitative evaluation
	Pinhole test	BS 1377-5 ASTM D4647-93	2	Need to consider specifying different compaction conditions for specimens Avoid drying of the specimen before testing Qualitative evaluation of internal erosion
Critical stress and erosion coefficient	Jet erosion test	ASTM D5852-95	2	In-situ or laboratory on small surface Representativeness External erosion
Table 7.7 Laboratory tests to determine compaction properties				
Fragmentability and degradability	Evolution of particle size distribution after dynamic compaction or humidification drying of soil	NF P 94-066 NF P 94-067	4	for aggregates
Table 7.8 Laboratory tests to determine chemical properties of ground				
Carbonate content	Loss in dry weight after reaction with hydrochloric acid	ASTM D4374	3	-
Table 7.9 Laboratory tests to determine chemical properties of groundwater				
Carbonate content	-	ASTM D4373	-	-

Table B.6 — List of national test standards for strength properties

Property	Method	Standard	MQC	Comments on suitability and interpretation
Table 8.1 Direct determination of soil strength properties				
Peak effective cohesion and friction (c'_p, φ'_p)	Direct simple shear	ASTM 6528-17	1	See 8.2.1 (4) to (10)
Angle of friction critical state (φ'_{cs})	Direct simple shear	ASTM 6528-17	1-4	See 8.2.1 (4) to (9)
Peak undrained cohesion ($c_{u,p}$)	Laboratory vane	ASTM D4648	1	
	Direct simple shear	ASTM 6528-17	1	
Remoulded undrained cohesion $c_{u,rmd}$	Laboratory vane	ASTM D4648	1	
Table 8.3 Determination of rock material strength properties				
Compressive strength (σ_{ci})	Unconfined compression test (UCT)	ISRM Suggested Methods (2017)	-	See (2) and (3) See ISRM (2007a)
	Triaxial test (TX)	ISRM Suggested Methods (2017)	-	See (2) and (3) See ISRM (2007b)
	Point load test	ISRM Suggested Methods (2017)	-	See (2) and (3)
	Schmidt hammer test	ISRM Suggested Methods (2017)	-	See (2) and (3)
Parameter (m_i)	Triaxial test (TX)	ISRM Suggested Methods (2017)	-	See (2) and (3) See ISRM (2007b)
Tensile strength (σ_t)	Direct tensile tests	ISRM Suggested Methods (2017)	-	See (2) and (3) See ISRM (2007b)
	Point load test	ISRM Suggested Methods (2017)	-	See (2) and (3)
Flexural strength (σ_{fl})	3 and 4-point bend tests for flexural strength	ASTM C880-98	-	See (2) and (3)
Table 8.4 Determination of strength properties for rock discontinuities				
Peak shear strength along discontinuity (c'_p, φ'_p)	Direct shear of rock discontinuities	ISRM Suggested Methods (2017)	-	See (6)-(8) Only derived for a certain range at a particular value of σ_3 or σ_n
	Triaxial test (TX)	ISRM Suggested Methods (2017)	-	See (6)-(8)
Basic shear strength along discontinuity (φ_b)	Basic friction angle of rock discontinuities	ISRM Suggested Methods (2017)	-	See (6)-(8). The tilt test defines under which dip angle the upper block on a discontinuity starts sliding
Residual shear strength of discontinuity (φ_r)	Residual friction angle of rock discontinuities	ISRM Suggested Methods (2017)	-	See (6)-(8)

Table B.7 — List of national test standards for stiffness

Strain level	Property	Test	Standard	Comments on suitability and interpretation
Table 9.1 Direct determination of ground stiffness properties from field investigation				
Very small (< 10^{-5})	E_{rm}	Rigid Plate Loading	ASTM D4394-17	-
		Flexible plate loading method	ASTM D4394-17	-
		Radial jacking test	ISRM suggested methods	-
		Large flat jack tests	ISRM suggested methods	-
Medium (10^{-2}-10^{-1})	-	Drill hole deformation gauges	ISRM suggested methods	Full curve-
Table 9.2 Direct determination of ground stiffness properties from laboratory tests				
Very small (< 10^{-5})	G_0	Resonant column tests	ASTM D4015-15	Several/full curve
Small (10^{-5}-10^{-2})	G, G_{cyc}	Consolidated Undrained Direct Simple Shear Testing	ASTM 6528-07	Several/full curve
	G, E	Triaxial tests for rock specimens (with global or local strain measurement)	ISRM suggested methods	-
	G, E	Direct shear test for discontinuities (for normal and tangential stiffness)	ISRM suggested methods	-
	G_{cyc}, E_{cyc}	Determination of the Modulus and Damping Properties of Soils Using the Cyclic Triaxial Apparatus (CTxT)	ASTM D3999-91	Several/full curve
	$G_{0,RC}$	Modulus and Damping of Soils by Fixed-Base Resonant Column Devices (RC)	ASTM D4015-15	Several/full curve
Medium (10^{-2}-10^{-1})	E_{OED}	Constant Rate of strain test (CRS)	ASTM D4186-6 SS 27126	Full curve
	G, G_{sec}	Consolidated Undrained Direct Simple Shear Testing (DSS)	ASTM 6528-17	Several/full curve
	G, E	Consolidated triaxial compression tests on water saturated soils (with measurement of local strains)	EN ISO 17892-9	Several/full curve
	G, E	Triaxial tests for rock specimens (with global or local strain measurement)	ISRM suggested methods	-
	G, E	Direct shear test for discontinuities (for normal and tangential stiffness)	ISRM suggested methods	-
	E	Unconfined compression test (UCT)	ISRM suggested methods	Several
	K_s, K_n	Discontinuity shear test	ISRM suggested methods	-
Large (> 10^{-1})	G, G_{sec}	Consolidated Undrained Direct Simple Shear Testing (DSS)	ASTM 6528-17	Several/full curve
	G, E	Triaxial tests for rock specimens (with global or local strain measurement)	ISRM suggested methods	-
	G, E	Direct shear test for discontinuities (for normal and tangential stiffness)	ISRM suggested methods	ISRM suggested methods
	G, E	Unconfined Compression Test (for rocks)	ISRM suggested methods	-

Table B.8 — List of national test standards for compressibility, consolidation and swelling

Property	Method	Standard	MQC	Comments on suitability and interpretation
Table 9.4 Direct determination of compression and consolidation properties				
Compression index (C_c)	Constant rate of strain	ASTM D4186-6 SS 27126	1	1-dimensional value
Recompression index (C_r)	Constant rate of strain	ASTM D4186-6 SS 27126	1	Single value
Pre-consolidation pressure (σ'_p)	Constant rate of strain	ASTM D4186-6 SS 27126	1	1-dimensional value
Coefficient of vertical consolidation (c_v)	Constant rate of strain (CRS)	ASTM D4186-6 SS 27126	1	At any set of readings
Table 9.6 Direct determination of swelling properties from laboratory tests				
Swelling pressure (σ_g)	One specimen with axial surcharge Huder Amberg method	ISRM suggested methods	n/a	σ'_{vo} deduced from the Ground Model should be provided
	under zero volume change	NF P94-090 UNE 103602:1996 ASTM D4546	1	Specific to stress path
	Several specimens with axial surcharge	NF P94-091	-	-
Swelling amplitude (ε_g)	Free swelling	NF P94-090 UNE 103602:1996	1	Specific to stress path
	Linear swelling	EN 13286-47	-	Unbound and hydraulically bound mixtures
Swelling coefficient (C_g)	Several specimens with axial surcharge; one-Dimensional Swell or Settlement Potential	NF P 94-091 DIN 18135-K BS 1377 ASTM D2435 and D4546	1	Pressures should be specified
	Huder Amberg method	ISRM suggested methods		
Swelling index (C_{sw})	Constant rate of strain test	ASTM D4186-6 SS 27126		-

Table B.9 — List of national test standards for response to cyclic and dynamic actions

Table 10.1 Laboratory tests for measuring response to cyclic and dynamic actions						
Test	**Cyclic torsional shear**	**Cyclic direct simple shear**	**Cyclic triaxial**	**Resonant column**	**Bender elements**	**Cyclic triaxial for rock**
Standard	JGS0543	ASTM D8296-19	ASTM D3999 ASTM D5311	ASTM D4015-07	ASTM D8295-19	JGS 2561 JGS 2562

Table B.10 — List of national test standards for determine shear and compressional wave velocities

Parameter	Test	Standard
Table 10.2 Geophysical tests to determine shear and compressional wave velocities		
Shear wave velocity (vS)	Cross-Hole Test	ASTM D4428/D4428M-14
	Down-Hole Test	ASTM D7400-19
	P-S suspension logging test	-
	Seismic Refraction	ASTM D5777-18
	Seismic Cone Penetration Test	-
	Seismic Flat Dilatometer Test	-
	Surface Wave Methods	-
Compressional wave velocity (vP)	Cross-Hole Test	ASTM D4428/D4428M-14
	Down-Hole Test	ASTM D7400-19
	P-S suspension logging test	-
	Seismic Refraction	ASTM D5777-18
	Seismic Reflection	ASTM D7128-18

Table B.11 — List of national test standards for determine geothermal properties

Property	Method	Standard	Comments on suitability and interpretation
Table 12.1 Direct determination of geothermal properties			
Thermal conductivity	Multi-probe method	ASTM D5334	Transient field and lab. method
	Single-probe method (needle-probe)	ASTM D5334	Transient field and lab. method
	Divided-bar method	ASTM D5334	Stationary laboratory method
	Transient plane source (TPS)	ASTM D5334	Transient laboratory method
Thermal diffusivity	Multi-probe method	ASTM D4612	Transient field and lab. method
	Single-probe method (needle-probe)	ASTM D4612	Transient field and lab. method
	Transient plane source (TPS)	ASTM D4612	Transient laboratory method

Annex C

(informative)

Desk study and site inspection

C.1 Use of this Informative Annex

(1) This Informative Annex provides supplementary guidance to Clause 5.2 regarding the desk study and site inspection.

NOTE 1. National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

C.2 Scope and field of application

(1) This Informative Annex covers desk studies and site inspection.

C.3 Desk study

(1) <RCM> The desk study should comprise factual information supplemented by interpretation to summarize surface, geological, geo-environmental and geotechnical aspects of the site in the formulation of the ground model.

(2) <RCM> The successive stages of assessment and investigation should identify potential geotechnical, environmental and health and safety issues that are likely to affect the site, its investigation and its development.

(3) <RCM> Sources of information to be consulted should include, when available:

- site details:
  - location (address, coordinates);
  - boundaries;
  - land ownership;
  - present and proposed land use;
  - site protection and environmental status;
  - topographic maps and site surveys including drainage courses;
  - presence of services and utilities (above and below ground);
  - remotely sensed images;
  - details of site accesses, and other relevant information.
- site history:
  - historical maps, photographs, remotely sensed images;
  - maps and documentary evidence of past site usage;
  - identification of changes in topography and unstable ground;
  - the presence of watercourses and potential for flooding;
  - archaeological potential; the presence of and protective designation;
  - man-made structures including foundations, infrastructure and mine workings;
  - the potential for anthropogenic contamination or naturally occurring harmful substances given current/past uses of the site and other relevant information.

- site geology:
  - geological, engineering geological, geomorphological, soil and hydrogeological maps and memoirs;
  - reports and other documents including digital data;
  - borehole logs and well records;
  - past ground investigations in the vicinity, seismological information and records;
  - information on natural voids and anthropogenic cavities.
- previous experience:
  - previous experience in the area;
  - performance of other constructions in the area;
  - properties of similar ground from the site or elsewhere.
- database of geotechnical and geological information:
  - historical maps;
  - archive material of previously constructed structure in the zone of influence;
  - stress fields in use for rocks (World stress map)

(4) <RCM> Interpretation of the desk study should include:

- ground-related site constraints:
  - cataloguing of the identified site-specific factors that might affect the ground investigation and development proposals;
- ground-related hazards:
  - list the identified ground hazards (both site- and project-specific) and identify and prioritize proposals for further investigation and subsequent mitigation;
  - ground hazards can be topographic, geological, hydrogeological or man-made;
  - assessment of the information for reliability and completeness in terms of identifying possible hazards;
  - possible unexploded ordnance;
  - list of potentially seismic active faults.

(5) <RCM> Recommendations for ground investigation should be made and include the following:

- recommendations for the scope of the ground investigation required;
- specific site/project-specific issues identified which require particular investigation;
- sources of construction materials including water supplies.

C.4 Site inspection

(1) <RCM> The following information should be collated in preparation for carrying out the site inspection:

- site maps and plans, district maps or charts, geological maps, and remotely sensed data;
- permission to gain access from both owner and occupier;
- listing of items of evidence which are lacking or where local verification is needed on a particular matter;
- information about the local area including excavations, exposures, structures of relevant interest, underground structures;
- health and safety risk assessment including natural and anthropogenic hazards.

(2) <RCM> The site inspection should be carried out once the factual information for the site and its environs has been compiled (the desk study) in order to collect additional information on the geology

and hydrogeology, relevant geotechnical conditions, potential construction and access and environmental constraints for ground investigation.

(3) <RCM> Items to inspect during the site inspection should include:

- geotechnical, geological, and geomorphological conditions;
- indications of ground water;
- ground stability or instability;
- vegetation and changes in vegetation;
- current and former drainage systems;
- openings to underground structures, tunnels or mines;
- indications of excavation and their backfilling;
- the presence of harmful or toxic material in any form;
- the presence and location of previous structures;
- the presence of any designated historical asset or monument;
- any indication of contamination or the presence of potentially harmful soil gases;
- ecological conditions (including protected flora and fauna);
- access routes and storage areas for investigation and construction;
- sources of construction materials including water supply for construction;
- availability of utilities (water, gas, telecommunications) for investigation and construction.

(4) <RCM> The site inspection should include activities and observations as follows:

- traverse the whole area, preferably on foot;
- set out the proposed location of work on plans;
- inspect and record details and integrity of existing structures;
- check access, including the probable effects of investigation plant and construction traffic and heavy construction loads on existing roads, bridges and services;
- check and note water levels, direction and rate of flow in rivers, streams and canals, and also flood levels and tidal and other fluctuations, where relevant;
- observe and record:
    - adjacent property and the likelihood of its being affected by proposed works and any activities that might have led to contamination of the site under investigation;
    - mine or quarry workings, old workings, old structures, and any other features that might be relevant;
    - any obvious immediate hazards to public health and safety (including to trespassers) or the environment;
    - any areas of discoloured soil, polluted water, distressed vegetation or significant odours;
    - any evidence of gas production or underground combustion;
    - tree types and locations if site underlain by fine soils;
    - obstructions;
    - differences and omissions on plans and maps;

NOTE 1. Obstructions can include transmission lines, ancient monuments, trees subject to preservation orders, manhole covers, gas and water pipes, electricity cables, sewers.

NOTE 2. Differences and omissions can include: boundaries, buildings, roads and transmission lines

- observe the ground morphology and associated features to provide information on the geomorphology of the site and surrounding area, including:
    - type and variability of surface conditions;

- comparison of surface topography with previous maps to check for presence of fill, erosion or cuttings;
- in mining areas steps in surface, mining subsidence, compression and tensile damage in brickwork, buildings and roads structures out of plumb;
  mounds and hummocks in more or less flat country which frequently indicate former glacial conditions;
- mounds and hummocks or depressions which can also indicate historical mining;
- broken and terraced ground on hill slopes, small steps and inclined tree trunks;
- crater-like holes in chalk or limestone country;
- low-lying flat areas in hill country, sites of former lakes and the presence of soft silty soils and peat;
- details of ground conditions in exposures in quarries, cuttings and escarpments, on-site and nearby;
- ground water level or surface water levels, positions of wells and springs, any signs of artesian flow;
- record the vegetation in relation to the soil type and to the wetness of the soil, unusual green patches, or varieties indicating wet ground conditions;
- study embankments, buildings and other structures in the vicinity having a settlement history, in particular, looking for cracks in walls, subsiding floors, and other structural defects.

(5) <RCM> The inspection should be enhanced considerably by suitably referenced photographs.

(6) <RCM> Inspection of the site for ground investigation purposes should include:

- the location and conditions of access to working sites;
- obstructions such as overhead or underground pipes and cables, boundary fences and trenches, trees and other vegetation clearance requirements;
- environmental conditions;
- areas for depot, offices, sample storage, field laboratories;
- ownership of working sites;
- liability to pay compensation for damage caused;
- suitable water supply where applicable and record location and estimated flow;
- suitable means of disposing of solids and liquid arising from the investigation;
- particulars of lodgings and local labour;
- particulars of local telephone including mobile phone reception, employment, transport and other services;
- surface conditions at each exploratory location and the particular reinstatement requirements;
- details of post investigation access to instrumentation and any requirements to protect the instrument;
- mapping of visible geotechnical and geological features.

NOTE 1. Reinstatement requirements include e.g. breaking out pavement and replacement.

NOTE 2. Requirements to protect instrument e.g. fencing.

(7) <RCM> Determination of rock quality, rock outcrops, zones of degradation, and discontinuities should be included in site inspections.

Annex D

(informative)

Information to be obtained from ground investigation

D.1 Use of this Informative Annex

(1) This Informative Annex provides supplementary guidance to Clause 6 for information to be obtained from ground investigation.

NOTE 1. National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

D.2 Scope and field of application

(1) This Informative Annex covers the information to be obtained from ground investigation.

D.3 Information to be obtained from ground investigation

(1) <RCM> The information obtained from the ground investigation should enable assessment of the following aspects for execution:

- the suitability of the site with respect to the proposed execution and the level of acceptable risks;
- the level and type of uncertainty of the ground investigation results with respect to the proposed execution and the level of acceptable risks;
- the deformation of the ground caused by the structure or resulting from execution, its spatial distribution and behaviour over time;
- the safety with respect to limit states, including subsidence, ground heave, uplift, slippage of soil and rock masses, and buckling of piles;
- the loads transmitted to the structure from the ground and the extent to which they depend on its design and execution, including:
  - the foundation construction methods;
  - the sequence of construction works;
  - the effects of the structure and its use within the zone of influence;
  - the need for and types of ground improvement;
  - any additional structural measures required;
  - the potential for seismic ground motion amplification and soil liquefaction;
  - the possible densification under dynamic and seismic loads;
  - 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;
  - health and safety risks from natural and anthropogenic hazards
  - identification of potentially seismic faults.

(2) <RCM> The information obtained from investigation for materials to be used in execution should include assessment of the following:

- 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 the ground;
- the workability of the ground during execution 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.

(3) <RCM> Information obtained from investigations of rock conditions should be sufficient to determine the following:

- presence of weakness zones, weathered zones, and discontinuities;
- geometrical properties of any weakness zones, weathered zones, or discontinuities;
- physical properties of any weakness zones, weathered zones, or discontinuities;
- the level of the bedrock, rockhead or transition zone between soil and rock;
- the in-situ stress conditions;
- quality, strength, and stiffness properties of the rock mass;
- groundwater according (4).

(4) <RCM> Information obtained from investigations of groundwater conditions should be sufficient to determine the following:

- the depth, thickness, extent and permeability of water-bearing geotechnical unit in the ground, and joint systems in 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 groundwater pressure distribution;
- the chemical composition and temperature of groundwater.

(5) <RCM> The information obtained should be sufficient to assess the following:

- the scope for and nature of groundwater-lowering work;
- possible harmful effects of the groundwater on excavations or on slopes;
- any measures necessary to protect the structure;
- data to enable the design of soakaways and other infiltration devices;
- the effects of groundwater
- to absorb water injected during construction work;
- whether it is possible to use local groundwater, given its chemical constitution, for construction purposes after lowering, desiccation, impounding etc. on the surroundings;
- the water storage capacity of the ground.

NOTE 1. Harmful effects of groundwater include e.g. risk of hydraulic failure, excessive seepage pressure, erosion or dissolution.

NOTE 2. Meassures to protect the structure include e.g. waterproffing, drainage and measures against aggressive water.

Annex E

(informative)

Methods for determining density index and strength properties

E.1 Use of this Informative Annex

(1) This Informative Annex provides supplementary guidance to Clause 7 for evaluation density index of and to Clause 8 for evaluating the strength properties of soils and rock.

NOTE 1. National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

E.2 Scope and field of application

(1) This Informative Annex covers:

- density index;
- angle of peak effective friction;
- peak undrained cohesion;
- Geological Strength Index.

NOTE 1. The relationships in this Annex are provided as examples.

E.3 Density index

(1) <PER> Density index may be determined from the results of field tests using the expressions in Table E.1

<Drafting note: the following "table" should be rewritten into an appropriate format>

Table E.1 — Correlations to determine density index from results of field tests

Field test	Correlation
CPT	$I_D = \frac{1}{C_2} \cdot \ln\left[\frac{q_c}{C_0 \cdot (\sigma'_m)^{C_1}}\right]$
DPT	$I_D = C_1 + C_2 \cdot \log(N)$
SPT	$I_D = \left(\frac{(N_1)_{60}}{C_1}\right)^{0,5}$
PMT	$I_D = C_1 (p_{LM})^{C_2}$
WST	$I_D = C_2 + \left(\frac{N_{WST1}}{C_1}\right)^{0,5}$
BDP	$I_D = C_1 + C_2 \cdot \log(N)$

where:

C_0, C_1 and C_2 are soil constants that depends on the particular test	
q_c is the measured cone resistance in kPa	as per EN ISO 22476-1
σ'_m is the mean effective stress in kPa	
N of blows to drive the penetrometer over a defined distance	for DPT as per EN ISO 22476-2 for BDP as per EN ISO 22476-14
p_{LM} is the pressumeter limit pressure of the ground	as per EN ISO 22476-4
N_{WST1} is the number of half rotations per 1-m penetration	

NOTE The data supporting relationships in Table E1 derive from Baldi et al (1986) for CPT,

E.4 Angle of peak effective friction

E.4.1 From CPT results

(1) <PER> Provided the conditions given in (2) are satisfied, the angle of peak effective friction (φ'_p) may be determined from the results of cone penetration tests (CPTs) using Formula (E. 1):

$$\varphi'_{\mathrm{p}} = min\left(11\log_{10}\left(\frac{q_{\mathrm{t}}}{\sqrt{\sigma'_{\mathrm{v0}}/p_{\mathrm{a}}}}\right) + 17.6°; 45°\right) \quad \text{(E. 1)}$$

where:

q_t is the cone resistance as per EN ISO 22476-1;

σ'_{v0} is the vertical effective stress at the measurement location;

p_a is atmospheric pressure (approximately 100 kPa).

(2) <RCM> Formula (E. 1) should only be used if:

– the fine content of the soil is less than 20 %;
– the soil D_{50} is less than 40 mm;
– the soil mineralogy is consistent mostly of quartz;
– the vertical effective stress σ'_{v0} is less than 1 MPa.

NOTE 1. The standard error associated with Formula (E. 1) is 3.2°.

NOTE 2. The data supporting this relationship derive from triaxial compression tests by Ching et al., 2017.

E.4.2 From SPT results

(1) <PER> Provided the conditions given in (2) are satisfied, the angle of peak effective friction (φ'_p) may be determined from the results of standard penetration tests (SPTs) using Formula (E. 2):

$$\varphi'_{\mathrm{p}} = \min\left(22.3° + 3.5\sqrt{N_{60}/\sqrt{\sigma'_{\mathrm{v0}}/p_{\mathrm{a}}}}, 45°\right) \qquad (E.2)$$

where:

N_{60} is the energy-normalized SPT blow count as per EN ISO 22476-3;

σ'_{v0} is the vertical effective stress at the measurement location;

p_{a} is atmospheric pressure.

(2) <RCM> Formula (E. 2) should only be used if:

- the soil is classified as sand according to EN ISO 14688-2;
- the fines content of the sand is below 15 %;
- the sand mineralogy comprises mostly quartz.

NOTE 1. The standard error associated with Formula (E. 2) is 2.3°.

NOTE 2. The data supporting this relationship derive from triaxial compression tests by Hatanaka and Uchida (1996).

E.4.3 From DMT results

(1) <PER> Provided the conditions given in (2) are satisfied, the angle of peak effective friction (φ'_{p}) may be determined from the results of dynamic penetration tests (DMTs) using Formula (E. 3):

$$\varphi'_{\mathrm{p}} = \min(28° + 14.6\, log_{10} K_{\mathrm{D}} - 2.1(log_{10} K_{\mathrm{D}})^2, 45°) \qquad (E.3)$$

where:

K_{D} is the DMT horizontal stress index as per EN ISO 22476-11.

(2) <RCM> Formula (E. 3) should only be used if the soil is classified as sand according to EN ISO 14688-2.

NOTE 1. Formula (E. 3) is believed to give conservative estimates of φ'_{p}.

E.4.4 From density index

(1) <PER> In the absence of date indicating otherwise, the angle of peak effective friction (φ'_{p}) of coarse soil may be determined from measurements of density index using Formula (E. 4):

$$\varphi'_{\mathrm{p}} = \varphi'_{\mathrm{cs}} + m(I_{\mathrm{D}}[Q - \ln p'] - 1) \qquad (E.4)$$

where:

φ'_{cs} is the critical state angle of friction;

m is a coefficient that depends on the relevant shear mode to failure (m = 5 in plane strain and m = 3 in triaxial compression);

I_D is the density index of the coarse soil (see 7.1.6);

Q is a coefficient that depends on the crushability of the material;

p' is the mean principal effective stress at failure.

NOTE 1 For quartz and feldspar grains, Q = 10. For carbonate grains, Q = 7.

NOTE 2 The critical state angle of friction can be evaluated by testing or inferred from particle size, particle shape, and nature.

NOTE 3 The data supporting this relationship is given by Bolton (1986).

E.5 Peak undrained cohesion

E.5.1 From plasticity and pre-consolidation pressure

(1) <PER> Provided the conditions given in (2) are satisfied, the peak undrained cohesion ($c_{u,p}$) of a clay may be determined from its plasticity index and pre-consolidation pressure using Formula (E. 5):

$$c_{u,p} = (0.11 + 0.0037 \times I_P)\sigma'_p \qquad (E.5)$$

where:

σ'_p pre-consolidation pressure;

I_P is the plasticity index of the clay.

(2) <RCM> Formula (E. 5) should only be used if:

- the soil is classified as clay according to EN ISO 14688-2;
- the soil is not silt-dominated or formed by diatomite;
- the clay organic matter content is below 2 %.

NOTE 1 The bias in the measurement/prediction ratio for Formula (E. 5) is 0.97, with a coefficient of variation of 0.35.

NOTE 2 The data supporting this relationship derive from field vane measurements by D'Ignazio, et al. (2016).

E.5.2 From CPT results

(1) <PER> Provided the conditions given in (2) are satisfied, the peak undrained cohesion of a clay ($c_{u,p}$) may be determined from the results of cone penetration tests (CPTs) using Formula (E. 6):

$$c_{u,p} = \frac{q_n}{N_{kt}} = \frac{q_n}{10.5 - 4.6 \log_e\left(\frac{\Delta u_2}{q_n} + 0.1\right)} \qquad (E.6)$$

where:

q_n is the net cone tip resistance measured as per EN ISO 22476-1 (= $q_c - \sigma_{v0}$);

N_{kt} is a cone factor;

Δu_2 is the excess pore water pressure measured at the gap between cone tip and friction sleeve as per EN ISO 22476-1.

(2) <RCM> Formula (E. 6) should only be used if:

- the soil is classified as clay according to EN ISO 14688-2;
- the clay is saturated when the CPT is performed;
- the clay is of low sensitivity according to EN ISO 14688-2;
- the clay has OCR < 2.5.

NOTE 1 The bias in the measurement/prediction ratio for Formula (E. 6) is 1.09, with a standard deviation of 0.28.

NOTE 2 The data supporting this relationship derive from triaxial compressions tests on samples anisotropically consolidated to the in-situ stress state, as given by Mayne and Peuchen (2018).

E.5.3 From SPT results

(1) <PER> Provided the conditions given in (2) are satisfied, the peak undrained cohesion of a clay ($c_{u,p}$) may be determined from the results of standard penetration tests (SPTs) using Formula (E. 7):

$$c_{u,p} = 7.57 \cdot N_{60} \tag{E. 7}$$

where:

N_{60} is the energy-normalized SPT blow count complying with EN ISO 22476-3.

(2) <RCM> Formula (E. 7) should only be used if:

- the soil is classified as clay according to EN ISO 14688-2;
- the clay must be of low sensitivity according to EN ISO 14688-2;
- the Atterberg limits are such that 20 % < w_L < 110 % and 14 % < w_p < 44 %.

NOTE 1 The standard error associated with Formula (E. 7) is 36 kPa.

NOTE 2 The data supporting this relationship derive from unconsolidated undrained triaxial compression tests results, as given by Sivrikaya and Toğrol (2006).

E.5.4 From Pressuremeter test results

(1) <PER> The peak undrained cohesion (c_{up}) of a clay may be determined from the results of pressuremeter tests (PMTs) using Formula (E. 8):

$$c_{u,p,Y} = \frac{p_{LM} - p_1}{K_{PMT}} \tag{E. 8}$$

where:

p_{LM} is the pressuremeter limit pressure of the ground (EN ISO 22476-4);

p_1 is the corrected pressure at the origin of the pressuremeter modulus pressure range (see EN ISO 22476-4);

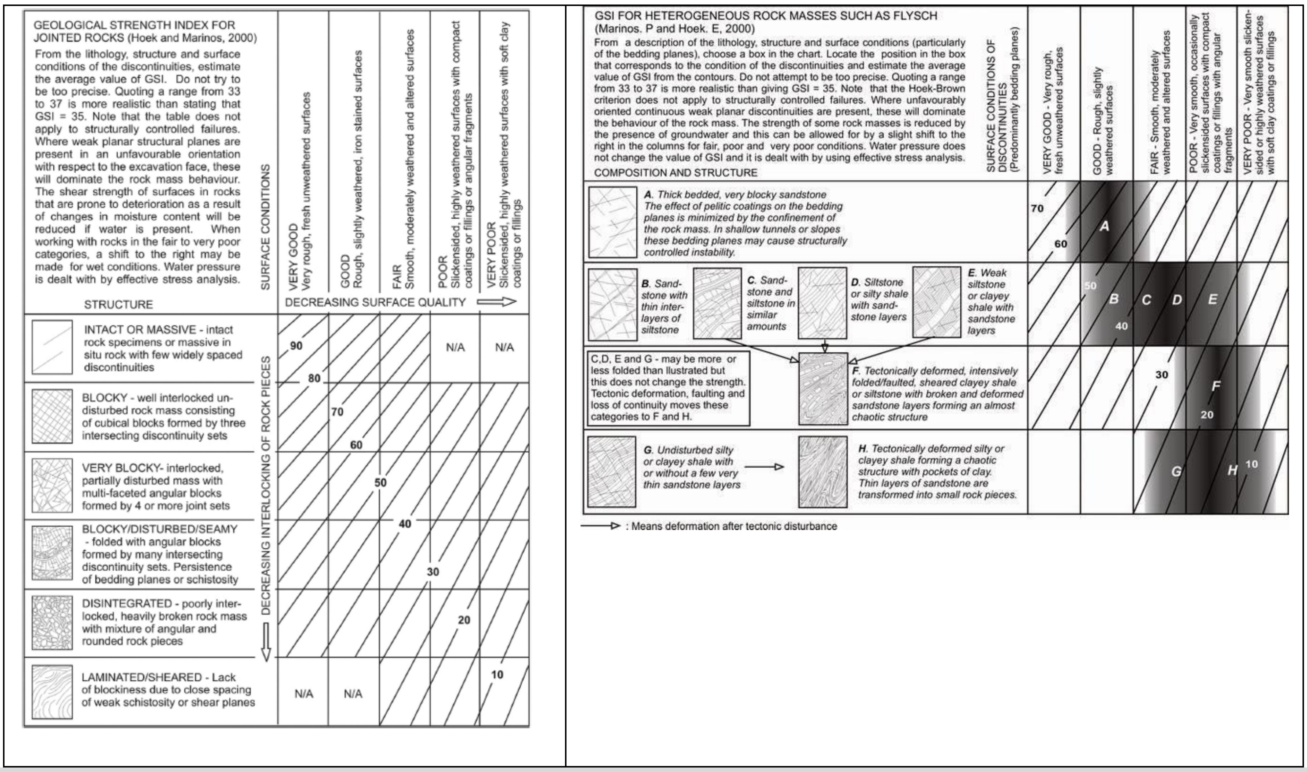
K_{PMT} is a calibration factor.

NOTE 1. The value of K_{PMT} typically ranges from 2 to 20 (with higher values corresponding to stiffer soils), unless the National Annex specifies a narrower range of values for K_{PMT} for specific geological formations.

E.6 Geological Strength Index (GSI)

(1) <PER> An initial estimate of the Geological Strength Index for a rock mass may be obtained following the guidelines contained in Figure E.1.

NOTE The chart given in Figure E.1 is taken from Marinos and Hoek (2000).



<Drafting note: To be replaced with a link>

Figure E.1 – Charts to obtain an initial estimate of Geological Strength Index

Annex F

(informative)

Methods for determining stiffness and consolidation properties of soils

F.1 Use of this Informative Annex

(1) This Informative Annex provides supplementary guidance to Clause 9 for evaluating the stiffness and consolidation properties of soils.

NOTE 1. National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

F.2 Scope and field of application

(1) This Informative Annex covers:

- evaluation of sample disturbance;
- definitions of soil stiffness;
- parameters for empirical models;

F.3 Evaluation of specimen disturbance

NOTE 1. While no definitive method exists for determining the quality of intact samples, valuable information can be obtained using the following qualitative and quantitative methods.

(1) <PER> Qualitative assessment of specimen quality may be made by visual inspection, amplified where appropriate using X-rays or CT scans as described in ISO 19901-8.

(2) <PER> Petrographic examination of soil fabric may be used to assess the amount of disturbance in fine, fragile carbonate soils.

(3) <PER> Quantitative assessment of specimen quality for intact, low to medium overconsolidation ratio clays may be made by measuring volume change at the estimated in-situ stress state during laboratory consolidation.

(4) <RCM> The normalized specimen quality parameter $\Delta e/e_0$ should be computed from Formula (F. 1):

$$\Delta e/e_0 = \varepsilon_{\text{vol}} \cdot (1 + e_0)/e_0 \qquad \text{(F. 1)}$$

where:

Δe is the change in void ratio;

e_0 is the void ratio of the prepared specimen;

ε_{vol} is the volumetric strain (= $\Delta V/V_0$) from reconsolidation to (σ'_{v0}, σ'_{h0});

σ'_{v0} is the in-situ vertical effective stress;

σ'_{h0} is the in-situ horizontal effective stress.

(5) <RCM> The values of $\Delta e/e_0$ and ε_{vol} should be computed and reported for laboratory consolidation tests conducted on intact clay soils, provided the best estimate in-situ effective stresses are given.

NOTE 1. Laboratory consolidation tests conducted on intact clay soils include incremental load oedometer, constant rate of strain and anisotropic consolidation phase of strength tests such as triaxial and direct simple shear.

NOTE 2. The minimum specimen quality classes can be determined using Table F.1.

NOTE 3. Minimum quality classes 4 and 5 correspond to specimens subject to a decrease in effective stress, a reduction in the inter-particle bonds, and a rearrangement of the soil particles. These classes are used for determination of physical and chemical properties according to EN ISO 22475. Specimen quality class is different from sample quality class defined in EN ISO 22475 qualifying the a priori disturbance induced by the sampling technique.

Table F.1 — Evaluation of intact specimen quality for low to medium OCR clays

Quality	$\Delta e/e_0$		Specimen MQC
	OCR = 1 to 2	OCR = 2 to 4	
Very good	< 0,04	< 0,03	1 (small strain)
Good	0,04 to 0,07	0,03 to 0,05	1
Poor	0,07 to 0,14	0,05 to 0,10	2
Very poor	> 0,14	> 0,10	3

NOTE 1. The specimen quality criteria in Table F.1 are not valid for data for load step durations during which secondary compression are observed. For marine soil, a duration below 24 h is commonly used.

(6) <PER> Disturbance of a soil specimen may be determined by comparison of values of the wave propagation velocities determined on lab specimens with respect to the material in its natural state at the site scale.

F.4 Definitions of soil stiffness

(1) <RCM> Values of the modulus of elasticity (G or E) should be determined at strain levels appropriate for the structure.

NOTE 1. Strain levels appropriate for different structures are shown in Figure F.1.

NOTE 2. Figure F.1 shows the measuring ranges of laboratory and in-situ equipment and the strains generated in the vicinity of geotechnical structures during their construction and operation. The current ranges of use are extended on the left to the maximum threshold, which can be reached during very careful tests where the remoulding of the soil is limited.

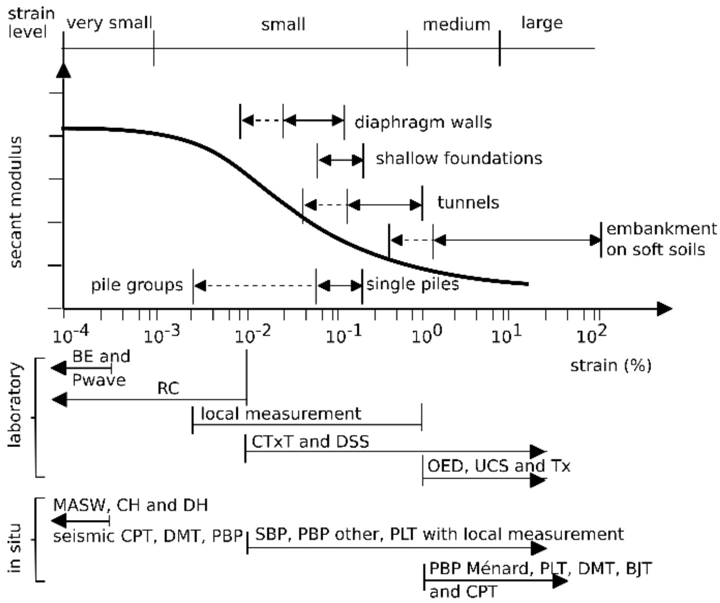


Figure F.1 – Typical strain ranges for common geotechnical constructions and tests (see Atkinson and Sällfors, 1991) To be replaced with the final version from NEN

(2) <RCM> The determination on the experimental curves should be adapted to the range of variation possible for these parameters.

(3) <RCM> The homogeneous deformability of the soil mass should be evaluated judiciously in order to be representative of the average behaviour. It cannot be relevant for all observable behaviours from initial loading to failure.

NOTE 1 The determination of the parameters is thus a compromise between the possibilities of the tests and a satisfactory representation of structures and grounds behaviour. For that, it is necessary to set a suitable data acquisition frequency and to adapt the procedure to get the experimental curves at the proposed rheological model and at the range of possible variation of the parameters.

NOTE 2 As shown on Figure F.1 for triaxial compression test, the use of an E_{sec} secant modulus makes it possible to study the evolution of the stress-strain relationship during the appearance of plastic strains. The secant modulus can be calculated for very small strains where the determination of the tangent E_{tan} modulus becomes problematic because of the increasing resolution that this requires.

NOTE 3 An alternative to determining an initial or secant modulus by tests such as the resonant column is the determination of an E_{cyc} cyclic modulus for low amplitude unloading (i.e. loops). Often, the modulus obtained then is higher than the initial modulus E_0 (obtained on the first part of the curve). This means that the elastic domain exists only for the smaller strains, which the usual test does not achieve.

(4) <RCM> The elastic modulus should be determined from an unloading path.

(5) <RCM> The realization of cycles during laboratory and field tests should be stated when planning the ground investigation.

NOTE 1 The same goes for the moduli and their variation in power of the average pressure. It is preferable to have spread mean pressures to catch this non-linearity.

NOTE 2 For other more sophisticated and non-linear criteria, stakeholders can prescribe to find experimentally the peculiarities of the model, to increase the number of specimens, and to perform tests around, but also at a distance from, singularities.

F.5 Parameters for empirical models

(1) <PER> The secant shear modulus of a soil (G_{sec}) may be estimated from Formula (F. 2F. 2):

$$\frac{G_{\text{sec}}}{G_0} = \left[1 + \left(\frac{\gamma - \gamma_{\text{e}}}{\gamma_{\text{ref}}}\right)^m\right]^{-1} \tag{F. 2}$$

where:

G_0 is the soil's very-small-strain shear modulus;

γ is the shear strain in the soil;

γ_e is the elastic threshold strain beyond which shear modulus falls below its maximum value;

γ_{ref} is a reference value of engineering shear strain (at which G_{sec}/G_0 = 0.5); and

m is a coefficient that depends on soil type.

NOTE 1. A database supporting this relationship is given by Oztoprak and Bolton (2013).

(2) <PER> Formula (F. 2) may also be used to validate direct or indirect measurements of stiffness.

(3) <PER> Values of the parameters for use with Formula (F. 2) may be taken from Table F.2.

(4) <PER> Values other than those given in Table F.2 may be used with Formula (F. 2) provided the testing, reporting, and interpretation procedures comply with to the general prescriptions given in 9.

Table F.2 — Values of parameters for use with Formula (F. 2)

Soil type as per 14688-1	Parameters			Reference
	γ_{ref}	m	γ_e	
Sand	0,02-0,1	0,88	0,02% + 0,012 γ_{ref}	Oztoprak and Bolton (2013)
Clay and silt	0,0022 I_p	0,735 ±0,122	0	Vardanega and Bolton (2013)
Note: Ip expressed as a value, not as a percentage				

(5) <PER> The very-small-strain shear modulus of a soil (G_0) may be determined from Formula (F. 3):

$$\frac{G_0}{p_{\text{ref}}} = \frac{k_1}{(1+\text{e})^{k_2}}\left[\frac{p'}{p_{ref}}\right]^{k_3} \tag{F. 3}$$

where:

e is the void ratio;

p' is the mean effective stress in the soil;

p_{ref} is a reference pressure;

k_1, k_2, and k_3 are coefficients that depend on soil type.

NOTE 1. See Bolton et al. (2000) and Clayton et al. (2011) for further information.

(6) <PER> Formula (F. 3) may also be used to validate direct or indirect measurements of very-small strain stiffness.

(7) <PER> Values of the parameters for use with Formula (F. 3) may be taken from 9.

(8) <PER> Values other than those given in Table F.2 may be used with Formula (F. 3) provided the testing, reporting, and interpretation procedures comply with 9.

Table F.3 — Values of parameters for use with Formula (F. 3)

Soil type as per 14688-1	Parameters				Reference
	k_1	k_2	k_3	p_{ref} (kPa)	
Fine grained soil	2100	0	0,6-0,8	1	Viggiani and Atkinson (1995)
Sand[a]	370-5760	3	0,49-0,86	100	Oztoprak and Bolton (2013)
Clay and silt	20000 ±5000	2,4	0,5	1	Vardanega and Bolton (2013)
[a]decreasing with strain					

(9) <PER> The rock mass modulus (E_{rm}) may be estimated from empirical models, such as Formula (F. 4):

$$\frac{E_{\mathrm{rm}}}{E_{\mathrm{i}}} = 0{,}02 + \frac{1 - D/2}{1 + e^{[(60+15 \cdot D - GSI)/11]}} \quad \text{(F. 4)}$$

where:

E_i is the Young's modulus intact rock;

GSI is the Geological Strength Index, with a value between 0 and 100;

D is the disturbance factor, with a value between 0 and 1.

NOTE 1. See Hoek and Brown (2018) for further information.

(10) <PER> Young's modulus of intact rock (E_i) may be determined directly from test results (see 9.1.2) or estimated indirectly from Formula (F. 5):

$$E_i = MR\sigma_{\mathrm{ci}} \quad \text{(F. 5)}$$

where:

MR is the rock's modulus ratio;

σ_{ci} is the is the uniaxial compressive strength of intact rock (see 8.3.1).

Annex G

(informative)

Indirect methods for determining cyclic, dynamic, and seismic properties of soils

G.1 Use of this Informative Annex

(1) This Informative Annex provides supplementary guidance to Clause 10 for evaluating the mechanical response of soils and rocks to dynamic actions and parameters for seismic design.

NOTE 1. National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

G.2 Scope and field of application

(1) This Informative Annex covers:

- indirect methods for the evaluation of normalised secant shear modulus and damping ratio curves;
- indirect methods for the evaluation of shear wave velocity (v_S).

G.3 Indirect methods for the evaluation of normalised secant shear moduli and damping ratio curves

G.3.1 Fine soils

(1) <PER> The secant shear modulus G_{sec} of fine soils may be determined as a function of cyclic shear strain from Formulae (G. 1)-(G. 2):

$$\frac{G_{\mathrm{sec}}}{G_0} = \left[1 + \left(\frac{\gamma_{\mathrm{cyc}}}{\gamma_{\mathrm{ref}}}\right)^{\alpha}\right]^{-1} \tag{G. 1}$$

$$\gamma_{\mathrm{ref}}(\%) = \left(\phi_1 + \phi_2 \times I_{\mathrm{P}} \times OCR^{\phi_3}\right) \times \left(\frac{\sigma'_0}{p_{\mathrm{a}}}\right)^{\phi_4} \tag{G. 2}$$

where:

G_0	is the very small-strain shear modulus of the soil;
γ_{cyc}	is the cyclic shear strain;
γ_{ref}	is a reference value of engineering shear strain (at which G_{sec}/G_0 = 0.5)
α	is a curvature coefficient, given in Table I.1 as ϕ_5;
I_P	is the plasticity index;

OCR is the overconsolidation ratio;

σ'_0 is the mean effective stress;

$\phi_1, \phi_2, \phi_3, \phi_4$ are constants given in Table G.1.

NOTE 1. Formulae (G. 1)-(G. 2) were originally proposed by Darendeli (2001).

(2) <PER> The shear damping ratio D of fine soils may be determined as a function of cyclic shear strain from Formulae (G. 3)-(G. 5):

$$D = D_0 + f\left(G\left(\gamma_{\text{cyc}}\right)/G_0\right) \tag{G. 3}$$

$$D_0 = \left(\phi_6 + \phi_7 \times I_{\text{P}} \times OCR^{\phi_8}\right) \times \sigma_0'^{\phi_9} \times [1 + \phi_{10}\, ln(f)] \tag{G. 4}$$

$$f\left(\frac{G_{\text{sec}}}{G_0}\right) = b \times D_M \times \left(\frac{G_{\text{sec}}}{G_0}\right)^{0,1} = b \times \left[c_1\left(D_{\text{M},\alpha=1}\right) + c_2\left(D_{\text{M},\alpha=1}\right)^2 + c_3\left(D_{\text{M},\alpha=1}\right)^3\right] \times \left(\frac{G_{\text{sec}}}{G_0}\right)^{0,1} \tag{G. 5}$$

where:

b is given by $b = \phi_{11} + \phi_{12}\, ln\, N$

D_0 is the small strain damping ratio;

$D_{\text{M},\alpha=1}$ is given by

$$D_{\text{M},\alpha=1} = \frac{100}{\pi}\left[4\frac{\gamma_{\text{cyc}} - \gamma_{\text{ref}} \ln\left(\dfrac{\gamma_{\text{cyc}} + \gamma_{\text{r}}}{\gamma_{\text{r}}}\right)}{\dfrac{\gamma_{cyc}^2}{\gamma_{\text{cyc}} + \gamma_{\text{r}}}} - 2\right]$$

c_1 is given by $c_1 = 0.2523 + 1.8618\alpha - 1.1143\alpha^2$

c_2 is given by $c_2 = -0.0095 - 0.0710\alpha + 0.0805\alpha^2$

c_3 is given by $c_3 = 0.0003 + 0.0002\alpha - 0.0005\alpha^2$

N is the number of cycles (default value 10)

I_{P} is the plasticity index;

OCR is the overconsolidation ratio;

σ'_0 is the mean effective stress;

f is the frequency of the load in Hz (default value: 1 Hz);

$\phi_6, \phi_7, \phi_8, \phi_9, \phi_{10}$ are constants given in Table G.1.

NOTE 1. Formulae (G. 3)-(G. 5) were originally proposed by Darendeli (2001).

Table G.1 – Constants for the evaluation of normalised shear modulus and damping ratio of fine soils

Parameter	Value	Parameter	Value	Parameter	Value	Parameter	Value
ϕ_1	0.0352	ϕ_5	0.9190	ϕ_9	−0.2889	ϕ_{13}	−4.23
ϕ_2	0.0010	ϕ_6	0.8005	ϕ_{10}	0.2919	ϕ_{14}	3.62
ϕ_3	0.3246	ϕ_7	0.0129	ϕ_{11}	0.6329	ϕ_{15}	−5.00
ϕ_4	0.3483	ϕ_8	−0.1069	ϕ_{12}	-0.0057	ϕ_{16}	−0.25

(3) <PER> The variability of the normalised shear modulus may be estimated assuming a normal distribution and a value of variance σ_{NG} given by Formula (G. 6):

$$\sigma_{\mathrm{NG}} = e^{\phi_{13}} + \sqrt{\frac{0.25}{e^{\phi_{14}}} - \frac{([G_{\mathrm{sec}}/G_0]_{mean} - 0.5)^2}{e^{\phi_{14}}}} \tag{G. 6}$$

where:

$[G_{\mathrm{sec}}/G_0]_{\mathrm{mean}}$ is given by Formula I.1; and

ϕ_{13}, ϕ_{14} are constants given in Table G.1.

NOTE 2. Formula (G. 6) was originally proposed by Darendeli (2001).

(4) <PER> The variability of the shear damping ratio may be determined assuming a normal distribution and a value of the variance σ_{D} from Formula (G. 7):

$$\sigma_{\mathrm{D}} = e^{\phi_{15}} + e^{\phi_{16}}\sqrt{(D)_{\mathrm{mean}}} \tag{G. 7}$$

where:

$(D)_{\mathrm{mean}}$ is given by Formula I.2;

ϕ_{15}, ϕ_{16} are constants given in Table G.1.

NOTE 1. Formula (G. 7) was originally proposed by Darendeli (2001).

G.3.2 Coarse soils

(1) <PER> Provided the conditions given in (2) are satisfied, the secant shear modulus G_{sec} for coarse soils may be determined as a function of cyclic shear strain from Formulae (G. 8)-(G. 9):

$$\frac{G_{\mathrm{sec}}}{G_0} = \left[1 + \left(\frac{\gamma_{\mathrm{cyc}}}{\gamma_{\mathrm{ref}}}\right)^{\alpha}\right]^{-1} \tag{G. 8}$$

$$\gamma_{\mathrm{ref}}(\%) = 0.12 c_{\mathrm{U,PSD}}^{-0.6} \times \left(\frac{\sigma_0'}{p_{\mathrm{a}}}\right)^{0.5 c_{\mathrm{u}}^{-0.15}} \tag{G. 9}$$

where:

G_0 is the soil small-strain shear modulus;

γ_{cyc} is the cyclic shear strain;

γ_{ref} is a reference value of engineering shear strain (at which $G_{sec}/G_{max} = 0.5$);

α is the curvature coefficient given by $\alpha = 0.86 + 0.1 \log\left(\frac{\sigma_0'}{p_a}\right)$;

$C_{U,PSD}$ is the coefficient of uniformity;

p_a is the atmospheric pressure;

σ'_0 is the mean effective stress;

NOTE 1. Formulae (G. 8)-(G. 9) were originally proposed by Menq (2003).

(2) <PER> The shear damping ratio D of coarse soils can be determined as a function of cyclic shear strain from Formula (G. 9):

$$D = D_0 + b \times D_M \times \left(\frac{G_{sec}}{G_0}\right)^{0,1} = \left[0.55 c_{U,PSD}^{0.1} \times D_{50}^{-0.3} \times \left(\frac{\sigma_0'}{p_a}\right)^{-0.05}\right] + \left[b \times D_M \times \left(\frac{G_{sec}}{G_0}\right)^{0,1}\right] \quad (G.9)$$

where:

D_0 is the small strain damping;

$C_{U,PSD}$ is the coefficient of uniformity;

D_{50} Is the median grain size

p_a is the atmospheric pressure;

σ'_0 is the mean effective stress;

b is given by: $b = 0.6329 - 0.0057 \ln N$

D_M is given by: $D_M = c_1(D_{M,\alpha=1}) + c_2(D_{M,\alpha=1})^2 + c_3(D_{M,\alpha=1})^3$

$D_{M,\alpha=1}$ is given by: $D_{M,\alpha=1} = \frac{100}{\pi}\left[4\frac{\gamma_{cyc} - \gamma_r \ln\left(\frac{\gamma_{cyc}+\gamma_{ref}}{\gamma_{ref}}\right)}{\frac{\gamma_{cyc}^2}{\gamma_{cyc}+\gamma_{ref}}} - 2\right]$

c_1 is given by: $c_1 = 0.2523 + 1.8618\alpha - 1.1143\alpha^2$

c_2 is given by: $c_2 = -0.0095 - 0.0710\alpha + 0.0805\alpha^2$

c_3 is given by: $c_3 = 0.0003 + 0.0002\alpha - 0.0005\alpha^2$

N number of cycles (default value 10),

NOTE 1. The model is reliable for dry soils and shear strains ranging between 0.0001 % and 0.6 %.

NOTE 2. Formula (G. 9) was originally proposed by Menq (2003).

G.4 Indirect methods for the evaluation of shear wave velocity or very small strain shear modulus

G.4.1 From Standard Penetration Tests

(1) <PER> The shear wave velocity v_S of sands may be determined from the results of Standard Penetration Tests using Formula (G.10):

$$v_S = k_{vs} N_{60}^{0.23} \sigma_v'^{0.25} \tag{G.10}$$

where:

k_{vs} is a constant equal to 27 for Holocene sands and 35 for Pleistocene sands;

N_{60} is the blow counts of a standard penetration test for energy efficiency of 60% [blows/30cm];

σ_v' is the vertical effective stress.

NOTE 1. Formula (G.10) was originally proposed by Wair et al. (2012).

G.4.2 From Cone Penetration Tests

(1) <PER> The shear wave velocity v_S of Pleistocene sands may be determined from the results of Cone Penetration Tests using Formula (G.11):

$$v_S = \sqrt{10^{(0.55 I_C + 1.68)} \frac{q_t - \sigma_v'}{p_a}} \tag{G.11}$$

where:

I_c is the soil behaviour type index;

p_a is the atmospheric pressure;

q_t is the corrected cone resistance

σ'_v is the vertical effective stress.

NOTE 1. Formula (G.11) was originally proposed by Robertson (2009).

G.4.3 From Flat Dilatometer Tests

(1) <PER> The small-strain shear modulus G_0 may be determined from the results of Flat Dilatometer Tests using Formula (G.12):

$$G_0 = k_1 K_D^{-k_2} M_{DMT} \tag{G.12}$$

where:

M_{DMT} is the dilatometer modulus;

K_D is the horizontal stress index;

I_D is the material index.

k_1 is a constant equal to: 27.177 for $I_D < 0.6$; 15.686 for $0.6 \leq I_D < 1.8$; and 4.5613 for $0.8 \leq I_D$.

k_2 is a constant equal to: 1.0066 for $I_D < 0.6$; 0.921 for $0.6 \leq I_D < 1.8$; and 0.7957 for $0.8 \leq I_D$.

NOTE 1. Formula (G. 12) was originally proposed by Monaco et al. (2009).

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