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

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Eurocode 7: Geotechnical design — Part 2: Ground investigation

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

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

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Drafting foreword by PT3

This document (prEN 1997-2:20xx) has been prepared by project team M515/SC7.T3.

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

This document is a working document.

Verbal forms are signified thus:

<REQ> signifies a requirement (verb form 'shall')

<RCM> signifies a recommendation (verb form 'should')

<PER> signifies permission (verb form 'may')

<POS> signifies a possibility (verb form 'can')

For ease of preparation of this draft, NOTES are numbered 1 … 100+ in this document. They will be re-numbered for the final draft in accordance with CEN's regulations (IR3) and TC250's drafting rules (N1250). There is therefore no need to comment on the numbering of NOTES in 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.

Introduction

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>

Introduction to EN 1997

[Drafting note: This contains information formerly included in “Scope of EN 1997”]

EN 1997 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.

EN 1997 establishes additional principles and requirements for the safety, serviceability, robustness, and durability of geotechnical structures.

EN 1997 is intended to be used in conjunction with the other Eurocodes for the design of geotechnical structures, including temporary geotechnical structures.

EN 1997 establishes rules for the calculation of geotechnical actions.

Design and verification in EN 1997 are based on the partial factor method, prescriptive measures, testing, or the observational method.

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.

National annex for EN 1997-2

This standard gives values within notes indicating where national choices can be made. Therefore, the national standard implementing EN 1997-2 can have a National Annex containing all Nationally Determined Parameters to be used for the design of building and civil engineering works to be constructed in the relevant country.

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

[Drafting note: list of clauses to be compiled by the Project Team for the final version]

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

[Drafting note: list of annexes to be compiled by the Project Team for the final version]

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 rules for specifying ground investigation used to gather information needed for the design and verification of geotechnical structures.

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

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

1.2 Assumptions

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

- <to be add later, if any>

2 Normative references

<to be completed later>

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

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

EN ISO 16907-1, Earthworks – Part 1: Principles and general rules.

ISO 1707-1: 2017, Buildings and civil engineering works — Vocabulary, Part 1: General terms.

3 Terms, definitions and symbols,

3.1 Terms and definitions

For purposes of this document, the following terms and definitions apply.

[Drafting note: CEN rules require the following sentence to be included]

ISO and IEC maintain terminological databases for use in standardization at the following addresses:

- IEC Electropedia: available at www.electropedia.org
- ISO Online browsing platform: available at www.iso.org/obp

3.1.1 Common terms used in EN 1997-2

3.1.1.1 site

area of land or water where construction work or other development is undertaken

NOTE 1. [to entry] the zone of influence can extend beyond the site.

[SOURCE: ISO 6707-1: 2017, 3.1.1.6]

[Drafting note: CEN rules require NOTES in sub-clause 3 to be labelled "NOTE to entry" – please do not comment!]

3.1.1.2 site inspection
observation and recording of features relevant to the surface conditions and any exposures of the ground

NOTE 2. [to entry] the inspection can extend beyond the site boundaries

3.1.1.3 water content
<add definition>, as shown in Figure 3.1

3.1.1.4 Atterberg limits (of soil)
collective name for liquid, plastic, and shrinkage limits of soil, see Figure 3.1

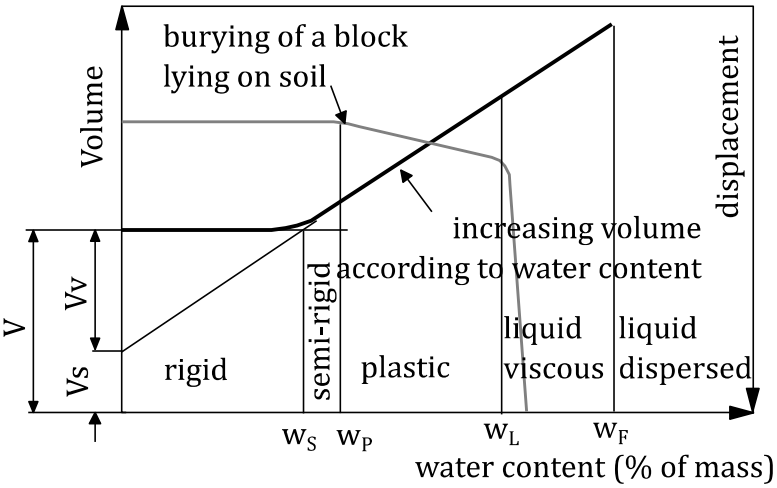

Figure 3.1 – Definition of Atterberg limits

3.1.1.5 shrinkage limit (of soil)
<add definition>

3.1.1.6 plastic limit (of soil)
water content 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: 2018, 3.9]

3.1.1.7 liquid limit (of soil)
water content 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: 2018, 3.6]

3.1.1.8 Sample Quality Class 1

samples that are effectively undisturbed such that the water content, consistency, void ratio, structure (bedding and discontinuities), texture, and in situ stresses are essentially unchanged

3.1.1.9 Sample Quality Class 2

samples that are undisturbed such that the water content, consistency, structure (bedding and discontinuities), and texture are essentially unchanged

3.1.1.10 Sample Quality Class 3

samples that are disturbed but with all the constituents of the in-situ soil or rock in their original proportions and retaining the natural water content; the general arrangement of the different layers or components is disturbed but can be identified at a fine scale (approximately 0.1 m)

3.1.1.11 Sample Quality Class 4

samples that are disturbed but with all the constituents of the in-situ soil or rock in their original proportions; the general arrangement of the different layers or components can only be identified at a coarse scale (approximately 0.25 m).

3.1.1.12 Sample Quality Class 5

samples that are very disturbed with the initial properties changed by the sampling including changes in constituent, chemistry and possible crushing of material; the general arrangement of the different layers or components can only be identified at a coarse scale (approximately 0.5 m)

3.1.1.13 pore-water pressure

pressure of the water that saturates the pores of a porous medium (e.g. soil or rock)

3.1.2 Terms relating to planning of ground investigation

3.1.2.1
ground investigation

use of non-penetrative and penetrative methods to investigate the ground conditions beneath the site

3.1.2.2 ground investigation point

location on the site where the ground is examined and investigated by intrusive methods

3.1.2.3 anthropogenic ground

soil placed by human activity and which can comprise reworked natural soils and synthetic materials

3.1.3 Terms relating to the Ground Model

<none>

3.1.4 Terms relating to ground investigations: type and extent

<none>

3.1.5 Terms relating to physical and chemical properties

3.1.5.1 density index

ratio of the difference between the maximum index void ratio and the field void ratio to the difference between its maximum and minimum index void ratios

3.1.5.2 relative density

synonym for 'density index'

3.1.5.3 activity index

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

3.1.5.4 sphericity (of a particle)

difference between the shape of an object and a mathematically perfect sphere

3.1.5.5 roundness (of a particle)

difference between the shape of an object and a mathematically perfect circle

3.1.6 Terms relating to strength properties

3.1.6.1 strength envelope

expression that identifies stress combinations that produce material failure

3.1.6.2 strength parameters

material parameters appearing on strength envelopes

3.1.6.3 peak strength

upper limit of stress conditions observed in a test

3.1.6.4 constant volume strength

stress conditions observed when shearing at constant volume and constant pore pressure

3.1.6.5 residual strength

steady stress conditions observed after prolonged shearing

3.1.6.6 undrained strength

strength envelope for water saturated soils defined in terms of total stress

3.1.6.7 drained strength

strength envelope defined in terms of effective stress

3.1.7 Terms relating to stiffness properties

3.1.7.1 stiffness modulus

ratio between stress and strain relevant for a test usually corresponding to the slope of the stress-strain relation

3.1.7.2 Young's modulus

(for an isotropic or non-linear elastic material) ratio of the variation of a principal stress by the linear strain obtained in the same direction, with the other principal stresses remaining unchanged

3.1.7.3 oedometer modulus

(for an isotropic or non-linear elastic material) ratio of the variation of a principal stress by the linear strain obtained in the same direction, with the other principal strains remaining unchanged

3.1.7.4 secant modulus

ratio between stress and strains accumulated from an initial reference state, as defined by Figure 3.2

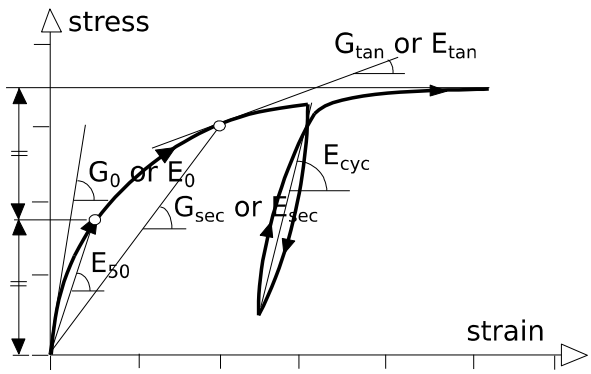

Figure 3.2 — Definition of modulus on load-deformation curve

NOTE 1. (to entry) Software implementing the finite element method sometimes makes use of E_{50} which is defined as a secant modulus at 50% of the failure stress

3.1.7.5 shear modulus

(for isotropic, linear elastic materials) ratio of shear stress to the corresponding shear strain, with other stresses remaining constant

NOTE 2. (to entry) G can be defined at small strain (G_0) or as a secant (G_{sec}) or tangent (G_{tan}) modulus as shown in Figure 3.2.

3.1.7.6 bulk modulus

ratio of pressure increase to the resulting relative volume decrease of the material

3.1.7.7 tangent modulus

ratio between small increments of stress and strain from a given reference state as shown in Figure 3.2.

3.1.7.8 cyclic modulus

slope of the line connecting the two points of reversal of the direction of deformation of the loop in cyclic loading

(alternatively) defined using the envelope of unloading/reloading phases corresponding to the load scenario investigated

3.1.7.9 stiffness

observed rigidity of the ground generally dependent on initial state and loading conditions.

NOTE 3. (to entry) Relationships and correlations are available in literature, for example, to relate unidimensional (1D) stiffness obtained during compressibility tests and tridimensional (3D) stiffness as derived from triaxial.

NOTE 4. (to entry) See Annex B for <add>.

3.1.7.10 Poisson ratio

(for a linear elastic isotropic material) absolute value of the ratio between the two linear deformations respectively perpendicular and parallel to the direction of a uniaxial stress

3.1.7.11 transformation model

Definition given in clause 10 – please add definition here!

3.1.7.12 correlation

synonym for 'transformation model'

3.1.8 Terms relating to mechanical response to dynamic loads

3.1.8.1 compressional wave velocity v_P

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

NOTE 5. (to entry) For a linear elastic homogenous isotropic medium, the compressional wave velocity is a function of the elastic oedometric modulus and mass per unit volume (density)

3.1.8.2 cyclic loading

loading variable in time with a regular repetition

NOTE 6. (to entry) Cyclic loading can be a special case of dynamic loading if the frequency is such that inertia forces cannot be ignored (see 3.1.8.9).

3.1.8.3 cyclic liquefaction

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

3.1.8.4 cyclic shear strength

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

3.1.8.5 cyclic strain

maximum strain attained or imposed during the application of cyclic loads

3.1.8.6 cyclic stress

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

3.1.8.7 damping ratio

quantification of energy loss in a cyclically loaded system based on hysteresis loops of stress vs strain

3.1.8.8 degradation

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

3.1.8.9 dynamic loading

loading that causes significant acceleration for which the role of inertia forces cannot be ignored

3.1.8.10 fundamental frequency

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

3.1.8.11 post-cyclic strength

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

3.1.8.12 post-cyclic creep

deformation associated to average constant loads after the application of a given number of load cycles

3.1.8.13 seismic bedrock

reference formation below a soil deposit characterised by a shear wave velocity larger than 800 m/s

3.1.8.14 shear wave velocity

velocity of propagation of a shear (secondary) wave in a medium

NOTE 7. (to entry) For a linear elastic homogenous isotropic medium, the shear wave velocity is a function of the elastic shear modulus and mass per unit volume (density)

3.1.8.15 small strain elastic modulus

value of the secant elastic modulus at very small strains

3.1.8.16 correlation

synonym for 'transformation model'

3.1.8.17 transformation model

relationship for the indirect estimation of a given parameter as a function of other parameters (typically derived from empirical observation of correlations between different parameters)

NOTE 8. (to entry) also known as 'correlation'

3.1.9 Terms relating to groundwater and hydraulic conductivity

3.1.9.1 piezometric pressure

static pore pressure measured at a point beneath the ground surface

3.1.9.2 piezometric head

height of water column above a point beneath the ground surface, also known as pressure head

3.1.9.3 piezometric level

sum of piezometric head and position head (elevation)

3.1.9.4 piezometer

instrument for measurement of piezometric head, including open and closed systems.

[SOURCE: EN ISO 18674-4, 20xx, x.x.x.x]

3.1.9.5 open system

measuring system in which the groundwater is in direct contact with the atmosphere and in which the piezometer head/piezometer pressure at the filter level is measured

3.1.9.6 closed system

measuring system in which the groundwater is not in direct contact with atmosphere and in which the pore pressure at the filter level is measured hydraulically, pneumatically or electrically

3.1.9.7 hydraulic conductivity

ratio of velocity to hydraulic gradient indicating permeability of porous media and depends on the intrinsic permeability of the material, the degree of saturation, and on the density and viscosity of the water.

NOTE 9. (to entry) Hydraulic conductivity also known as 'permeability' or 'coefficient of permeability'

3.1.9.8 aquifer

body of permeable rock or soil that can contain or transmit groundwater

3.1.9.9 aquiclude

geologic formation or geotechnical unit that confines groundwater in an adjacent aquifer

3.1.9.10 aquitard

geologic formation or stratum that lies adjacent to an aquifer and allows only a small amount of groundwater to pass

3.1.10 Terms relating to thermal properties

3.1.10.1 thermal conductivity

ability of a material to transport thermal energy

3.1.10.2 heat capacity

capacity of a material to store thermal energy

3.1.10.3 thermal diffusivity

ability of a material to level temperature differences

3.1.10.4 latent heat

heat required to convert a solid into a liquid or vapor, or a liquid into a vapor, without change of temperature

3.1.11 Terms relating to reporting

<None>

3.2 Symbols and abbreviations

3.2.1 Common symbols and abbreviations

c	effective cohesion
c_{peak}	peak effective cohesion
c_{res}	residual effective cohesion
c_u	undrained shear strength
$c_{u,cv}$	undrained shear strength under constant volume conditions
$c_{u,peak}$	peak undrained shear strength
$c_{u,res}$	residual undrained shear strength
CPT	Cone Penetration Test
CPTU	Cone Penetration Test with pore-water pressure measurement (piezocone test)
DMT	Flat Dilatometer Test (also known as Marchetti Dilatometer Test)

e	void ratio of soil
e_{max}	maximum void ratio of soil
e_{min}	minimum void ratio of soil
E	Young's modulus
I_D	density index of coarse soil, given by: $I_D = \frac{e_{max} - e}{e_{max} - e_{min}}$
I_P	plasticity index of fine soil (see EN ISO 17892-12)
FVT	Field Vane Test
G	shear modulus, given by: $\boldsymbol{G} = \frac{\boldsymbol{E}}{\boldsymbol{2(1+\nu)}}$
K	bulk modulus, given by: $K = \frac{E}{3(1-2\nu)}$
MPM	Ménard pressuremeter
PLT	Plate Loading Test
PMT	Pressuremeter Test
SPT	Standard Penetration Test
w	water content
w_L	liquid limit of soil
w_P	plastic limit of soil
w_S	shrinkage limit of soil
ν	Poisson's ratio
φ	angle of internal friction of soil
φ_{cv}	angle of internal friction of soil under constant volume conditions
φ_{peak}	peak angle of internal friction of soil
φ_{res}	residual angle of interface friction

3.2.2 Symbols and abbreviations relating to planning of ground investigation

<None>

3.2.3 Symbols and abbreviations relating to the Ground Model

<None>

3.2.4 Symbols and abbreviations relating to ground investigations: type and extent

<None>

3.2.5 Symbols and abbreviations relating to physical and chemical properties

D_n particle size that n % by weight are smaller than

D_{10} particle size that 10 % by weight are smaller than

D_{30} particle size that 30 % by weight are smaller than

D_{60} particle size that 60 % by weight are smaller than

I_A activity index, given by: $I_A = \frac{I_P}{\% < 2\mu m}$

R roundness of a particle

S sphericity of a particle

3.2.6 Symbols and abbreviations relating to strength properties

$c_{u,peak,CAUC}$ peak deviator stress obtained by undrained shearing in triaxial compression after anisotropic consolidation to the "in situ" stress that, following EN ISO 17892-9

$c_{u,peak,FV}$ peak undrained strength measured in a FVT

$c_{u,UU}$ peak undrained strength obtained from UU triaxial tests as per EN ISO 17892-8

$c_{u,X}$ peak undrained strength directly measured by a specific test X = UCS, UU, TXCU, FVT, etc.

dx horizontal displacement of a specimen

dz vertical displacement of a specimen

JCS joint wall compressive strength of rock

JRC joint roughness coefficient of rock

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

K_{PMT} calibration factor for PMT test results

m coefficient that depends on the relevant shear mode to failure

m_b	non-dimensional material parameter for Hoek-Brown envelope
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
p'	mean principal effective stress at failure
p_a	atmospheric air pressure
p_{LM}	Ménard pressuremeter limit pressure of the ground as per EN ISO 22476-4
p_1	corrected pressure at the origin of the pressuremeter modulus pressure range (see EN ISO 22476-4)
Q	coefficient that depends on the crushability of the material
q_c	cone tip resistance measured as per EN ISO 22476-1
q_n	net cone resistance (= $q_c - \sigma_{v0}$)
q_t	corrected cone resistance as per EN ISO 22476-1
s	non-dimensional material parameter for Hoek-Brown envelope
t_f	time to failure relevant to a reference strength value
α	non-dimensional material parameters for Hoek-Brown envelope
Δu_2	excess pore pressure measured at the gap between cone tip and friction sleeve as per EN ISO 22476-1
σ	normal stress on the failure plane
σ_{ci}	uniaxial compressive strength of intact rock
σ_n	normal stress acting on a rock joint
σ'_p	preconsolidation pressure
σ_t	tensile strength of intact rock
σ_v	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 on the failure plane

φ'_{ds} effective friction angle measured in direct shear

φ'_{ps} effective friction angle under plane stress [strain?] conditions

φ_r is the joint residual friction angle of rock

ψ_{ds} angle of dilatancy

3.2.7 Symbols and abbreviations relating to stiffness properties

E_{oed} oedometer modulus, given by: $E_{oed} = \frac{(1-\nu)E}{(1+\nu)(1-2\nu)}$

E_s stiffness modulus

E_{sec} secant modulus

E_{tan} tangent modulus, given by: $E_{tan} = \delta\sigma / \delta\varepsilon$

E_{cyc} cyclic modulus

G_0 shear modulus at small strain

$G_{0,RC}$ shear modulus at small strain obtained by resonant column test

$G_{0,BE}$ shear modulus at small strain obtained by bender elements test

$G_{0,VP}$ shear modulus at small strain obtained by wave propagation: DH, UH, CH

$G_{0,FT}$ shear modulus at small strain obtained in parallel with field test: DMT, CPT, PMT

OED oedometer test

TX triaxial test

DSS direct simple shear test

UCS unconfined compression test

BE bender element

RC resonant column

PLT plate loading test

BJT borehole jack test

FDP full displacement pressuremeter

SBP self-boring pressuremeter

3.2.8 Symbols and abbreviations relating to mechanical response to dynamic loads

v_P compressional wave velocity

v_S	shear wave velocity
H_{800}	depth of the bedrock formation identified by a shear wave velocity v_s greater than 800 m/s
$V_{S,H800}$	equivalent value of the shear wave velocity of the soil column above the depth of the bedrock formation
f_0	natural frequency of a soil deposit
BE	bender element test
CSST	cyclic (direct) simple shear test
CTST	cyclic torsional shear test
CTxT	cyclic triaxial test
RC	resonant column test
CHT	cross-hole test
DHT	down-hole test
HVSR	horizontal-to-vertical spectral ratio survey
P-S	<to be added>
P-S Log	P-S suspension logging test
Refl	Seismic Reflection Survey
Refr	Seismic Refraction Survey
SCPT	Seismic Cone Penetration Test
SDMT	Seismic Flat Dilatometer Test
SWM	surface wave methods
SASW	spectral analysis of surface waves
MASW	multi-station analysis of surface waves
CSSW	continuous source surface waves
AVA	ambient vibration analysis
	Symbols from associated Annex not yet added

3.2.9 Symbols and abbreviations relating to groundwater and hydraulic conductivity

<None>

3.2.10 Symbols and abbreviations relating to thermal properties

λ thermal conductivity (W/(m*K))

κ thermal diffusivity (m^2/s)

C thermal capacity (J/(m^3*K))

c thermal capacity (J/(kg*K))

L latent heat (J/kg)

λ thermal conductivity (W/(m*K))

κ thermal diffusivity (m^2/s)

C thermal capacity (J/(m^3*K))

c thermal capacity (J/(kg*K))

L latent heat (J/kg)

3.2.11 Symbols and abbreviations relating to reporting

GIR Ground Investigation Report

4 Planning of ground investigation

4.1 General

(1) <REQ> Ground investigations shall be planned so that the necessary geotechnical information in all the geotechnical units influencing the anticipated design situations is collected.

NOTE 1. Guidance on suitable ground investigation and test methods is given in Annex A.

(2) <RCM> Before designing the ground investigation programme, a preliminary ground model should be established based on existing information identifying the relevant geotechnical units and the possible ground related hazards.

NOTE 2. Guidance on the evolution of the ground model is given in Clause 5.

(3) <RCM> Ground investigations should be carried out in phases to identify and progressively reduce uncertainties and increase reliability of the information about the ground.

NOTE 3. Complete removal of uncertainty is often neither possible nor economically feasible.

(4) <REQ> Ground conditions that influence the determination of Geotechnical Category shall be identified as early as possible in the investigation. <Possibly delete, if already in Part 1>

NOTE 4. As a result of the geotechnical investigations, it can be necessary to change the geotechnical category of the project (see EN 1997-1. 4.1.8.2).

(5) <REQ> For brownfield sites the natural and anthropogenic ground shall be clearly identified.

(6) <RCM> The presence and nature of any natural or anthropogenic contamination or aggressive ground, groundwater or gassy conditions that may influence the durability or safety of the structure should be identified as early as possible in the ground investigations.

NOTE 5. Aggressive ground can include Unexploded Ordnance,

(7) <REQ> Appropriate safety measures as specified by the relevant authority or agreed for a specific project by the relevant parties shall be taken when contaminated or aggressive ground or groundwater conditions are likely to be encountered which can affect the site investigators and the structure.

4.2 Desk study

(1) <REQ> Before designing the ground investigation programme, a desk study shall be carried out.

(2) <REQ> The desk study shall identify and examine all available and relevant information.

(3) <REQ> The desk study shall comprise a systematic review and interpretation of this information about the site and its environs, including the geology, groundwater conditions, any likely ground related hazards, previous uses of the site, and prior knowledge about the characteristics of the ground.

NOTE 6. Guidance on the desk study is given in Annex B.

4.3 Site inspection

(1) <RCM> The site should be visited and inspected, and the findings recorded and cross-checked against the information gathered by desk studies.

NOTE 7. Guidance on the site inspection is given in Annex B.

(2) <RCM> The site inspection should extend to the whole of the zone of influence which can extend outside the site.

NOTE 8. The zone of influence includes the extent of the structure affecting the ground and the extent of the ground affecting the structure.

(3) <RCM> Provided access to the site is available the site inspection should be made before the grounds investigation programme is designed.

(4) <PER> The site inspection may include geological or geomorphological mapping, shallow intrusive ground investigation and geophysical surveying to map the geological conditions.

NOTE 9. Shallow intrusive investigation includes, for example, use of hand augers or probing.

4.4 Sequencing of ground investigations

4.4.1 General

(1) <REQ> The results of the desk study and site inspection and then any phases of ground investigation shall be used to form proposals for the next and subsequent phases of ground investigations.

(2) <RCM> A phased ground investigation should be considered for GC2 and 3 as outlined in 4.1.2 to 4.4.4 below.

NOTE 10. A phased ground investigation provides a progressive reduction in ground risk to the project through the collection of more, and more reliable, information about the site.

(3) <REQ> The results from one phase of ground investigation shall be reviewed and the implications used in the design of the subsequent phase(s).

(4) <RCM> Any limitations in the data should be considered for clarification by further ground investigations.

4.4.2 Preliminary ground investigations

(1) <REQ> Preliminary ground investigations shall be planned such that adequate data are obtained to:

- assess the suitable positioning of the structure;
- assist preliminary design of the structure and the foundations;
- assess the potential need for and suitability of possible ground improvement methods;
- identify geotechnical hazards at the site;
- assess the possible effects of the proposed works on surroundings, such as neighbouring buildings, structures and sites;
- identify borrow areas;

- plan the design ground investigations, including identification of the extent of ground which may influence the behaviour of the structure and so needs to be investigated.

NOTE 11. A preliminary ground investigation can adopt relatively widely spaced ground investigation points which might be to clarify the geological succession and structure, and to identify the key issues that need to be addressed by the design investigation.

(2) <RCM> A preliminary ground investigation should supply estimates of ground data concerning the:

- material types and their stratification;
- discontinuity patterns in the ground;
- preliminary values of the strength and deformation properties of the units;
- groundwater table or pore pressure profile;
- potential occurrence of natural or anthropogenic contamination in the ground or groundwater.

4.4.3 Ground investigations for design and construction

(1) <REQ> Design ground investigations shall, in addition, be planned in order to:

- provide information for design of the temporary and permanent works;
- provide information to plan the method of construction;
- identify any difficulties that may arise during construction;
- identify ground conditions that have the potential to deleteriously affect the durability of the structure;
- identify the groundwater conditions and ground hydraulic properties relevant to the design and the proposed works
- identify the need for any more detailed ground investigations for optimisation of the design.

NOTE 12. Design ground investigation(s) are aimed at obtaining geotechnical parameters through testing and measurement for use in design.

NOTE 13. There may be one or more campaigns of design ground investigation; each phase should be used to update the ground model and thus help inform the necessary ground investigations to be carried out in the subsequent phase(s);

(2) <REQ> Parameters that are necessary to verify strength and stability of the structure shall be established before the start of the final design.

(3) <RCM> In order to ensure that the design ground investigation covers all relevant ground formations, particular attention should be paid to the following:

- profile of geotechnical units;
- natural or man-made cavities;
- stability of the rock, soil, or fill materials;
- hydrogeological conditions;
- creeping, expansible or collapsible soil and rock masses;
- presence of waste or man-made materials;
- presence of contaminated or aggressive ground or groundwater;
- possibility of ground gas, including radon.

4.4.4 Control and monitor investigations

(1) <REQ> Inspections, measurements and additional tests shall be made during the construction and execution of the project, in order to check whether the ground conditions encountered agree with those determined in the design ground investigations and whether the properties of the ground, construction materials and the construction works correspond to those presumed or specified.

NOTE 14. Monitoring of the ground at and around the site, and of the construction may also be carried out. Details of monitoring for different geotechnical structures are given in EN 1997-3.

(2) <REQ> Where design is by the observational method (EN 1997-1, 4.8), appropriate measurements and tests shall be identified together with limiting criteria and actions to be taken when those criteria are exceeded.

(3) <PER> The monitoring measures may include inspection or measurement of:

- the ground conditions and the formation exposed by excavation;
- soil and rock strengths and stiffnesses;
- groundwater level or pore pressures and their fluctuations;
- the behaviour of neighbouring constructions, services or civil engineering works;
- the behaviour of the actual construction.

(4) <REQ> The results of the monitoring measures shall be compiled, reported and checked against the design requirements.

4.4.5 Personnel for ground investigations

<For PT6 consideration>

<1st DRAFTING – having the competence requirements in 3 places (1997-1, 1997-2, 22475-1) is not good – they should be brought together in 1997-1 or in 1997-2. Annex G in 1997-1 does not include testing personnel (non-degree technicians) so has been broadened to include these in Annex A here >

<2nd DRAFTING NOTE> Discussion in revision of EN ISO 22475-2 (ISO TC 182) is looking to extend and unify the definitions of "operators" to include testing and monitoring technicians of all types. If this is done, and the wording is good, paragraphs 2 – 4 below can be combined and the need for the Annex here will reduce or even disappear. Situation being kept under review.

(1) <REQ> All persons planning, carrying out, reporting and evaluating the results of the ground investigation shall be suitably qualified and experienced for the tasks being carried out.

(2) <RCM> The qualifications and experience of those responsible for planning, supervising, reporting and evaluating ground investigations shall meet the requirements given in Annex D.

(3) <RCM> The qualifications and experience of those carrying out sampling (other than in a borehole), data collection, measurement and testing in the field and laboratory shall follow the requirements given in Annex D.

(4) <RCM> The qualifications and experience of those carrying out sampling and measurement in a borehole shall follow the requirements given in EN ISO 22475-2.

(5) <RCM> At all stages, the evaluation and interpretation of the results of any aspect of ground investigation should be checked by a competent person, who is familiar with the earlier stages of the ground investigation and design process.

5 Ground model

(1) <REQ> A ground model shall be formed of the conditions at, under, and around the site.

(2) <RCM> The preliminary ground model should be progressively developed by means of the desk study, site inspection and ground investigations.

(3) <REQ> The ground model shall identify the geological sequence, the structure (including layers, folds and faults) and the nature and variability of the ground and groundwater conditions.

NOTE 15. The sequence of geotechnical ground units often reflects the geological succession.

(4) <REQ> The scope of and detail within the ground model shall be consistent with the zone of influence of the structure.

NOTE 16. The zone of influence for different types of structure is given in EN 1997-3.

(5) <RCM> The ground model should be continually updated as more information becomes available through the sequence of ground investigations.

(6) <REQ> The ground model includes the results of the desk study, site inspection, and ground investigations (including field and laboratory tests) and shall be presented in the Ground Investigation Report.

NOTE 17. Any edition of the Ground Investigation Report will include the ground model available at the time of reporting.

<Drafting NOTE: following clause to be added to Part 1 >

(7) <REQ> In rock masses, the Geotechnical Design Model shall have enough information to allow the designer of the geotechnical structure to decide, for each limit state, if an approach of continuous or discontinuous media shall be used.

6 Ground investigations: type and extent

6.1 General

(1) <REQ> The type and extent of the ground investigations shall be based on the anticipated type and design of the construction.

(2) <REQ> The results of the desk study and site inspection shall be considered when selecting ground investigation methods and locating ground investigation points.

(3) <RCM> Ground investigations should consist of field investigations, testing, instrumentation and control and monitoring investigations.

(4) <PER> Field investigations may include:

- probing;
- drilling and/or excavations (test pits including shafts and headings) for inspection and sampling;
- geophysical investigations;
- field tests;
- description of soils, rock and discontinuities in exposures (natural and artificial) and samples;
- recovery of samples for laboratory testing of the soil or rock;
- groundwater measurements to determine the groundwater table or the pore pressure profile and their fluctuations
- installation of instrumentation to monitor ground movements or environmental conditions;
- large scale tests, for example to determine the bearing capacity or settlement characteristics or the behaviour of prototype elements, such as anchors.

(5) <PER> Back analysis of existing structures or failed structures may be used to provide calibration of ground conditions and material parameters at a larger scale than can be achieved by testing.

(6) <REQ> The ground investigation points shall be sufficient in number and arranged so as obtain sufficient and sufficiently accurate information of the characteristics and variability of the ground and groundwater at the site.

(7) <REQ> Ground investigations of soil and rock for use as construction materials shall provide a description of the materials to be used and shall establish their relevant parameters.

(8) <RCM> The information to be obtained from ground investigation for construction and for materials to be used in construction should include the items listed in Annex C.

(9) <RCM> Field investigation should normally be followed by testing of samples in the laboratory.

(10) <RCM> Where comparable experience of the type of construction in the local ground conditions is not available, representative preliminary field tests or trials should be considered using equipment, material and methods envisaged for the main works.

(11) <REQ> The results of field and laboratory tests shall be presented in the Ground Investigation Report (see Clause 13).

6.2 Description of the ground

(1) <REQ> All available natural or artificial exposures and samples shall be inspected and described to establish the ground profile.

(2) <RCM> The inspection should be supported by simple manual tests to identify the materials and to give a first impression of their consistency or strength and mechanical behaviour.

(3) <REQ> Soils observed in the site inspection and ground investigations shall be described and classified in accordance with EN ISOs 14688-1 and 14688-2.

(4) <REQ> Rocks observed in the site inspection and ground investigations shall be described and classified in accordance with EN ISO 14689.

(5) <REQ> Discontinuities such as bedding planes, cleavage, joints, fissures and faults shall be quantified with respect to their orientation, spacing, surface form and persistence in accordance with EN ISO 14689.

(6) <RCM> The recording of the soils, rock and discontinuities should take into account the size and quality of the exposure and any disturbance in its creation.

NOTE 18. The level of detail in the descriptions will depend on the size and quality of sample or exposure.

(7) <REQ> Weathering description and classification shall conform to EN ISO 14689 and be related to the geological processes and environmental conditions and shall cover the grades between fresh rock and rock decomposed or disintegrated to soil.

(8) <RCM> The soil and rock classification should be compared with available geological background information, including such as geological maps, where available.

(9) <PER> The ground may also be classified by the results of indirect measurements such as probing (including CPTu, DMT, PMT) or geophysical measurement.

(10) <REQ> The classification of the soil or rock mass should use samples of an appropriate quality class to enable identification of strata, discontinuities and possible cavities.

(11) <RCM> Total core recovery (TCR), solid core recovery (SCR) and Rock Quality Designation (RQD) should be determined on all rock core samples.

NOTE 19. TCR, SCR and RQD are defined in EN ISO 22475-1. <Definitions currently removed from that Revision>

6.3 Groundwater investigations

(1) <REQ> All possible information shall be gathered on the groundwater conditions at every stage of ground investigation, design and construction with observations made whenever possible, samples recovered when appropriate and with the installation and reading of monitoring instruments wherever necessary (see Clause 11).

6.4 Disposition of ground investigations

6.4.1 Location of ground investigation points

(1) <REQ> The depth and lateral extent of ground investigation shall be sufficient to identify the distribution of geotechnical units, their strength and deformation properties and their ease of excavation over the influence zone(s) of the construction on the ground and the ground on the structure .

NOTE 20. Guidance on the spacing and depth of ground investigations is given in Annex E.

(2) <RCM> The selection of ground investigation point locations should consider:

- the need to assess the depth and extent of geotechnical units across the site;
- the presence of critical points relative to the shape, structural behaviour and expected load distribution for a building or structure;
- the conditions along and at adequate offsets to the centre line of a linear structure, depending on the overall width of the structure, such as an embankment or a cutting;
- the stability of slopes or cuttings near structures and steps in the terrain (including excavations); ground investigation points should also be arranged outside the project area so that the stability of the slopes or cuttings can be assessed. Where anchorages are to be installed, due consideration should be given to the location of the load transfer zone and the likely stresses therein;
- avoidance of presenting a hazard to the structure, the construction work, or the surroundings;
- investigation of ground conditions to a distance where no harmful influences on the neighbouring area are expected;
- installation of groundwater measuring points and other monitoring instruments to determine background conditions;
- the possible need to continue monitoring after the ground investigation during and after the construction**.**

(3) <RCM> Where previous knowledge, local experience or the results of preliminary ground 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 ground investigation points may be used.

6.4.2 Spacing and depth of ground investigation points

(1) <RCM> The spacing and depth of ground investigation points for specific structures should conform to the requirements given in EN 1997-3.

(2) <RCM> The spacing of ground investigation points should be no greater than the values given in Table E.1 (NDP).

DRAFTING NOTE – the spacings that were given below (and crossed out) are taken from existing 1997-2; PT3 have prepared alternative figures (see E.1) based on some national practices (UK, ES, FR) and these are with WG2 for discussion. The figures in E.1 will be revised once feedback received.

(3) <REQ> The depth of ground investigation shall cover:

- the influence zone of the structure and the magnitude of loading;
- the effects of unloading of the ground;

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

(4) <RCM> Greater depths should be adopted in more complex ground conditions.

(5) <RCM> When mapping of the discontinuities is to be carried out in boreholes the use of vertical and non-vertical holes should be considered together with the use of down hole observation or logging.

(6) <REQ> In cases where more than one type of ground investigation is planned at a certain location the ground investigation points shall be separated by a distance appropriate to avoid any interference.

(7) <RCM> The separation of holes should normally be no less than 2 m.

NOTE 21. The sequence in which the different types of ground investigation are carried out should reflect the sensitivity of the methods to disturbance of the surrounding ground.

6.4.3 Number of tests

(1) <REQ> The minimum number of field or laboratory tests shall be established depending on the homogeneity of the ground, and how well that homogeneity is documented, and the quality and amount of comparable experience with the ground, and how well that comparable experience is documented.

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

(3) <RCM> The number of tests planned should consider the test method or range of test methods used, and what measurement(s) constitutes a test result.

NOTE 22. The use of more than one test method can provide confidence in the derived values particularly where the results are consistent.

6.4.4 Ground investigation programme

(1) <REQ> The specification of the scope of a ground investigation programme shall include:

- the locations of the ground investigation points;
- the types of ground investigation at each point;
- the depth of the ground investigation at each point;
- the field tests to be carried out including their number and depth;
- the samples to be recovered, their sampling category (according to EN 22475-1) and their number and depths;
- the details of the soil and rock description and discontinuity logging to be made in exposures, pits or boreholes;
- the measurement of groundwater levels or pressures including locations, depths, methods to be used and monitoring frequency and periods;
- details of instrumentation to be installed including types, locations and depths of installation and monitoring frequency and periods;
- the standards to be applied to all aspects of the works.

(2) <RCM> Results from the in ground vestigation programme should be kept under continual review and the programme of work adjusted as necessary to optimize the information available.

6.5 Testing and sampling

6.5.1 General

(1) <RCM> In setting up a testing and sampling programme, the likely geotechnical units at the site should inform the specification of number and type of tests.

(2) <RCM> Identification of geotechnical units at this stage should be based on the local geology, the geotechnical structure, and the required parameters for design.

6.5.2 Geophysical testing

(1) <PER> Geophysical tests may be used to identify:

- stratigraphy and lithology and, in particular, any lateral variation;
- natural and man-made cavities or in-filled cavities and holes;
- buried objects and artefacts that may interfere with the project;
- measurement of relevant ground properties;
- thickness of the weathering zone.

(2) <REQ> Suitability of a geophysical test shall be selected according to the sensitivity of the geophysical parameter to the ground characteristic being measured.

(3) <RCM> The suitability of surface geophysical tests should be checked according to their spatial resolution.

(4) <REQ> Results obtained from surface geophysical tests shall be calibrated against the ground data obtained from sampling and testing.

(5) <PER> Downhole geophysical tests may be used to investigate specific layers or changes with depth.

6.5.3 Sampling programme for soils and rocks

(1) <RCM> The sampling programme should recover samples from all geotechnical units for which identification and classification and laboratory testing is to be carried out .

(2) <REQ> The characteristics, quality class and number of samples to be recovered shall be based on the design verification methods, ground conditions, and laboratory testing proposed.

(3) <RCM>The sample quality classes relevant to the anticipated testing programme should be considered (EN ISO 22475-1) as given in Table 6.1.

(4) <REQ> Specimens for testing shall be selected from the trial pits, samples or cores according to the appropriate quality class.

NOTE 23. Recovered samples can be divided into specimens on which different tests can be carried out.

(5) <RCM> The disturbance of the samples and cores by the drilling process should be evaluated.

Table 6.1 — Quality classes of samples for laboratory testing

Soil or rock properties	**Quality classes of soil or rock specimens for laboratory testing**				
Unchanged ground properties	**1**	**2**	**3**	**4**	**5**
identification of soil and rock	*	*	*	*	*
particle shape and mineralogy	*	*	*	*	*
dimensions and grading of all particles	*	*	*	*	
Soil or rock texture	*	*			
particle size	*	*	*	*	
water content	*	*	*		
density, density index, porosity, permeability	*	*			
compressibility, shear strength	*				
Properties that can be determined					
boundaries of strata - coarse definition	*	*	*	*	*
boundaries of strata - fine definition	*	*	*	*	
weathering	*	*	*		
discontinuity description	*	*	*		
discontinuity properties	*	*			
Overall fracture state (including RQD in rock)	*	*			
Atterberg limits, particle density, organic content	*	*	*	*	
water content	*	*	*		
density, density index, porosity, permeability	*	*			
compressibility, shear strength, stiffness (at small strain)	*				

(6) <RCM> When preparing a sampling programme, the factors affecting sample quality should be considered in order to decide the degree of disturbance that can be accepted in a particular design and therefore the sampling methods to be specified.

(7) <REQ> The equipment for taking each sample shall be selected according to:

- the parameter to be measured; and therefore
- the test(s) to be carried out and the specimen quality required; and therefore
- the necessary sample quality class; and therefore
- the required diameter and mass of the sample.

(8) <RCM> In soils and rocks where sample damage might occur, and thus inappropriate quality specimens be recovered, the recovery of additional samples should be specified whenever possible.

(9) <REQ> Samples shall not be contaminated by material from other strata or from additives used during the sampling procedure.

(10) <REQ> Soil and rock sampling shall conform to EN ISO 22475-1.

(11) <REQ> Sample preservation, transport, and storage shall also conform to EN ISO 22475-1.

(12) <REQ> For Quality Class 1 and 2 samples the storage time before testing shall be minimised.

(13) <PER> Sampling equipment and methods other than those specified in EN ISO 22475-1may be used provided the specimen quality is no less than Quality Class 1.

NOTE 24. This may be the case when the deformation moduli (stiffness) at small strains are to be determined in undisturbed samples.

(14) <RCM> In inhomogeneous soil or rock, or if a detailed definition of the ground conditions is required, continuous, or very close, sampling by drilling should be carried out.

NOTE 25. Sampling of thin layers may be difficult, but such layers may be important to the design.

(15) <RCM> Sampling may be replaced by field tests if there is enough local experience to reliably correlate the field test results with the ground conditions.

6.5.4 Laboratory testing

(1) <RCM> If a sample contains more than one soil type, classification tests should be determined on specimens representing the different soil types.

(2) <RCM> Soil and rock descriptions and classification tests should be used to assess whether specimens and test results are representative of a geotechnical unit, and to report when this is not the case.

NOTE 26. In a first step, classification tests and strength index tests are performed on as many samples as possible to determine the variability of the index properties.

NOTE 27. In a second step, an assessment of how representative the samples tested are can be checked by comparing with the classification and strength index tests.

(3) <RCM> The quality of recovered samples should be assessed before testing and reported with the test result.

NOTE 28. The quality of samples can be made by visual examination with more detail provided by petrographic examination of soil fabric, x-rays or CT scanning.

NOTE 29. Quantitative assessment of sample quality for intact, low to medium overconsolidation ratio clays can be made by measuring volume change at the estimated in situ stress during laboratory consolidation (see Lunne et al., 2006).

6.6 Monitoring and instrumentation

(1) <RCM> A programme of monitoring of the ground and structures (pre-existing and under construction) should be planned using measurement and instrumentation.

(2) <RCM> The programme of monitoring should:

- establish conditions for inclusion in the design;
- determine seasonal and progressive changes with time;
- establish baseline conditions before construction;
- measure effects of construction.

(3) <REQ> Geotechnical monitoring by field instrumentation shall conform to EN ISO 18674.

(4) <PER> A range of instruments may be installed to allow monitoring of:

- groundwater conditions (groundwater levels and piezometric pressures);

- ground movements;
- strains in the ground;
- ground pressures;
- vibrations;
- other environmental conditions.

(5) <RCM> The programme of monitoring should commence as early as possible in the preliminary ground investigation and continue through the design period into construction and post construction.

(6) <RCM> The instruments should be located to avoid being damaged by construction activities.

(7) <RCM> The programme of monitoring should specify the reading frequency, the persons who will take the readings, and the persons who will receive and review the readings.

(8) <RCM> The party or parties responsible for maintenance of the instruments should be identified.

(9) <RCM> Damaged and non-functioning instruments should be replaced.

6.7 Test results and derived values

(1) <REQ> The various test results from a geotechnical unit shall be reviewed together and compared with descriptions, drilling logs, geophysical logs and comparable experience.

(2) <RCM> Designers should verify that the results of field or laboratory tests are at a scale, rate and with boundary conditions appropriate to the design situation(s).

(3) <RCM> Values of parameters should be derived from test results either:

- directly, when explicitly allowed by this code; or
- indirectly by using correlation based on local and relevant experience to relate the results obtained to those of a different test that allows direct derivation.

(4) <REQ> The correlations applied shall be documented by reporting, either directly or through reference to:

- the classes of ground for which they are applicable, preferably by reference to standardized physical and chemical properties (see Clause 7);
- the database that supports the model;
- the estimated transformation error, if available.

(5) <RCM> Use of site-specific data to support generic correlations should be considered for structures in Geotechnical Category 2 and above

NOTE 30. Site specific data frequently results in correlations with smaller error

7 Physical and chemical properties

7.1 Classification

(1) <RCM> Classification of soil for civil engineering purposes should conform to EN ISO 14688-2.

(2) <RCM> Classification of materials for earthworks should conform to EN 16907-2.

(3) <RCM>Classification of rock for civil engineering purposes should conform to EN ISO 14689.

(4) <RCM> Classification of a site for seismic purposes should conform to EN 1998-1.

(5) <RCM> Soil and rock classification tests should be performed to determine the composition and index properties of each geotechnical unit.

(6) <RCM> The samples for the classification tests should be selected such that the tests are approximately equally distributed over the complete area and the full depth of the geotechnical units.

(7) <RCM> For all classification tests, the temperature for oven-drying should take account of the constituents of the soil.

(8) <RCM> The classification of a specimen should be checked using the results from different types of tests.

(9) <RCM> Where practicable, the quality of the sample should be assessed by inspection, scanning or testing before laboratory tests are scheduled or performed.

7.2 Intrinsic physical properties

7.2.1 Particle density

(1) <REQ> The density of solid soil particles shall be determined in accordance with EN ISO 17892-3 for soils or EN 1097-6 for coarse soils and aggregates.

(2) <REQ> The test method used shall be appropriate for the soil type.

(3) <RCM> Soil specimens for measuring the particle density should be at least of Quality Class 4.

(4) <REQ> If, for a particular stratum, the measured values of the particle density are outside the range of (2 500 to 2 800) kg/m3, the mineralogy of the soil, its organic matter and its geological origin shall be confirmed by further testing.

(5) <RCM> In the case of soil with organic materials, the measured values should be used with caution.

7.2.2 Maximum and minimum void ratio

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

NOTE 31. No ISO or EN standard currently exists for these tests.

NOTE 32. National standards (including DIN 1812:xxxx, BS 1377-4:1990, NF P 94-059:2000) are available.

(2) <REQ> These values shall only be determined on coarse soils and an appropriate test method shall be reported.

NOTE 33. 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 method applied.

(3) <REQ> If the measured values of the void ratio are not within the normally expected range of (0,35 to 0,9), the particle size distribution shall be checked.

7.2.3 Particle size analysis

(1) <REQ> The mass percentage of individual particle size ranges found in the soil shall be determined using particle size analysis according to EN ISO 17892-4.

(2) <REQ> One of the following methods shall be used for particle size analysis, according to the size of the particles:

- the sieve method, for particles > 63 µm (or closest sieve available); or
- the sedimentation method using a hydrometer or pipette, for particles ≤ 63 µm (or closest sieve available);
- lase diffraction.

(3) <PER> Other methods that incorporate detection systems using density measurements or particle counters may also be used to measure particle size.

(4) <RCM> Other methods should be calibrated as given in Table 7.1.

(5) <PER> Equivalent methods may be used provided that they are calibrated against the two methods given in (2).

(6) <REQ> Preparation of the specimen shall be done by means which do not alter dimensions or shapes of particles. Prior to sedimentation, the specimens of fine soil shall not be dried.

(7) <RCM> Mineralogy and chemical composition should be investigated before performing particle size analysis.

NOTE 34. For some soils (for example chalk), treatment for carbonate removal is unsuitable.

NOTE 35. Carbonates and organic matter can have a cementing or coagulating effect and influence the particle size distribution.

Table 7.1 — Laboratory tests to determine particle size properties

Parameter/ property to be determined	***Test to measure property***	***Test standard***	***MQC***	***Comments as to suitability and interpretation***
Grain size distribution curve	*sieve method*	*EN ISO 17892-04 ISO 11277*	*4*	
coefficient of uniformity C_U	*sedimentation method*	*EN ISO 17892-04 ISO 11277*	*4*	*Carbonates and organic matter influence test results*
coefficient of curvature C_C	*Laser diffraction*	*EN ISO 13320*	*4*	*particle sizes ranging from approximately 0,1 µm to 3 mm. With special instrumentation and conditions, the applicable size range can be extended above 3 mm and below 0,1 µm*

(8) <REQ> In addition to the requirements of EN ISO 17892, the test report shall state:

- the drying method used;
- whether organics, salts and carbonates have been removed and by which method;
- the carbonate and/or organic content, if relevant;
- whether the mass fractions are reported with respect to the total mass (including carbonate and organic matter).

7.2.4 Particle shape

(1) <REQ> Descriptive and quantitative representation of particle shape and morphology shall be made according to ISO 9276-6:2008.

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

NOTE 36. Particle shape is a significant soil index property that influences engineering behaviour (e.g. in permeability, compressibility, shrinking/swelling potential).

(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.

7.2.5 Consistency (or Atterberg) limits

(1) <REQ> The change in behaviour of fine and many organic soils, when the water content is changing, shall be characterised using the consistency (or Atterberg) limits.

(2) <RCM> The consistency limits (Atterberg limits) should comprise the liquid limit, plastic limit and shrinkage limit.

(3) <REQ> The testing method used to determine the consistency limits shall be as specified in Table 7.2.

Table 7.2 — Laboratory tests to determine consistency limits

Parameter/ property to be determined	***Test to measure property***	***Test standard***	***MQC***	***Comments as to suitability and interpretation***
Plastic limit w_P	*Thread method*	*EN ISO 17892-12*	*4*	*Check specimen preparation, especially homogenisation and mixing* *Check whether drying has been used*
Liquid limit w_L	*Fall cone*	*EN ISO 17892-12*	*4*	*Liquid limit affected by method of test (cone, Casagrande cup)*
	Casagrande method	*EN ISO 17892-12*	*4*	*Results may not be reliable for thixotropic soil*
Methylene blue value MBV	*Methylene blue test*	*EN 933-9*	*4*	*For aggregates*
Shrinkage limit w_S	*Volumetric or linear method*	*ASTM D427* *NF P94-060.1 and 2*	*2*	

(4) <RCM> The fall cone method should be used to determine the liquid limit of soils in preference to the Casagrande method.

NOTE 37. The fall cone method gives more reliable results particularly for low plasticity soil.

(5) <RCM> The specimens should at least be of Quality Class 4, according to 6.1.3, if the test results are supposed to characterise the soil in situ.

NOTE 38. Further information on a procedure, presentation and evaluation of the determination of consistency limits can be found in EN ISO 17892-12.

7.2.6 Organic content determination

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

(2) <REQ> In very organic soil (OM ≥ approximately 20%) with little or no clay particles and little or no carbonate content, the organic content shall be determined directly from the loss on ignition at a controlled temperature. If the carbonate content is significant the temperature should not exceed 600 °C in order to avoid subtracting carbon present as carbonate (which will introduce an additional uncertainty).

(3) <REQ> In organic soils with a significant 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.

(4) <RCM> The organic content of soils may be determined from any of the tests specified in Table 7.3.

Table 7.3 — Laboratory tests to determine organic contents

Parameter/ property to be determined	***Test to measure property***	***Test standard***	***MQC***	***Comments as to suitability and interpretation***
organic content C_{OM} (loss on ignition LOI Vgl dry weight md weight after glowing mgl)	*Determination of loss on ignition*	*EN 15935*	*4*	*Environmental testing*
	sulfochromatic oxydation	*ISO 14235*	*4*	*not commonly used in geotechnical laboratories*
	Determination of loss on ignition	*ISO 10694*	*4*	*Methods given are combustion or acid dissolution not commonly used in geotechnical laboratories*
	Chemical analysis	*EN 1744-1*	*4*	*Unsuitable due to high oven temperatures*

(5) <REQ> For each test or series of tests, the following shall be specified:

- the drying temperature;
- the ignition temperature;
- the required corrections for bound water, carbonates, etc.;
- the factor used for converting carbon content into organic content.

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

(7) <RCM> Special testing methods that minimize the errors involved in the correction for bound water or carbonates should be used for clays and silty soils with moderate organic content.

7.2.7 Soil dispersibility and rock stability

(1) <REQ> The dispersive characteristics of clayey soil shall be identified according to one of standards specified in Table 7.4. Usual tests for classifying soil for engineering purposes do not identify the dispersive characteristics of a soil.

(2) <REQ> The stability of rock when immersed in water shall be identified according to standards listed in Table 7.4.

NOTE 39. 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 7.4 — Laboratory tests to determine dispersibility properties

Parameter/ property to be determined	***Test to measure property***	***Test standard***	***MQC***	***Comments as to 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*	*ASTM D4221-99* *BS 1377-5*	*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	*EN ISO 10930* *ASTM D6572-00*	*2*	*stability of soil aggregates subjected to the action of water* *Qualitative evaluation*
	Pinhole test	*ASTM D4647-93* *BS 1377-5*	*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*	*French standard at drafting level*	*2*	*Internal erosion on undisturbed or reconstituted specimens* *Hydraulic gradient should be specified according to the structure*
	jet erosion test	*ASTM D5852-95*	*2*	*In situ or laboratory on small surface* *Representativeness* *External erosion*

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

- the storage of samples such that the samples are not allowed to dry before testing;
- the testing procedures to be applied;
- the specimen preparation method.

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

(5) <RCM> Dispersibility tests are not applicable to soil with clay content of less than 10 % and with a plasticity index less than or equal to 4 %.

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

(7) <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.

7.3 State properties

(1) <REQ> To classify, identify and describe soils according to EN ISO 14688 series and rocks according to EN ISO 14689, state properties shall be identified according to standards listed in Table 7.5.

(2) <RCM> If a sample contains more than one soil type, state properties should be determined on the specimens representing the different soil types.

(3) <PER> For coarse soil, correction of measured water content and density may be needed. For large earthwork projects, method may need to be adapted, or use field method.

Table 7.5 — Laboratory and in situ tests to determine state properties

Parameter/ property to be determined	***Test to measure property***	***Test standard***	***MQC***	***Comments as to suitability and interpretation***
Bulk (total) 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*
	Determination of water absorption coefficient by capillarity	*EN 1925*	*2*	*Natural stone test methods*
	immersion in fluid method; fluid displacement method	*EN 1936*	*2*	*Natural stone test methods Determination of real density and apparent density, and of total and open porosity*
	Sand replacement method	*ISO 11272:2017 ASTM D 446-82 BS1377:Part 4 NF P 94-061-3*		*In situ test*
	Determination of loose bulk density and voids	*EN 1097-3*	*3*	*Suitable for coarse soils and aggregates*
water content w	*oven drying at a temperature of (105 ± 5) °C*	*EN ISO 17892-01*	*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*
	oven drying in a ventilated oven	*EN 1097-5*		*Suitable but the oven temperatures used vary between standards*
Macropores porosity	*Mercury intrusion porosimetry*	*ASTM D4044*	*2*	
	Water method	*EN 1936*	*3*	*Natural stone test methods - Determination of real density and apparent density, and of total and open porosity*
Bulk (total) density ρ	*Nuclear gauge*	*ASTM D6938 - 17a NF P 94-061-1*	*-*	
	Electrical density method	*ASTM D7698 - 11a*	*-*	
	Dynamic cone penetration	*EN ISO 22476-2*	*-*	*Use of correlations*
water content w	*Neutron depth probe method.*	*ISO 10573*		*Determination of water content in the unsaturated zone*

(4) <PER> The value of the degree of saturation (according to EN 17892-11), density index, and porosity may be computed from bulk density, water content and particle density. These parameters may be used in soil and rock classifications and in derivation of other geotechnical properties.

7.3.1 Bulk density

(1) <REQ> The bulk (total) mass density of soil or rock specimens, including any liquid or gas contained shall be determined according to EN ISO 17892 2 and EN 1936.

(2) <RCM> The test specimens should be at least of Quality Class 2, according to 6.1.3.

(3) <RCM> The evaluation of the test results should consider the possible sample disturbance.

(4) <PER> Check whether material can have enclosed pores; for such material, special techniques might be appropriate.

(5) <POS> If results fall outside the range of typical values, consider additional determinations; mineralogy and organic content may affect the results.

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

(7) <POS> The bulk, dry and particle densities can be used in evaluating other soil characteristics, establishing design values of actions derived from soil and in processing results of other laboratory tests.

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

7.3.2 Water content

(1) <REQ> The water content of a soil material shall be determined according to EN ISO 17892-1 and for rock according to ISRM suggested method (NF P94-410-1).

(2) <RCM> Soil specimens for measuring the water content should be at least of Quality Class 3, according to 6.1.3.

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

7.3.3 Porosity

(1) <REQ> The density and porosity of a sample shall be determined for soil according to EN ISO 17892-3, and for rock in EN 1936.

(2) <RCM> The pore volume may be calculated based on the dry density and the particle density determined using methods as for soil. Porosity is the ratio of pore volume to total volume.

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

- the selection of test samples;
- the conditions of storage before testing;

- whether desiccated samples are to be re saturated and by which technique;
- the number of tests required per formation;
- whether parallel tests are to be run on the same formation.

(4) <RCM> The porosity should be integrated in the reporting of rock description and established strength and deformation characteristics of the rock types in boreholes and at test sites.

(5) <RCM> The existence of closed pores can influence the porosity. The determination of the total pore volume should be based on the density of solids of a powdered sample.

7.3.4 In situ stress state

(1) <RCM> In situ stress state is an input of design models as initial state for the different limit state verifications. Although the initial stress state is often of limited interest for simple constitutive models as Mohr-Coulomb, the knowledge of the exact stress conditions is of primary importance for advanced constitutive models involving e.g. non-linear stress- and strain-dependent stiffness or creep behaviour.

(2) <REQ> In situ stress in rock masses is a very important aspect for design of structures in rock, namely where excavation is involved. The following points shall be checked:

- origin of the insitu stresses in rock masses (gravitational, tectonic, swelling, pre-existing structures or openings, influence of ground surface topography, previous loading or unloading, residual stresses, etc.);
- strategies for stress measurement programs;
- measurement methods used in rock masses;
- methodologies of analysis to obtain estimates of the stress field to use in design.

(3) <RCM> The initial stress state of soils and rocks should be determined using one or more of the tests given in Table 7.6.

Table 7.6 — Laboratory and in situ tests for determination of stress properties

Parameter/ property to be determined	***Test to measure property***	***Test standard***	***MQC***	***Comments as to suitability and interpretation***
In situ stress state: horizontal effective stress p'_0, K_0	*Flat jack*	*ISRM suggested method*	-	*measured response of the rock mass in a stress-disturbed zone (e.g. the wall of a tunnel)*
	Ménard pressuremeter	*EN ISO 22476-04*	-	*Pre-bored expansion test Specific procedure shall be used*
	Self-boring pressuremeter	*EN ISO 22476-06*	-	
	Marchetti dilatometer	*EN ISO 22476-11*	-	*Insertion by full displacement Choice of correlation depending of soil type*
	Total pressure cells	*EN ISO 18674-05*	-	*Insertion by full displacement*
	Triaxial	*EN 17892-09*	*1*	*Specific procedure shall be used*
Initial pore pressure	*Piezometers*	*EN ISO 18674-04*	-	
Pre-consolidation state: σ'_p, OCR	*Oedometer*	*EN 17892-05*	*1*	*Specific apparatus and procedure shall be used*

(4) <RCM> In general for the in situ stress state, the self-boring pressuremeter method should be preferred to displacement, pre-boring and laboratory method. The self-boring pressuremeter method gives more reliable results particularly for clayey soil.

(5) <RCM> For laboratory tests, the specimens should at least be of Quality Class 1, according to 6.1.3, if the test results are supposed to characterise the soil in situ.

NOTE 40. Further information on a procedure, presentation and evaluation of the determination of in situ stress state can be found in EN ISO 22476 series.

(6) <PER> Stress properties for soils may be determined indirectly using tests not given in Table 8 provided the testing, reporting, and interpretation procedures are explicitly provided in the Ground Investigation Report.

(7) <PER> In the absence of reliable test results, the coefficient of earth pressure at rest, K0, may be estimated from Formula 7.1:

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

where:

K_0 is the coefficient of earth pressure at rest;

φ' is the angle of shearing resistance in terms of effective stress;

OCR is the over-consolidation ratio.

NOTE 41. See also “Ko-OCR (At rest pressure - Overconsolidation Ratio)” relationships in soil [1].

7.3.5 Saturation

(1) <RCM> During laboratory testing when application of a back pressure, full saturation of soil or rock specimens (i.e. Sr = 100%) should be confirmed by checking the B value, if applicable.

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

NOTE 42. Further information on a procedure, presentation and evaluation of the determination of the volume of water can be found in EN ISO 17892-1 and of the volume of void in the EN ISO 17892-2 and EN ISO 17892-3.

NOTE 43. Further information on a procedure, presentation and evaluation of the determination of the volume of water can be found in ASTM D-5298-03.

7.4 Density index

(1) <REQ> Density index for coarse soils (sands and gravels) shall be determined upon the void ratio and minimum and maximum void ratios, as measured in the laboratory, according to ISO 14688-2.

(2) <RCM> The density index ID should be used for coarse soils to evaluate their compaction.

(3) <PER> Density index may be derived using correlation from in situ tests as cone penetration test (CPT), dynamic probing (DP), Borehole dynamic probing (BDP), standard penetration test (SPT), weight sounding test (WST).

NOTE 44. Table 7.7 gives some examples of in situ tests that can be used to determine density index indirectly.

Table 7.7 — In situ tests available to determine indirectly density index

Parameter/ property to be determined	***Test to measure property***	***Test standard***	***Confidence level***	***Example of correlation***
Density index I_D	*CPT*	*EN ISO 22476-01*	*medium*	$I_D = \frac{1}{C_2} \cdot ln\left[\frac{q_c}{C_0 \cdot (\sigma'_m)^{C_1}}\right]$
	DPT	*EN ISO 22476-02*	*low*	$I_D = C_1 + C_2 \cdot \log(N)$
	BDP	*EN ISO 22476-14*	*medium*	$I_D = C_1 + C_2 \cdot \log(N)$
	SPT	*EN ISO 22476-03*	*low*	$I_D = \left(\frac{(N_1)_{60}}{C_1}\right)^{0,5}$
	WST	*EN ISO 22476-10*	*low*	$I_D = C_2 + \left(\frac{N_{WST1}}{C_1}\right)^{0,5}$

(4) <REQ> A statistical analysis shall be provided to derive Cn coefficients for the formula used from Table 7.7. A confidence level shall be derived.

7.5 Degree of compaction

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

NOTE 45. In civil engineering works, compaction is widely used and practiced on soils intended to support mobile (roads, aerodromes, etc.) or static (building foundations, embankments, etc.) loads than on structures in earth (earth dams, embankments, etc.).

NOTE 46. The laboratory determination of compaction can miss the macro structure due to scale effects.

<Drafting NOTE: following clause needs to be reworded and linked to Table 7.8>

(2) <RCM> CBR (California Bearing Ratio) test consists in measuring the punching resistance of a number of soil specimens made according to a specific process. Proctor compaction procedure and CBR test should be systematically associated. The curves of variation of the immediate CBR index which directly characterize the water sensitivity of the soil under consideration are determined.

Table 7.8 — Laboratory tests to determine compaction properties

Parameter/ property to be determined	*Test to measure property*	*Test standard*	*MQC*	*Comments as to suitability and interpretation*
reference density and water content $\rho_{d;max}$, w_{opt}	*Proctor compaction*	*EN 13286-2*	*3*	*Unbound and hydraulically bound mixtures* *Limited in particle dimension to 20 mm*
	Vibrating hammer *Vibrating table*	*EN 13286-4* *EN 13286-5*		*Suitable for coarse soils and aggregates*
California bearing ratio CBR, immediate bearing index and linear swelling	*CBR test*	*EN 16905* *EN 13286-47*	*4*	*Unbound and hydraulically bound mixtures* *Limited in particle dimension to 20 mm*

(3) <PER> Proctor density and modified Proctor density may be used to characterise compaction.

(4) <RCM> 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.

NOTE 47. No ISO or EN standard currently exists for these tests.

7.6 Ground chemistry

7.6.1 General

(1) <REQ> The presence of certain chemical constituents in soil and rock and rock mass shall be determined when relevant for the design situation.

NOTE 48. Their action can be very significant, for example for the durability of the geotechnical structure

(2) <REQ> Chemical tests shall be performed to determine the presence of elements that can disrupt the setting of the concrete constituting the structure, create void by dissolution or pollute the site. These elements are assimilated in the form of ions in solution (state soluble in water of the soil); besides the soluble part, there are other states in which we can find these elements:

- adsorbed on the exchange complex;
- insolubilized in various forms;
- constituents of primary and secondary minerals.

NOTE 49. Many factors can interact with the soil solution such as organic matter but also pH.

(3) <RCM> Routine chemical testing in a soil laboratory is usually limited to organic content (loss on ignition, total organic content, organic matter), carbonate content, sulfate content, pH value (acidity or alkalinity) and chloride content.

(4) The chemical properties of soil and rock may be detrmined using any of the tests given in Table 7. 9.

NOTE 50. There are other chemical components that may cause an environment very aggressive to steel and concrete, for example magnesium and ammonium. The corresponding chemical testing is not covered in this standard.

NOTE 51. Corrosiveness to steel constructions in soil is usually evaluated by means of electrical resistivity tests and determination of the redox potential (not covered in this standard), pH, chlorides and sulfate determinations.

Table 7. 9 — Laboratory tests to determine chemical properties of soil and rock

Parameter/ property to be determined	***Test to measure property***	***Test standard***	***MQC***	***Comments as to suitability and interpretation***
mineralogy	*X-Ray diffraction*	*EN 13925*	*3*	
	Simplified petrographic description	*EN 932-3*	*3*	*For aggregates*
carbonate content C_{CaCO3}	*Volumetric method*	*EN ISO 10693* *DIN 18129* *BS 1377-3* *NF P 94-048*	*3*	*Scheilber instead of Dietrich-Früling*
sulfate, sulphide C_{SO4} *or* C_{SO3}		*EN 196-2*	*3*	
Hydrogen potential pH	*electrometric methods (acidity and alkalinity)*	*EN ISO 10390*	*3*	
chloride content C_{Cl} *and other salts*	*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*

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

(6) <REQ> The proper procedures of mixing, riffling and quartering shall be strictly followed in order to avoid inconsistent results.

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

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

(9) <REQ> The test results shall be reviewed together with the geological description and the prevailing environment.

(10) <RCM> Where appropriate, account shall be taken of recognised classifications in terms of the parameter measured.

7.6.2 Mineralogy

(1) <REQ> The mineralogical composition and petrographic description of a rock sample for identification purpose shall be performed according to EN ISO 14689-1 for rocks in EN 932-3 for aggregates and in EN 13925 (all parts) for soils.

(2) <RCM> The description should be carried out on cores and other samples of natural rock and on rock masses in situ.

(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.

7.6.3 Carbonate content determination

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

NOTE 52. 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).

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

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

7.6.4 Sulfate content determination

(1) <REQ> The sulfate content shall be measured to evaluate the possible detrimental effect of the soil and rock and rock mass on steel and concrete.

NOTE 53. All naturally occurring sulfates, with rare exceptions, are soluble in hydrochloric acid. Some are soluble in water.

(2) <REQ> The acid-soluble sulfate content is referred to as the total sulfate content, as distinct from the water-soluble sulfate content. Appreciation of which value is relevant shall be reported.

(3) < RCM > Groundwater containing dissolved sulfates, especially sodium and magnesium sulfates, can attack concrete and other materials placed in the ground or on the ground surface. Classification of soil and groundwater in terms of sulfate content should be made so that suitable precautionary measures can be taken.

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

7.6.5 pH value determination (acidity and alkalinity)

(1) <REQ> The pH value of groundwater or solution soil in water shall be measured to assess the possibility of excessive acidity or alkalinity.

(2) <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.

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

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

7.6.6 Chloride content determination

(1) <REQ> The salinity of the pore water or soil shall be assessed to determine the water-soluble or acid-soluble chloride content. The results provide an index for the possible effect of the groundwater towards concrete, steel, other materials and soil.

(2) <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.

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

7.7 Groundwater properties

7.7.1 General

(1) <RCM> Chemical tests are used to characterize groundwater and to assess the detrimental effect of the groundwater on concrete, steel and the soil itself and the specifications the designer shall provide for the laboratory. The tests are not intended for environmentally related purposes.

7.7.2 Density

(1) <REQ> The density of a water sample shall be determined using a hydrometer according to EN ISO 17892-4.

(2) <RCM> The evaluation should consider that the measured values can be influenced by temperature.

7.7.3 Chemistry

(1) <REQ> Chemical tests shall be performed to determine the presence of elements that can disrupt the setting of the concrete constituting the structure and also affects the water holding capacity, biological activity.

(2) <RCM> Routine chemical testing for water is usually limited to carbonate and carbon dioxide content, sulfate content, pH value (acidity or alkalinity) and magnesium content.

NOTE 54. Table 7.10 lists the most common chemical tests and their interpretation, and some guidelines.

Table 7.10 — Laboratory tests to determine groundwater chemical properties

Parameter/ property to be determined	***Test to measure property***	***Test standard***	***MQC***	***Comments as to suitability and interpretation***
carbonate content		*EN ISO 9963-1 and 2*		
Carbon Dioxide Content		*EN 13577* *ASTM D 513*		*Aggressive CO2 content* *Total CO2 content*
sulfate, sulphide,		*EN 196-2*		
Hydrogen potential (pH)	*electrometric methods (acidity and alkalinity)*	*EN ISO 10523*		*within the range pH 2 to pH 12 with an ionic strength below I = 0,3 mol/kg (conductivity: $\gamma_{25\,°C}$ < 2 000 mS/m) solvent and in the temperature range 0 °C to 50 °C*
Dissolved magnesium content	*flame atomic absorption spectrometry*	*EN ISO 7980*		*magnesium content of up to 5 mg/l*

8 Strength properties

8.1 Description of strength

(1) <REQ> Ground strength shall be described using strength envelopes.

NOTE 55. Strength envelopes can describe one or various failure modes. A specific failure mode or combination thereof is dominant in most practical applications.

(2) <PER> Strength envelopes may be defined in terms of total or effective stress.

(3) <RCM> The stress range of application should be indicated when a ground strength envelope is specified for design.

(4) <RCM> It should be indicated if a ground strength envelope applies to a:

- peak strength condition; or
- constant volume shearing strength condition; or
- residual strength shearing condition.

(5) <PER> A tensile strength σt different from zero may be included in ground strength descriptions.

NOTE 56. Tensile strength is particularly relevant for rock material and improved ground.

8.2 Strength parameters

8.2.1 Mohr-Coulomb strength envelopes

(1) <PER> Ground shear strengths may be described by a linear Mohr-Coulomb envelope given by Formula (8.1):

$$\tau = \sigma \tan \varphi + c \tag{8.1}$$

where:

τ is the shear stress on the failure plane;

σ is the normal stress on the failure plane;

c is the cohesion;

φ is the friction angle.

NOTE 57. Other expressions of Mohr Coulomb envelopes are found in ISRM Suggested Methods (Ulusay, R. (Ed.). (2014). The ISRM suggested methods for rock characterization, testing and monitoring: 2007-2014. Springer)

(2) <REQ> Under constant volume conditions, cohesion shall be assumed to be zero (c = 0).

(3) <PER> Undrained shear strength may be represented by a Mohr-Coulomb envelope given by Formula (8.2):

$$\tau = c_u \tag{8.2}$$

where:

τ is the shear stress on the failure plane;

c_u is the undrained strength.

NOTE 58. Conditions for total stress analyses are given in EN 1997-1, 4.2.2.

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

8.2.2 Hoek-Brown strength envelope

(1) <PER> For both rock material and rock-mass, shear strengths may be described using appropriate non-linear Hoek-Brown envelopes given by Formula (8.3):

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^{\alpha} \tag{8.3}$$

where:

σ_1 and σ_3 are principal stresses;

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

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

NOTE 59. Other expressions of Hoek-Brown envelopes are found in ISRM Suggested Methods (Ulusay, R. (Ed.). (2014). The ISRM suggested methods for rock characterization, testing and monitoring: 2007-2014. Springer)

(2) <RCM> For rock material (intact rock), the values of the parameters s and α should be taken as s = 1 and α = 0.5.

8.2.3 Other models

(1) <PER> Alternative strength envelopes to those defined above may be also employed.

NOTE 60. 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) <RCM> If strength envelopes are implicitly defined in numerical models, results should be presented demonstrating the fitting of Mohr-Coulomb envelopes appropriate to the relevant design conditions.

8.3 Evaluation of soil strength parameters

8.3.1 Direct determination

(1) <PER> Strength parameters for soils may be determined directly using any of the laboratory tests listed in Table 8.1.

Table 8.1 – Tests for indirect determination of soil strength parameters

Test	*Test Standard*	*Parameters that can be obtained*	*Specific interpretation guidelines*
Unconfined compression strength (UCS)	EN ISO 17892-7	$c_{u,\,peak}$	See (5) to (9) below
Unconsolidated undrained triaxial compression test (UUTX)	EN ISO 17892-8	$c_{u,\,peak}$	See (5) to (9) below
Consolidated triaxial compression test (TX)	EN ISO 17892-9	$c_{u,\,peak}$; $c_{u,cv}$ c'_{peak}, φ'_{peak}; φ'_{cv}	See (5) to (8), (10) and (11) below
Direct shear	EN ISO 17892-10	c'_{peak}, φ'_{peak}; φ'_{cv} c'_{res} , φ'_{res}	See (5) to (8), (10), (12) and (13) below
Ring shear	EN ISO 17892-10	c'_{peak} , φ'_{peak}; φ'_{cv} c'_{res} , φ'_{res}	See (5) to (8) and (10) below
Field Vane (FVT)	EN ISO 22476-9	$c_{u,peak}$; $c_{u,res}$	See (14), (15) below

(2) <PER> Strength parameters for soils may be determined directly using tests not given in Table 8.1 provided the testing, reporting, and interpretation procedures conform to (3) below.

(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 (if applicable);
- time for consolidation increments (if applicable);
- shearing rate;
- criteria for terminating tests (e.g. strain at which the test shall be stopped);
- acceptability criteria (e.g. saturation, scatter);
- accuracy of measurements.

(4) <RCM> For laboratory tests not listed in Table 8.1, the test results should include, where applicable:

- effective stress paths;

- Mohr circles at failure;
- stress-strain curves;
- pore pressure-strain curves.

(5) <RCM> For the determination of the peak shear strength parameters of clay, silt, and organic soil, specimens prepared from undisturbed samples (Quality Class 1) should be used.

(6) <PER> For the determination of constant volume or residual strength parameters of clay, silt, and organic soil, reconstituted specimens from samples of Quality Class 4 and above may be employed.

(7) <PER> For the determination of strength parameters of coarse soils, reconstituted specimens from samples of Quality Class 4 and above may be employed.

(8) <REQ> If reconstituted specimens are employed to determine shear strength parameters, 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 61. It is generally desirable to reconstitute specimens at state conditions close to those in situ.

NOTE 62. Methods of preparing reconstituted specimens are given in EN ISO 17892-9.

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

(10) <RCM> If effective cohesion greater than zero is reported in a test, the applicability of this parameter in the design situation should be checked.

NOTE 63. 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.

(11) Non-uniform failure modes in triaxial specimens should be taken into account when determining constant volume strength parameters.

NOTE 64. Special procedures not covered by EN ISO 17892-9 (e.g. lubricated end platens) are sometimes needed to avoid non-uniformities in triaxial specimens.

(12) <PER> Provided the conditions given in (13) are satisfied, friction angles derived from direct shear tests may be related to friction angles in plane strain conditions through formula (8.4):

$$\sin \varphi'_{ps} = \frac{\tan \varphi'_{ds}}{\cos \psi_{ds}\left(1+\tan \psi_{ds} \tan \varphi'_{ds}\right)} \tag{8.4}$$

where:

φ'_{ps}	is the effective friction angle corresponding to plane stress conditions
$\varphi'_{ds} = \tan^{-1}\left(\frac{\tau}{\sigma_v}\right)$	is the effective friction angle measured in direct shear, where τ is the shear stress and σ_v the vertical stress
$\psi_{ds} = \tan^{-1}\left(\frac{dz}{dx}\right)$	is the angle of dilatancy, measured in direct shear as the incremental ratio of vertical (dz) to horizontal (dx) specimen displacement

(13) <RCM> Formula (8.4) should only be used if:

– the soil is coarse according to ISO 14688-2;
– the design of the shear box ensures that the box does not rotate during shearing.

(14) <RCM> The effect of the rate of testing should be corrected when the undrained strengths obtained from FVT are compared to those obtained from laboratory tests or back-analyses of field failures.

(15) <PER> Rate effects affecting FVT undrained strength may be corrected using formula (8.5)

$$c_{u,ref} = \mu_R c_{u,FV}$$

$$\mu_R = 1.05 - b\sqrt{I_P} \qquad (8.5)$$

$$b = 0.015 - 0.0075 \log_{10} t_f$$

where:

t_f is the time to failure (minutes) relevant to the reference strength value;

I_P > 5%, is the plasticity index of the soil (see EN ISO 17892-12).

NOTE 65. The original database supporting the relation is described in Chandler (1988).

<DRAFTING NOTE> ISO 22476-9 is currently under revision and may finally include there a correction for rate effects. If so, this paragraph will not be needed here.

8.3.2 Indirect determination

(1) <PER> Strength parameters for soils may be determined indirectly using any of the tests listed in Table 8.2.

Table 8.2 – Tests for indirect determination of soil strength parameters

Test	Test Standard	Parameters that can be obtained	Specific interpretation guidelines
Fall cone	EN ISO 17892-6		See (5) below
Plasticity	EN ISO 17892-12	c'_{peak}, φ'_{peak}; $c_{u,peak}$	See (5) below and Annex 8A
CPT	EN ISO 22476-1	$c_{u,peak}$; φ'_{peak}	See Annex 8A
SPT	EN ISO 22476-3	$c_{u,peak}$; φ'_{peak}	See Annex 8A
PMT	EN ISO 22476-4	$c_{u,peak}$; φ'_{peak}	See Annex 8A
DMT	EN ISO 22476-7	$c_{u,\,peak}$; φ'_{peak}	See (6) below and Annex 8A

(2) <PER>Strength parameters for soils may be determined indirectly using tests not given in Table 8.2 provided the testing and reporting procedures conform to (3) below and interpretation to (4).

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

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

(4) <REQ> For tests not listed in Table 8.2 the transformation models used for interpretation shall comply with 4.3.

(5) <REQ> Design shall not rely on strength values which are derived from this kind of tests alone.

(6) <RCM> Relations between DMT results and peak undrained strength should be derived from those established between DMT results and OCR (see Clause 7).

(7) <PER> In the absence of data indicating otherwise, density index values (see 7.4) may be used to determine peak strength of coarse soils through formula (8.6):

$$\varphi'_{peak} = \varphi'_{cv} + m(I_D[Q - \log_e p'] - 1) \tag{8.6}$$

where:

φ'_{cv} constant volume angle of friction;

m coefficient that depends on the relevant shear mode to failure (m = 5 in plane strain and m = 3 in triaxial compression);

Q coefficient that depends on the crushability of the material (for indicative values see Note 1 below);

p' mean principal effective stress at failure.

NOTE 66. For quartz and feldspar grains Q = 10. For carbonate grains Q = 7.

8.4 Evaluation of rock and rock-mass strength parameters

<DRAFTING NOTE> Section only partly developed as PT6 is tasked with reviewing and developing rock mechanics aspects of the code.

8.4.1 Direct determination

(1) <POS> Strength parameters for rock material may be determined directly using any of the laboratory tests listed in **Error! Reference source not found.**.

Table 8.3 – Laboratory tests for direct determination of rock strength parameters

Test	Test procedures	Parameters that can be obtained	Specific interpretation guidelines
Unconfined compression strength (UCS)	ISRM Suggested Method[a] NF P 94-420 ASTM D 2938-95	σ_{ci}	See (2) below
Brazilian test (BT)	ISRM Suggested Method[b] NF P 94-422 ASTM D 2936-95	σ_t	See (2) below
Triaxial test (TX)	ISRM Suggested Method[c] NF P 94-423 ASTM D 2664-95	c_{peak} , φ_{peak}; m_b	See (2) below

[a]ISRM (2007) Suggested method for determination of the uniaxial compressive strength of rock materials
[b]ISRM (2007) Suggested methods for determining tensile strength of rock materials
[c]ISRM (2007) Suggested methods for determining the strength of rock material in triaxial compression

(2) <REQ> If an ISO CEN test standard dealing with this test is approved it shall take precedence over the alternative procedures mentioned in the table

8.4.2 Indirect determination

(1) <POS> Strength parameters for rock material may be determined indirectly using any of the laboratory tests listed in Table 8.4.

Table 8.4 – Laboratory tests for indirect determination of rock strength parameters

Test	Test procedure	Parameters that can be obtained	Specific interpretation guidelines
Point load test	ISRM Suggested Method[a] NF P 94-429 ASTM D 5731-08	σ_{ci}	See (2), (3) below
Schmidt hammer test	ISRM Suggested Method[b] ASTM D5873 - 14	σ_{ci}	See (2), (3) below

[a]ISRM (2007) Suggested method for determining point load strength.
[b]ISRM (2014) Suggested Method for Determination of the Schmidt Hammer Rebound Hardness.

(2) <REQ> If an ISO CEN test standard dealing with this test is approved it shall take precedence over the alternative procedures mentioned in the table

(3) <REQ> Design shall not rely on strength values which are derived from this kind of tests alone

8.5 Evaluation of rock-mass strength parameters

8.5.1 From rock–mass classifications

<DRAFTING NOTE> This Clause to be developed with assistance from PT6

8.5.2 From back-analyses

<DRAFTING NOTE> This Clause to be developed with assistance from PT6

8.6 Rock joint strengths

(1) <RCM> The peak shear strength of joints should be described using non-linear strength envelopes.

(2) <PER> The expression given by Formula (8.9) may be used to describe a peak shear strength envelope for rock joints:

$$\tau = \sigma_n \tan\left(JRC \log_{10}\frac{JCS}{\sigma_n} + \varphi_r\right) \tag{8.9}$$

where

σ_n is the normal stress acting on the joint;

JRC is the joint roughness coefficient;

JCS is the joint wall compressive strength;

φ_r is the joint residual friction angle.

NOTE 67. Methods to evaluate the parameters intervening in formula 8.9 are given in ISRM (2004) Suggested methods for the quantitative description of discontinuities in rock masses

(3) <RCM> Scale effects should be taken into account when extrapolating laboratory based joint strength envelopes in design.

NOTE 68. Criteria to adapt the Formula 8.9 to account for scale effects are given by Barton & Bandis (1990).

8.7 Interface strengths

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

(2) <RCM> When measuring interface strengths, the roughness of the material surface should be recorded.

9 Stiffness and consolidation properties

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 elastic moduli, each modulus limited to a particular stress or strain range.

NOTE 1. Relevant moduli include tangent moduli, such as the initial Young's modulus of elasticity (E0), and secant moduli, such as Young's modulus at 50 % of the maximum shear stress (E_{50}).

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

(4) <PER> For structures in Geotechnical Categories 1 or 2, ground stiffness properties may be determined indirectly (using appropriate transformation models), according to 9.1.3.

(5) <PER> For structures in Geotechnical Category 1, ground stiffness properties may be estimated using empirical models, according to 9.1.4.

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

(7) <REQ> In tests employed for direct determination of ground stiffness:

- relevant stress and strain measurements shall be directly available from the test; and
- the operator shall be able to control the loads or deformations imposed in the test.

(8) <REQ> Elastic moduli shall be defined at specified stress or strain levels taking into account any non-linearity in soil or rock behaviour.

(9) <REQ> Consolidation conditions and stress paths to be used during acquisition of stiffness parameters shall be defined in accordance with the limit state to be verified (i.e. use of advanced constitutive models).

(10) <REQ> When determining stiffness of soils and rocks, the factors affecting (measurement of) stiffness and therefore test method selection shall be considered:

- intrinsic and state properties (density, etc.);
- scale of specimens according to ground mass;
- strain level compared to the one present in the ground;
- strain rate adapted to the limit state i.e. short term/long term, undrained/drained, ultimate/serviceability;
- degree of freedom.

(11) <REQ> The effect of time (i.e. short- or long-term behaviour) on stiffness shall be taken into account.

(12) <REQ> Repeated loading and/or seismic action shall be taken into account when they can affect ground stiffness (see 10).

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

(14) <REQ> The way parameters are transformed using transformation model or theory shall be mentioned in in Ground Investigation Report (see Annex E).

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

(16) <REQ> Instrumentation capable of measuring stresses and strains with high resolution shall be used for stiffness determination at strain levels below 0,1 %.

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

(18) <PER> Poisson ratio may be derived from drained triaxial compression tests.

(19) <PER> Bulk modulus may be determined using specific stress paths in triaxial tests.

9.1.2 Direct determination of ground stiffness

9.1.2.1 By field testing

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

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

Table 9.1 – Field tests to determine ground stiffness

Parameter	Strain level	Field test (and associated test standard)									
		PLT	DMT	BJT	PMT	FDT	FDP	Drillhole deformation gauges	SBP	Seismic DMT/ CPT/PMT	MASW-CH-DH-UH
		EN ISO 22476-13	EN ISO 22476-11	EN ISO 22476-7	EN ISO 22476-4	EN ISO 22476-5	EN ISO 22476-8	ISRM suggested methods	EN ISO 22476-6	Draft needed	Draft needed ISRM suggested methods
	Modulus	E_{PLT}	E_{DMT}	E_{BJT}	G, E_{PMT}	E_{FDT}	E_{FDP}		G, E_{SBP}	$G_{0;FT}$	$G_{0;CH}$
very small strain	< 10^{-5}	-	-	-	-		-	-	-	P	P
small strain	10^{-5} - 10^{-2}	-	-	-	-	S-C	-	-	C	-	-
medium strain	10^{-2} - 10^{0}	S-C	P	S-C	S-C	-	S-C	S-C	C	-	-
large strain	> 10^{0}	S-C	-	-	S-C	-		-	-	-	-
"-" not applicable; result type: Point; S: Slope; C: Curve											

(3) <PER> Field measurements of stiffness may be obtained using tests not listed in **Error! Reference source not found.** 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) <REQ> The stiffness derived from field tests for known stress condition shall be related to shear or elastic modulus either by theoretical relationships or correlation.

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.

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

Table 9.2 – Laboratory tests to determine ground stiffness

Parameter	Strain level	Laboratory test (and associated test standard)							
		OED	DSS	TX (local)	UCS	CTX	RC	BE	P wave
		EN ISO 17892-5	EN ISO 17892-9	Draft needed	EN ISO 17892-7 EN 14580	Draft needed	Draft needed	Draft needed	EN 14579 EN 14146
	Modulus	E_{oed}, C_c	G, G_{cycl}	E, G		G_{cycl}, E_c	$G_{0;RC}$	$G_{0;BE}$	K
very small strain	$< 10^{-5}$	-	-	-	-	-	-	P	P
small strain	10^{-5} - 10^{-2}	-	S-C	(S-C)	(S-C)	S-C	C	-	-
medium strain	10^{-2} - 10^{0}	S	S-C	S-C	S-C	-	-	-	-
large strain	$> 10^{0}$	-	-	S-C	S-C	-	-	-	-
— = not applicable; ( ) = partially applicable only									

(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) <REQ> The stiffness derived from laboratory tests for a defined confining stress shall be related to shear or elastic modulus either by theoretical relationships or correlation.

NOTE 69. Some relationships are provided in the according standard.

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

NOTE 70. Annex E gives indicators of specimen quality that can be used to ensure a minimum quality class.

NOTE 71. 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.

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

(7) <PER> Reconstituted, reconsolidated specimens may be used to obtain lower bound measurements of stiffness, particularly for coarse soils and fills.

(8) <RCM>Reconstituted specimens should have approximately the same composition, density and water content as the in situ material.

(9) <REQ> The procedure used to reconstitute the soil shall be reported in the laboratory test report and Ground Investigation Report.

9.1.3 Indirect determination of ground stiffness

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

Table 9.3 – Indirect determination of ground stiffness

Parameter	Symbol	Test	Test Standard	Confidence level
Shear modulus	G	SPT	EN ISO 22476-3	low
Young's modulus	E	DP	EN ISO 22476-2	low
		Back analysis	EN 1997-1 §4.3.2 (2) and §4.8 (6) EN ISO 18674-1	medium
Drained Young's modulus	E'	CPT	EN ISO 22476-1	low
		PMT	EN ISO 22476-4	high
Oedometer modulus	E_{oed}	CPT	EN ISO 22476-1	medium
		SPT	EN ISO 22476-3	low
Ménard modulus	E_M	MWD	EN ISO 22476-15	low

(2) <PER>Indirect measurements of stiffness may be obtained using tests not listed in Table 9.3 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) <PER> The confidence levels in Table 9.3 may be improved by statistical analysis on site specific data.

(4) <RCM> A statistical analysis should be provided to derive a mean value and standard deviation according to Annex D of EN 1990. A confidence level should be derived.

(5) <REQ> The formula used to obtain the modulus should be given and all terms and parameters defined.

9.1.4 Empirical models for estimating ground stiffness

(1) <PER> For structures in Geotechnical Category 1, the secant shear modulus of a soil (Gsec) may be estimated from Formula (9.3):

$$\frac{G_{sec}}{G_0} = \left[1 - \left(\frac{\gamma - \gamma_e}{\gamma_{ref}}\right)^m\right]^{-1} \tag{9.3}$$

where:

G_0 is the soil's very-small-strain shear modulus;

γ is the engineering 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 72. See Bolton et al. "Stiffness of sands through a laboratory database" for further information about this formula.

NOTE 73. Values of the parameters in Formula (9.3) are given in Annex E.

(2) <PER> Formula (9.3) may also be used to validate direct or indirect measurements of stiffness.

(3) <PER> For structures in Geotechnical Category 1, the very-small-strain shear modulus of a soil (G0) may be estimated from Formula (9.4):

$$\frac{G_0}{p_{ref}} = \frac{k_1}{(1+e)^{k_2}} \left[\frac{p'}{p_a}\right]^{k_3} \tag{9.4}$$

where:

e	is the soil's void ratio;
p'	is the mean effective stress in the soil;
p_a	is atmospheric pressure (100 kPa);
k_1, k_2, and k_3	are coefficients that depend on soil type.

NOTE 74. Bolton et al, (2000) "non-linear soil stiffness in routine design" and Clayton C.R.I. et al. (2011) "stiffness at small strain: research and practice" for further information about this Formula.

NOTE 75. Values of the parameters in Formula (9.4) are given in Annex E.

(4) <PER> Formula (9.4) may also be used to validate direct or indirect measurements of very-small strain stiffness.

9.2 One-dimensional compression

9.2.1 General

(1) <RCM> One-dimensional soil compression should be described by a load-compression curve over the expected stress and strain ranges for the anticipated design situation.

(2) <PER> One-dimensional ground compression may be approximated by one or more compression parameters, each parameter limited to a particular stress or strain range and time period.

NOTE 76. Relevant parameters include the swelling index (C_s), compression index (C_c), creep index (C_α) and pre-consolidation pressure (σ'_p).

(3) <RCM> One-dimensional compression properties should be determined directly (from test results), according to 9.2.2.

(4) <RCM> The coefficient of consolidation (c_v) should be determined directly (from test results), according to 9.2.2.

(5) <RCM> Testing program should be adapted in some soils and rocks when swelling or viscous (time dependent) behaviour is encountered.

(6) <PER> Compressibility parameters for soils in unsaturated state may be determined to establish and 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.

NOTE 77. No ISO or EN standard currently exists for these tests.

9.2.2 Direct determination of ground compression or swelling

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

Table 9.4 – Laboratory tests to determine compression and creep

Laboratory test (and associated test standard)	**Compression , consolidation and creep of soils (OED)**	**CRS**	**triaxial**	**Creep characteristics of Rock**
	EN ISO 17892-5	BS 1997-6 SS 0271 26	EN ISO 17892-9	ISRM Suggested Methods
Parameter	C_s; C_c; c_v; C_α	m_v; c_v; u	C_c	K

9.2.3 Indirect determination of ground compression or swelling

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

Table 9.5 – Indirect determination of ground compression or swelling

Parameter	Symbol	Test	Test Standard	Confidence level
Compression index	C_c	Liquid limit	EN ISO 17892-12	medium
		Water content	EN ISO 17892-1	medium
Coefficient of secondary compression	C_α	Compression index	EN ISO 17892-5	low
Coefficient of consolidation	c_v	Liquid limit	EN ISO 17892-12	low
Swelling index	C_s	Compression index	EN ISO 17892-5	low

(2) <PER>Indirect measurements of ground compression or swelling 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) <PER> The confidence levels in Table 9.5 may be improved by statistical analysis on site specific data.

(4) <RCM> A statistical analysis should be provided to derive a mean value and standard deviation according to Annex D of EN 1990. A confidence level should be derived.

(5) <REQ> The formula used to obtain the ground compression or swelling parameters shall be given and all terms and parameters defined.

9.2.4 Empirical models for estimating ground compression or swelling

(1) <REQ> The coefficient of consolidation (C_v) shall be derived using the one-dimensional consolidation theory.

9.2.5 Swelling properties

(1) <RCM> The axial swelling strain and stresses developed against a constant axial surcharge, when a radially confined, undisturbed soil or rock specimens are immersed in water shall be evaluated in accordance with standards listed in Table 9.6.

NOTE 78. The swelling pressure of soil and rock materials plays an essential role in the design of raft foundations placed above the swelling ground or walls supporting these soils.

Table 9.6 — Laboratory tests to determine swelling properties

Parameter/ property to be determined	***Test to measure property***	***Test standard***	***MQC***	***Comments as to suitability and interpretation***
Swelling pressure σ_g	*One specimen with axial surcharge Huder Amberg method*	*ISRM suggested methods*	*1*	*σ'_{vo} deduce from ground model should be provided*
	under zero volume change	*NF P94-090 ASTM D4546*	*1*	
Swelling amplitude ε_g	*free swelling*	*NF P94-090*	*1*	
	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*

(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 preferably be tested where possible, since material fabric has an important effect on swelling characteristics. Where the sample is too weak or too broken to allow preparation, such as joint fill material, the swelling index tests may be carried out on remoulded and re-compacted specimens. The procedures used should then be described in the report.

10 Mechanical response to dynamic loads and parameters for seismic design

10.1 General

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

- seismic design;
- design for cyclic loadings;
- design for vibrations induced by human activities.

(2) <RCM> Ground investigations for dynamic loading should provide the relevant information on:

- stress-strain response to cyclic loads, including small strain elastic moduli;
- development of excess pore pressures under cyclic loads;
- cyclic shear strength .
- post cyclic behaviour in terms of post-cyclic shear strength, consolidation of cyclic-induced pore water pressure and post-cyclic creep

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

10.2 Measurement of cyclic response

(1) <RCM> The response to cyclic loading should be investigated in the laboratory.

(2) <REQ> In laboratory tests (see Table 10.1), the range of cyclic strains investigated shall be consistent with the expected level of strains for the specific design situation.

(3) <RCM> The testing method should be chosen to match, as far as possible, the frequency bandwidth of the expected loading.

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

(5) <REQ> Care shall be exercised during specimen preparation to preserve soil density and microscopic soil fabric.

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

(7) <PER> When undisturbed sampling is not feasible, the test may be performed on reconstituted samples that reproduce the state conditions (stress, density and saturation) of the soil in situ.

(8) <RCM> If reconstituted specimens are employed, the possible differences between the fabric of the natural and reconstituted soil should be taken into account.

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

(10) Methods of preparing reconstituted specimens are given in EN ISO 17892-9

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

Table 10.1 – Laboratory methods for cyclic tests

Shear strain range γ	Test	Acronym	Test Standard
$10^{-6} \div 10^{-2}$	Resonant Column Test	RC	ASTM D4015 - 15e1
$10^{-5} \div 10^{-2}$	Cyclic Torsional Shear Test	CTS	-
$10^{-5} \div 10^{-2}$	Cyclic (Direct) Simple Shear Test	CDSS	-
$10^{-4} \div 10^{-1}$	Cyclic Triaxial Test	CTx	ASTM D3999 / D3999M - 11e1; ASTM D5311/D5311M – 13
10^{-6}	Bender Element Test	BE	-

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.

(2) <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 79. Examples of indirect methods are reported in Annex I.

(3) <REQ> The indirect methods shall consider:

- grain size distribution;
- plasticity index;
- in situ state of stress.

(4) <RCM> The indirect methods should also take into account:

- overconsolidation ratio;
- frequency of the expected loading;
- number of equivalent cycles.

10.3.2 Measured values

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

(2) <REQ> When large shear strains (> 1 %) are expected, the secant shear modulus curves shall be corrected so that they are compatible with the estimated shear strength of the soil.

(3) <RCM> The variation of shear modulus versus cyclic shear strain should be normalized by the value of shear modulus at very low strain (cyc < 10-4 %) as measured during the test, to remove the effects of sample disturbance

(4) <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.

10.3.3 Methods

(1) <RCM> The measurement of shear modulus and damping ratio should be performed using any of the following methods:

- Resonant Column Test (RC); or
- Cyclic Torsional Shear Test (CTST); or
- Cyclic Direct Simple Shear Test (CDSST).

(2) <PER> To obtain a shear modulus from Cyclic Triaxial Tests (CTxT) a constant value of Poisson ratio may be assumed.

NOTE 80. Poisson's ratio can vary with strain.

10.4 Small strain moduli and seismic velocities

10.4.1 General

(1) <RCM> For seismic design in moderate and high seismicity classes, the small strain shear modulus G_0 should be determined directly using in situ geophysical measurements of the velocity of propagation of shear waves (see EN 1998-5:202x clause 6.1(8)), using Formula 10.1:

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

where:

ρ is the unit mass density of the soil; and

V_S is the shear wave velocity.

(2) <PER> For dry soils and rocks, the shear wave velocity may be estimated from the measured compressional wave velocity and an assumed value of Poisson ratio according to Formula (10.2):

$$G_0 = \frac{1-2\upsilon}{2(1-\upsilon)} \rho V_P^2 \tag{10.2}$$

where:

υ is the Poisson ratio; and

ρ is the mass density of the soil; and

V_P is the compressional wave velocity.

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

(4) <PER> Wave propagation velocities determined on lab specimens may be used to assess their representativeness and disturbance with respect to the material in its natural state at the site scale.

(5) <RCM> The reported value of shear wave velocity should be accompanied by an estimate of its uncertainty due to the quantity and quality of site-specific data and to the spatial variability of the property.

10.4.2 Geophysical Methods

(1) <RCM> Elastic wave velocities should be determined directly using one or more of the in situ geophysical methods listed in Table 10.2.

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

Test	***Acronym***	***Test Standard***	V_S	V_P
Cross-Hole Test	*CHT*	*ASTM D4428 / D4428M - 14*	X	X
Down-Hole Test	*DHT*	*ASTM D7400 - 19*	X	X
P-S suspension logging test	*P-S Log*	-	X	X
Seismic Cone Penetration Test	*SCPT*	-	X	X
Seismic Flat Dilatometer Test	*SDMT*	-	X	X
Surface Wave Methods	SWM	-	X	
Seismic Refraction	*Refr*	*ASTM D5777-18*	X	X
Seismic Reflection	*Refl*	*ASTM D7128 - 18*		X

NOTE 81. Surface wave methods (SWM) include all the geophysical method which are based on the spectral analysis of the propagation of surface (Rayleigh, Love or Stoneley) waves such as SASW (Spectral Analysis of Surface Wave), MASW (Multistation Analysis of Surface Waves), CSSW (Continuous Source Surface Wave), AVA (Ambient Vibration Analysis).

(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) <PER> When a testing standard is not available the measurement and interpretation may follow well-established guidelines.

NOTE 82. For example, guidelines for measurements of elastic wave velocity in rod-push methods (SCPT and SDMT) are available in: Butcher A.P., Campanella R.G., Kaynia A.M., Massarsch K.R. (2005) Seismic cone downhole procedure for measuring shear wave velocity - a guideline prepared by ISSMGE TC10: Geophysical Testing in Geotechnical Engineering. Proceedings of the XVI International Conference on Soil Mechanics and Geotechnical Engineering. ISBN-10: 9059660277.

NOTE 83. For example, guidelines for surface wave methods (SWM) are available in: Foti, S., Hollender, F., Garofalo, F. et al. Bull Earthquake Eng (2018) 16: 2367. https://doi.org/10.1007/s10518-017-0206-7

(4) <RCM> The CHT should be used whenever a very high resolution and accuracy is necessary also at large depth for the specific design situation.

(5) <REQ> The CHT interpretation 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.

(6) <REQ> The P-S suspension logging shall not be used whenever the properties at very shallow depth (less than 5m) are necessary for the specific design situation.

(7) <REQ> Uncertainties associated to solution non-uniqueness shall be taken into account and quantified in surface wave methods and seismic refraction surveys.

(8) <REQ> The seismic refraction survey shall not be used whenever the stratigraphic conditions are such that a reduction of velocity with depth is expected.

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

10.4.3 Indirect derivation

(1) <PER> For structures in Geotechnical Category 1 or seismic design situations involving very low to seismicity seismicity (see EN 1998-5:202x, 6.1(9)), the shear wave velocity may be determined indirectly using transformation models for any of the tests listed in Table 10.3.

Table 10.3 – Tests for indirect determination of shear wave velocity of soils

Test	Test Standard	See also
CPT	EN ISO 22476-1	Annex I
SPT	EN ISO 22476-3	Annex I
PMT	EN ISO 22476-4	
DMT	EN ISO 22476-7	Annex I

(2) <PER> Indirect determination with 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> The transformation models for the indirect derivation of the shear wave velocity from empirical correlations shall comply with 6.7(10).

NOTE 84. Examples of transformation models are reported in Annex I.

(4) <REQ> The transformation models for blow counts (SPT) and penetration resistance (CPT) shall take into account the:

– type of soil or the grain size distribution; and

- in situ state of stress or the depth at which the measurement is taken.

(5) <RCM> The transformation models for SPT blow counts (N) and cone penetration resistance (qc) should also take into account the geological age of the soil deposit.

(6) <RCM> The transformation models for SPT blow counts (N) should not be used for fine soils.

(7) <REQ> The uncertainty associated to the estimate shall be evaluated taking into account the:

- uncertainty associated to the measured value in the test; and
- random error and bias inherent to the transformation model.

10.5 Excess pore pressure

(1) <RCM> The development of excess pore pressure during cyclic loading should be investigated in the laboratory, on specimens obtained from samples of Sample Quality Class 1, using any of the following tests:

- Cyclic Torsional Shear Test (CTS); or
- Cyclic Direct Simple Shear Test (CDSS); or
- Cyclic Triaxial Test (CTx).

(2) <PER> Reconstituted samples may be used if undisturbed sampling is not feasible.

(3) <RCM> Samples should be reconstituted by a technique that mimics the genesis of the soil deposit. Wet tamping or water deposition should be preferred for sedimentary soils as a dry deposition technique may lead to an overestimation of the excess pore pressure.

(4) <PER> When laboratory tests are not feasible, the excess pore pressure may be determined indirectly by transformation models based on empirical correlations.

(5) <REQ> Indirect methods shall comply with 6.7(4).

(6) <REQ> Indirect methods shall account for:

- type of material; and
- plasticity index and the overconsolidation ratio for clays or the relative density for sands; and
- confining pressure; and
- expected level of shear strains in the soil; and
- expected number of equivalent cycles.

10.6 Cyclic shear strength

10.6.1 General

(1) <REQ> Cyclic shear strengths shall be expressed as the number of cycles required to attain a cyclic strength limit for a given combination of average shear stress τa and cyclic stress τcyc.

(2) <REQ> Cyclic strength limits shall be associated to 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.

10.6.2 Cyclic undrained shear strength of coarse soils

(1) <RCM> The cyclic undrained resistance of coarse soils should be determined with large strain cyclic tests on specimens obtained from samples of Sample Quality Class 1 using any of the following tests:

- Cyclic Torsional Shear Test (CTS); or
- Cyclic Direct Simple Shear Test (CDSS); or
- Cyclic Triaxial Test (CTx).

(2) <PER> Reconstituted samples may be used if undisturbed sampling is not feasible.

(3) <RCM> Samples should be reconstituted by a technique that mimics the genesis of the soil deposit. Wet tamping or water deposition should be preferred for sedimentary soils as dry technique may lead to an underestimation of the cyclic strength.

(4) <RCM> The reconstituted samples should be pre-sheared to restore the pre-shearing state during the event, through cyclic shear with drainage.

NOTE 85. This procedure simulates the history of small earthquakes and it can also reduce the effect of the sample preparation method

(5) <RCM> For coarse sands and gravels, the interpretation of the test should account for the membrane penetration which could modify the development of excess pore pressure and hence the cyclic undrained shear strength.

(6) <PER> When laboratory tests are not feasible the resistance to cyclic liquefaction may be determined indirectly using transformation models based on empirical correlations with the results of in situ tests (see EN 1998-5:200x, 7.3.3).

(7) <REQ> Empirical correlations with field tests shall account for the confining pressure and the fine content and shall comply with 6.7(4).

10.6.3 Cyclic undrained shear strength for fine soils

(1) <RCM> The cyclic undrained shear strength of fine soils should be evaluated on specimens obtained from samples of Sample Quality Class 1 using any of the following tests: (Quality class …)

- Cyclic Direct Simple Shear Test (CDSS); or
- Cyclic Triaxial Test (CTx).

(2) <RCM> The possible degradation of the shear strength caused by structural disturbance during cyclic loading should be investigated with cyclic laboratory tests on specimens obtained from undisturbed samples,

(3) <PER> The possible increase of undrained shear strength for dynamic loading may be taken into account.

(4) <RCM> The stress path imposed during the test should reproduce as much as possible the effective stress state relevant for the structure in question and the expected cyclic loading.

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

(6) <PER> The values of undrained shear strength derived for static conditions (see Clause 8.3) may be used for dynamic loads if cyclic degradation effects are expected to be negligible.

10.6.4 Shear strength on discontinuities

(1) <RCM> Cyclic shear tests should be carried out on natural discontinuities to estimate the shear strength decrease due to the damage effect induced on their asperities by the cumulative shear action in order to estimate the rock mass strength under seismic action.

10.7 Additional parameters for seismic site response evaluation

10.7.1 Depth to seismic bedrock

(1) <RCM> The depth of the conventional seismic bedrock H800, identified by a shear wave velocity VS larger than 800m/s, should be determined as part of the evaluation of the shear wave velocity profile as required for site categorisation according to EN1998-1:202x, 5.1.2.

(2) <RCM> Whenever it is possible, the position of the seismic bedrock should be determined directly as part of the evaluation of the shear wave velocity profile with the ground investigation methods given in 10.3.1.

(3) <PER> The position of the seismic bedrock may be determined directly through a seismic refraction survey or a seismic reflection surveys based on the propagation of compressional wave velocities if a suitable contrast in compressional wave velocity is expected against the above soil column, also accounting for the effect of saturation.

(4) <PER> The depth of the bedrock formation H800 may be estimated indirectly from the result of a single-station horizontal-to-vertical spectral ratio survey (see 10.7.2 (3)) according to Formula 10.3:

$$H_{800} = \frac{V_{S,H800}}{4f_0} \tag{10.3}$$

where:

f_0 is the natural frequency of the soil deposit; and

$V_{S,H800}$ Is the equivalent value of V_s down to the reference depth H_{800}.

10.7.2 Fundamental frequency of the soil deposit

(1) <RCM> The fundamental frequency of the soil deposit should be determined as an additional parameter for the calibration of models for seismic site response analyses and for site categorisation according to EN1998-1:202x, 5.1.2.

(2) <PER> The fundamental frequency of the soil deposit may be determined directly from a single-station Horizontal-to-Vertical Spectral Ratio (HVSR) survey

NOTE 86. Guidelines for Horizontal-to-Vertical Spectral Ratio (HVSR) survey are available in: SESAME team (2004).

(3) <PER> The fundamental frequency f_0 of the soil deposit may be calculated from the shear wave velocity profile accounting for the position of the seismic bedrock using Formula 10.4:

$$f_0 = \frac{V_{S,H800}}{4H_{800}} \tag{10.4}$$

where:

H_{800} is the depth of the bedrock formation identified by a shear wave velocity V_S > 800 m/s; and

$V_{S,H800}$ is the equivalent value of V_s down to the reference depth H_{800}.

11 Groundwater and hydraulic conductivity

11.1 General

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

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

- the depth, thickness, extent and conductivity of water-bearing strata in the ground;
- joint systems in the rock;
- the permeability or hydraulic conductivity of each geotechnical unit;
- the piezometric head of aquifers and their variation over time;
- actual piezometric heads including possible extreme levels and their periods of recurrence;
- the piezometric pressure distribution;
- the chemical composition and temperature of groundwater.

(3) <REQ> The following shall be specified for laboratory tests, 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 under which the specimen is to be tested;
  - the need for back-pressure and 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 need for back-pressure and the required degree of saturation.

(4) <REQ> The reference level and height system shall be reported together with test data.

(5) <REQ> Groundwater measurements shall be planned, conducted and reported in accordance with EN ISO 18674-4.

NOTE 87. Relevant considerations can include the topography, stratigraphy, ground conditions and especially the conductivity of the ground or identified aquifers.

(6) <REQ> For monitoring projects, piezometric measurements shall be located where groundwater conditions are expected to change.

NOTE 88. Monitoring is commonly used during groundwater lowering, excavation, filling, and tunnelling.

(7) <RCM> For reference purposes in monitoring projects, measurement of the natural fluctuations in groundwater should be made outside the area affected by the actual project.

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

(9) <REQ> Geohydraulic testing shall be planned and reported in accordance with all parts of EN ISO 22282.

(10) <REQ> The evaluation of groundwater measurements shall take into account:

- geological and geotechnical conditions of the site;
- accuracy of individual measurements;
- natural fluctuations of pore water pressures with time;
- duration of the observation period;
- season of measurements;
- nearby standing water;
- climatic conditions during and before that period.

11.2 Piezometric pressure and piezometric head

11.2.1 General

(1) <REQ> Piezometric pressure shall be assessed according to this clause and according to EN ISO 18674-4.

(2) <RCM> Piezometric pressure should be measured using one or both of the following systems:

- open systems (open standpipe and open pipe with an inner hose)
- closed systems.

NOTE 89. The use of filter tips connected to a small diameter hose in open systems can decrease the response time.

(3) <REQ> The type of equipment to be used for piezometric measurements shall be selected according to:

- the type and conductivity of the ground;
- the purpose of the measurements;
- the required observation period;
- the expected groundwater fluctuations;
- the response time of the equipment and ground.

(4) <RCM> Open systems should be used in the ground with high conductivity (aquifers and aquitards).

NOTE 90. With soils and rocks of low conductivity, the use of open systems often leads to erroneous interpretations, due to the length of response time.

NOTE 91. Ground with high conductivity includes, for example, coarse soil and highly jointed rock.

(5) <RCM> Closed systems should be used in the ground with low conductivity and aquicludes.

NOTE 92. Ground with low conductivity includes, for example, fine soils and lightly jointed rock.

(6) <RCM> Closed systems should be used to measure artesian water pressure.

(7) <PER> Piezometric head may be obtained from piezometric pressure using the Formula (11.1):

$$h_w = \frac{p_w}{\gamma_w} \tag{11.1}$$

where:

p_w is the piezometric pressure; and

γ_w is the weight density of water.

NOTE 93. The value of γ_w can be taken as 10 kN/m^3 if a specific value has not been established by measurement.

NOTE 94. The need for an accurate value of weight density depends on the measured pressure at the filter level. For water pressure up to 100 kPa, significant deviation is 2,5 % and above 100 kPa significant deviation is 1 %.

11.2.2 Test results

(1) <REQ> In closed systems, correction due to atmospheric pressure shall be made when deriving piezometric pressure from measurements.

(2) <REQ> The atmospheric pressure shall be recorded at the time of the reading whenever corrections are to be made.

(3) <REQ> Continuous recording shall be used when very short-term variations or fast pore water fluctuations are to be monitored.

NOTE 95. The continuous recording includes the use of transducers and data loggers.

(4) <REQ> To obtain piezometric pressure at a specific location in the ground, the measuring point shall be sealed off from other ground units, aquifers and surface water, as specified in EN ISO 22475-1.

(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 piezometric pressures to come into equilibrium.

NOTE 96. Under some circumstances, the time needed to reach equilibrium is long and even never reached.

(6) <RCM> The reading interval should be adjusted after an initial period with adjustments based on actual variations of the observed readings.

(7) <REQ> To assess piezometric pressure fluctuations, measurements shall be taken at intervals much shorter than the natural fluctuations observed.

(8) <RCM> For persistent design situations, measurements should be performed throughout at least twice the natural period of fluctuations.

(9) <PER> For transient design situations, a single measurement verified by local experience may be used.

(10) <RCM> For initial conditions in triaxial and shear tests, the uncertainty of the derived test parameter should take into account the uncertainty of the piezometric pressure.

11.2.3 Applicability

(1) <REQ> The measuring method for determining the piezometric head shall be chosen considering the ground conditions and required accuracy.

NOTE 97. Guidelines on the required accuracy for different design situations and geotechnical structures are given in EN 1997-3.

NOTE 98. Guidelines for suitable measuring methods depending on ground conditions and the anticipated level of uncertainty are given in Table 11.1.

NOTE 99. The precision can be used as an estimate of the uncertainty due to the quality of measuring method when determining the characteristic value according to EN 1997-1 4.3.2.

NOTE 100. Uncertainty in the measured piezometric pressure depends on ground type and response time.

Table 11.1 – Precision of piezometric pressure measurements in different ground conditions

Method	**Ground**					
	Rock			**Soil**		
	Igneous	**Sedimentary**	**Metamorphic**	**Very Coarse**	**Coarse**	**Fine**
Open standpipe	10 kPa	5 kPa	10 kPa	1 kPa	1 kPa	-
Open standpipe with seal	1 kPa	1 kPa	1 kPa	1 kPa	1 kPa	-
Piezometric cone	-	-	-	-	0,1 kPa	0,1 kPa
Piezometric cone with seal	0,1 kPa	0,1 kPa	0,1 kPa	0,1 kPa	0,1 kPa	0,1 kPa
CPTU	-	-	-	-	5 kPa	-
DPT	-	-	-	1 kPa	1 kPa	-
Observation while drilling	20 kPa	20 kPa	20 kPa	5 kPa	20 kPa	-

(2) <RCM> During the drilling process, water levels in boreholes should be recorded at the end of the day and at the start of the following day (before drilling is resumed).

<To be checked against 22475 when published>

(3) <RCM> Any sudden inflow or loss of water during drilling should be recorded.

(4) <RCM> Any loss or recovery of drill flush returns during drilling should be recorded.

(5) <REQ> The casing depth in the borehole before and after installation of a piezometer shall be recorded.

11.2.4 Reporting

< To be checked against 18674-4 when published>

(1) <REQ> The date and time of all observations shall be recorded.

(2) <RCM> Measured values should be presented in a graph (measured value vs time).

(3) <REQ> The evaluated results of piezometric pressure measurements shall comprise the observed maximum and minimum piezometric levels and the corresponding measuring period.

(4) <REQ> Where standing water is situated within the investigated area, the water level shall be reported.

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

11.2.5 Direct evaluation

11.2.5.1 Open standpipe

(1) <REQ> Tests in open standpipes shall be performed and reported according to EN ISO 22475-1.

(2) <RCM> Measurement of the piezometric head should be made manually by an electrical or mechanical contact gauge.

(3) <PER> Measurement of piezometric pressure may be made by an electric, pneumatic, or hydraulic system.

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

NOTE 101. To ensure the measurement is representative for the conditions along with the filter and filter pack.

(5) <REQ> The accuracy of open standpipe measurements shall be evaluated based upon the method of reading and the system components.

(6) <RCM> The depth and length of the slotted filter and sand filter shall be chosen according to the in situ hydraulic conditions, discontinuities, and the corresponding ground property variation.

(7) <RCM> The weight density of the standing water in the pipe should be reported if its value differs more than 2 % to the pore water density.

11.2.5.2 Piezometric cone

(1) <REQ> Piezometric cone tests shall be performed and reported according to EN ISO 18674-4.

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Constraints

Accuracy

(2) <REQ> The accuracy of piezometric cone tests shall be evaluated based upon EN ISO 18674-4, according to the equipment used.

Measured value to derived value

Reporting

11.2.6 Indirect evaluation

11.2.6.1 Cone penetration test - CPTU

(1) <REQ> Piezocone tests (CPTUs) shall be performed and reported according to EN ISO 22476-1.

(2) <REQ> Tests shall be performed with a CPTU cone assembled with pore pressure sensors, test type TE2, as defined in EN ISO 22476-1.

<To be checked with 22476-1 after revision of CPTU-standard>

(3) <REQ> Measurements in a geotechnical unit shall be performed over a length larger than 2 m.

(4) <REQ> The soil behaviour type shall be verified to be drained, with hydrostatic pore pressure over measured length.

(5) <REQ> Accuracy shall be evaluated depending on the CPTU cone used according to EN ISO 22476-1.

(6) <POS> During soundings, the piezometric head in a geotechnical unit may be evaluated on the assumption of a constant increase in pore pressure of 10 kPa/m.

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

11.3 Hydraulic conductivity

11.3.1 General

(1) <RCM>The evaluation of hydraulic conductivity should assess:

- 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;
- how well these conditions match the situation in the field.

(2) <REQ> The following items shall be considered when determining the coefficient of conductivity of a geotechnical unit:

- the preferred test type for conductivity determination;
- the orientation of the test;
- the need for additional classification tests.

NOTE 102. Further information on a procedure, presentation and evaluation of the conductivity test can be found in EN ISO 17892-11, (see X.4.7).

11.3.2 Test results

(1) <RCM> It should be checked that the volume changes due to the consolidation of the tested volume only negligibly affect the measured conductivity.

(2) <RCM> The temperature corrections specified in EN ISO XXXX should be applied.

(3) <PER> The coefficient of conductivity may be computed from the test data assuming that Darcy's law is valid.

11.3.3 Applicability

(1) <REQ> It shall be checked that the gradient in the laboratory test and the gradient in situ lie within a laminar flow.

(2) <RCM> For laboratory conductivity tests on fine or organic soil, only soil specimens of Quality Class 1 should be used.

<To be checked against 22475-1 after revision>

(3) <PER> For laboratory conductivity tests on coarse soils, specimens of Quality Class 2 and reconstituted soil specimens may be used.

11.3.4 Reporting

<To be checked against 22475-1 after revision>

11.3.5 Direct evaluation

11.3.5.1 Determination of permeability by constant and falling head

(1) <REQ> Determination of permeability by constant and falling head tests in the laboratory shall be performed and reported according to EN ISO 17892-11.

11.3.5.2 Water permeability tests in a borehole using open systems

(1) <REQ> Water permeability tests in a borehole using open systems in the field shall be performed and reported according to EN ISO 22282-2.

11.3.5.3 Water pressure tests in rock

(1) <REQ> Water pressure tests in rock in situ shall be performed and reported according to EN ISO 22282-3.

11.3.5.4 Infiltrometer tests

(1) <REQ> Ring infiltrometer tests in situ shall be performed and reported according to EN ISO 22282-5.

(2) <PER> In situ measurements using the constant head injection test in low permeability rocks may be performed and reported according to ASTM D4630-96.

NOTE 103. Also known as Lugeon type test.

11.3.5.5 Water permeability tests in a borehole using closed systems

(1) <REQ> Water permeability tests in a borehole using closed systems in situ shall be performed and reported according to EN ISO 22282-6.

11.3.6 Indirect evaluation

11.3.6.1 Pumping tests

(1) <REQ> Pumping tests in situ shall be performed and reported according to EN ISO 22282-4.

11.3.6.2 Dissipation test - DPT

Test requirements

(1) <REQ> Dissipation tests shall be performed and reported according to EN ISO 22476-1.

(2) <REQ> Test shall be performed with a CPTU cone assembled with pore pressure sensors, test type TE2.

(3) <REQ> Accuracy shall be evaluated depending on used CPTU cone according to EN ISO 22476-1.

(4) <REQ> Measurements shall be performed until 50 % of excessive porewater has dissipated

(5) <REQ> Accuracy shall be evaluated depending on used CPTU cone according to EN ISO 22476-1.

(6) <REQ> Soil behaviour type shall be reported together with the test result.

11.3.7 Empirical rules

(7) <Drafting NOTE: text still under development>

(1) <POS> Derivation of empirical hydraulic conductivity can be based upon grain size distribution and relative density for coarse and very coarse soil.

12 Thermal properties

<Drafting note>

Knowledge of the thermal properties of rock and soil is valuable in many different areas. Equipment for the analysis of thermal conductivity is developed. Laboratory or in situ determinations of thermal properties can be performed under stationary and transient conditions by several different methods. Two kinds of probe methods, the single-probe and the multi-probe method exist. Theory and different sources of potential errors, for instance, length/diameter ratio and influence of sample boundary, will be treated.

Knowledge of the thermal transport properties of rock and soil is valuable in many different areas. Some examples are the utilization and storage of ground heat, geothermal heat flow determinations and determinations of heat loss from buried cables and pipelines.

The thermal properties of a material depend on several properties, some of which can be time-dependent. The thermal conductivity of crystalline rock is mainly influenced by the following factors:

- mineral composition
- temperature
- isotropy/anisotropy
- fluid/gas in micro-fissures

Quartz has a thermal conductivity several times higher than that of other common rock-forming minerals. The quartz content is, therefore, an important factor. The thermal conductivity of rock decreases as the temperature increases.

If the texture of the rock is anisotropic, thermal conductivity is a function of the direction of the heat flow. If the micro-fissures in the rock are filled with air instead of water, the thermal conductivity decreases rapidly with small crack porosity (< 1%). At a larger scale, the ordinary cracks also influence heat transport.

In addition to the above-mentioned factors, the thermal conductivity of soil and sedimentary rock is a function of the porosity and the degree of water saturation.

Thermal conductivity decreases as porosity increases. Moreover, thermal conductivity sharply falls when the degree of saturation is below approximately 50%. At unsaturated conditions and above room temperature, vapour diffusion and radiation become more important with increasing temperature. Both these heat transport mechanisms can be added to the thermal conductivity and form an effective thermal conductivity as a function of temperature.

Measurement of thermal conductivity can be classified as in situ measurement and laboratory measurements. In situ measurements are performed at natural and undisturbed conditions. One problem at in situ measurements is to know how representative the measurement is due to natural changes in, e.g. water content. If a proper evaluation can be made on such time-dependent variables, in situ measurements are, in general, preferable.

Laboratory measurements comprise a smaller sample volume. The result of such measurements is reliable, provided the following points are fulfilled:

- the sample is undisturbed
- the sample volume is representative of the soil/rock
- the volume affecting the measurements is representative of the sample

- correction is made for temperature differences between laboratory and field
- correction is made for other time-dependent variables (e.g. water content)

Calculations of the thermal conductivity of ground materials from volume fractions of minerals, pore gas and pore fluid offer many advantages. Knowing the changes in, e.g. temperature and water content, it is possible to calculate the change in thermal conductivity. Estimates can be made from the result of a ground investigation. An analysis of the sensitivity of the thermal conductivity can be made from possible variations in the volume fractions.

<End of drafting note>

12.1 General

(1) <REQ> Ground investigations of thermal properties shall provide relevant information needed for geothermal design and construction.

(2) <RCM> Ground investigations for thermal engineering should provide information on:

- geological conditions;
- hydrogeological conditions;
- geotechnical conditions;
- hydrochemical conditions;
- geothermal conditions.

NOTE 104. This clause mainly covers geothermal conditions.

(3) <REQ> The reference level and height system shall be reported together with test data.

(4) <REQ> The method to be used for thermal measurements shall be selected according to:

- the type and 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.

(5) <REQ> The evaluation of thermal measurements shall take into account:

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

NOTE 105. Known sources of error include: non-radial heat flow; radius and material vs time of measuring; variation in heat input; non-constant temperature; sample boundary effects; not only conduction; potential error in water-saturated coarse material.

12.2 Frost susceptibility

(1) <REQ> The risk of frost heaving shall be determined directly from laboratory tests on natural, recompacted and reconsolidated, or reconstituted samples (see Table 12. 1 and (2)) or indirectly be estimated from correlation with soil classification properties (particle size distribution, height of capillary rise (according to EN 1097-10:2014) and/or fines content).

NOTE 106. 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 107. 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.

Table 12. 1 — Laboratory tests to determine frost susceptibility properties

Parameter/ property to be determined	***Test to measure property***	***Test standard***	***MQC***	***Comments as to suitability and interpretation***
frost susceptibility	*Particle size analysis*	*TRV 2011:072 TDOK 2011:264.*	*2*	
resistance to freezing and thawing	*10 cycles of freezing at -17,5°C and thawing at 20°C under water*	*EN 1367-1 NF P98-234-2*	*2*	*for road materials: coarse soils, rock materials and aggregates*

(2) <RCM> If the estimation of frost susceptibility based on classification properties (fine or clay content) of the soil does not clearly indicate the absence of risk of frost heaving, frost heaving tests in the laboratory should be run.

NOTE 108. Examples of soil types indicating the need of laboratory tests in addition to correlations to classification properties include organic soil, peat, saline soil, artificial soil and coarse soil with a wide range of grain size.

(3) <RCM> To determine the frost susceptibility of soil in its natural state, intact samples should be tested of at least Quality Class 2. To estimate the frost susceptibility of a constructed fill, frost heave tests should be run on recompacted and then reconsolidated specimens or on reconstituted specimens.

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

(5) <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> Thermal conductivity should be determined by using at least one of the methods given in Table 12.2.

Table 12.2 — Methods for determining thermal conductivity

Method	Applicable for determining property	Comment
Multi-probe method	Thermal conductivity Thermal diffusivity	Transient field and laboratory method. Applicable to rock and soil.
Single-probe method (needle-probe)	Thermal conductivity (Thermal diffusivity)	Transient field and laboratory method. Applicable to rock and soil.
Divided-bar method	Thermal conductivity	Stationary laboratory method. Applicable to rock.
THS-method (Transient hot strip)	Thermal conductivity Thermal diffusivity	Transient laboratory method. Applicable to rock, fluid, (soil).
Theoretical calculation	Thermal conductivity Heat capacity	Calculation from rock mineral content and soil mineral content, porosity and water content.

(2) <RCM> Thermal conductivity, by Geothermal Response Test in a borehole, should be determined in soil and rock using a borehole heat exchanger according to EN-ISO 17628.

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

NOTE 109. ASTM D5334 test method presents a procedure for determining the thermal conductivity (λ) of soil and soft rock using a transient heat method.

NOTE 110. ASTM D5334 test method 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> Specific Heat Capacity may be determined in soil and rock according to ASTM D4611.

NOTE 111. The value of specific heat depends upon chemical or mineralogical composition and temperature.

NOTE 112. 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> Thermal diffusivity of rock and soil may be calculated according to ASTM D4612.

NOTE 113. In order to use ASTM D4612 test method for determination of the thermal diffusivity, the parameters (κ, ρ, c_p) is to be determined under as near identical specimen conditions as possible.

13 Reporting

13.1 Ground Investigation Report

13.1.1 General

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

(2) <REQ> The Ground Investigation Report shall form a part of the Geotechnical Design Report.

(3) <REQ> The Ground Investigation Report shall consist of:

- a factual account of all ground investigation activities carried out
- a presentation of all appropriate geotechnical information including geological features and relevant data;
- a geotechnical evaluation of the information, stating the assumptions made in the interpretation of the test results.

(4) <RCM> The Ground Investigation Report should include the contents listed in Annex I.

DRAFTING NOTE – it is not logical for the ToC of the GIR to be in 1997-1. It is identified here that this would be better moved to an Annex in 1997-2, as presented here. It is left for PT6 to make the decision on this with deletion as appropriate to avoid repetition.

Further, the ToC given in 1997-1 is a <PER> – this should be upgraded to an "<RCM> as appropriate".

(5) <PER> The information may be presented as one report or as separate parts.

NOTE 114. Separate reports are generally prepared for each phase of ground investigation and each defined activity undertaken by an individual or organisation.

(6) <PER> The Ground Investigation Report may include derived values.

(7) <REQ> The Ground Investigation Report shall state known limitations of the results.

(8) <RCM> The Ground Investigation Report should identify gaps in knowledge about the site.

(9) <RCM> The Ground Investigation Report should propose necessary further field and laboratory investigations.

13.1.2 Presentation

(1) <REQ> The results of the desk study and the site inspection shall be presented identifying the information gained and its sources.

(2) <REQ> The observations made in the inspection of the site and surrounding area shall be presented identifying the persons carrying out the inspection, the dates of any site visits and any access limitations.

(3) <REQ> The presentation of geotechnical information shall include a factual account of all field and laboratory investigations.

(4) <REQ> The presentation shall include description of the ground encountered including fills, soils, rocks and discontinuities.

(5) <REQ> The presentation of geotechnical information shall document the methods used to carry out the work including, as appropriate, reports of:

- geophysical surveys or measurements;
- field investigations, such as sampling and field tests;
- records of readings of installations;
- groundwater observations and measurements;
- laboratory tests.

(6) <REQ> The results of the field and laboratory investigations shall be presented and reported according to EN ISOs 14688, 14689, 22475, 22476, 22477, 22282, 17892 and 18674.

(7) <RCM> The presentation should include the information given in Annex I.

13.1.3 Evaluation of geotechnical information

(1) <REQ> The evaluation of the geotechnical information shall:

- review the results of the field investigations and laboratory tests according to Clauses 5 to 12;
- describe the strata encountered and their geometry;
- for each stratum, identify the physical properties and the deformation and strength characteristics, referring to the results of the investigations and taking into account the ground type, sampling method, transport, handling and specimen preparation;
- review the derived values of geotechnical parameters;
- interpret the ground and groundwater conditions taking into account the ground type, the drilling method, and the period of readings;
- confirm and update the strata subdivision assumed from desk studies and site inspections in light of the results obtained;
- review any limitations in the data (including defective, irrelevant, insufficient, or inaccurate results) including the effects of inappropriate handling or long storage of samples;
- comment on irregularities such as cavities and zones of discontinuous material;
- propose further field or laboratory work, with justification and with specific reference to the questions that have to be answered in the ground model.

(2) <RCM> In addition, the evaluation of the geotechnical information should include:

- tabulation and graphical presentation of the results of the field and laboratory work in relation to the requirements of the project;
- depth of the groundwater table and its fluctuations (daily, seasonal, or longer periods);
- subsurface profiles showing the differentiation of the various units;
- the range and any grouping of measured and derived values of the geotechnical data for each stratum.

(3) <RCM> Significant variations in geotechnical parameters (weaker or stronger) should be identified.

NOTE 115. Averaging can mask the presence of a weaker zone or zones and should be used with caution.

(4) <RCM> The evaluation of each parameter should include comparison of the results with other laboratory and field tests measuring, or achieving by derivation, the same parameter and comparison with experience.

(5) <RCM> Special consideration should be given to apparently anomalous or outlier results for a parameter.

(6) <PER> A sequence of fine layers with differing composition or mechanical properties may be considered as one stratum if the overall behaviour can be adequately represented by averaged ground parameters.

(7) <PER> The boundaries between geotechnical units and groundwater levels may be interpolated linearly between the investigation points provided the spacing is sufficiently small and the geological conditions are sufficiently homogeneous.

(8) <RCM> The methods of interpolating the boundaries of ground layers and the groundwater levels should be reported and justified.

(9) <RCM> Consideration shall also be given to the influence of chemical degradation on the performance of the soil, the rock or the materials used in construction.

(10) <RCM> The sensitivity to environmental factors (including climate, weathering, stress changes, water content changes) shall be considered in assessing the ground quality.

13.2 Establishment of derived values

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

(2) <REQ> Where correlations have been used to derive geotechnical parameters or coefficients, the correlations and their applicability shall be documented.

(3) <RCM> Parameters arrived at by measurement and by derivation from other tests should be compared for consistency and critically reviewed where there are differences.

Annex A

(informative)

Suitability of test methods

A.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in Clauses 4-6 regarding suitable methods of test in investigation.

NOTE 116. 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.

A.2 Scope and field of application

(1) This informative Annex covers identification of the suitability of test methods given in EN 1997-2.

A.3 Suitable 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 117. An indication of the suitability of the test methods covered by EN 1997-2 is given in Table A.1.

<Drafting NOTE: top half of table might be best in 1997-3 not here – to be reviewed>

Table A.1 – Guidance on appropriate methods of ground investigation

Proposed structures/ engineering works and ground information needed (H = most necessary)		4.2.1 Desk study - History of site	4.2.2 Site reconnaissance - Ground features and geomorphology	4.2.2 Sequence of strata / ground profile	4.2.5 Description of soils and rocks	4.2.6 Description of discontinuities in soils and rocks	7 Physical properties	7 Chemical properties	8 Material strengths	9 Material stiffnesses	10 Dynamic properties	11 Groundwater conditions	11 Ground permeability	12 Thermal properties	Presence of voids (natural or man-made)	Properties for materials re-use	Contaminated ground	Chemical properties/ aggressive ground
	EN 1997-3 clauses (content to end up in Part 3 clauses?)																	
	4 Slopes	M	H	H	H	H	H	L	H	L	L	H	H	L	M	L	L	L
	4 Cuttings	M	H	H	]	H	H	L	H	M	L	H	H	L	H	H	L	L
	4 Embankments	H	H	H	H	H	H	L	H	H	H	M	M	L	M	M	L	L
	5 Spread foundations	H	M	H	H	M	H	M	M	H	L	M	L	L	H	L	M	L
	6 Piled foundations	H	H	H	H	M	H	M	M	H	L	M	L	L	H	M	M	M
	7 Retaining structures	H	H	H	H	H	H	L	H	H	M	H	H	L	M	L	M	H
	8 Anchors	M	L	H	H	H	H	H	H	H	M	H	H	L	H	L	M	H
	9 Reinforced soil structures	M	M	H	M	M	H	H	H	H	M	H	H	L	M	M	M	H
	10 Ground improvement	H	H	H	M	M	H	M	H	H	H	H	H	M	H	L	H	H
	EN 1997-2 miscellaneous clauses (existing Annex B)																	
	Linear - roads	M	M	M	M	M	H	L	M	M	M	M	L	L	M	H	L	M
	Linear pipelines	M	M	M	H	M	H	H	M	M	L	H	M	M	M	H	L	M
	Linear - tunnels	L	H	H	H	H	H	M	H	M	L	H	L	L	H	M	M	H
	Underground openings	L	H	H	H	H	H	M	H	M	L	H	L	L	H	M	M	M
	Dams and weirs	H	H	H	H	H	H	L	M	M	M	H	L	L	H	M	L	M
	Construction materials	L	L	M	H	M	H	H	M	M	L	H	L	L	L	H	H	H
	Ground source heat installations	M	M	H	M	H	L	L	L	L	L	H	H	H	M	L	L	L
Ground information needed (EN 1997-2 clauses) (content to act as linkage between Parts 2 and 3) →		4.2.1 Desk study - History of site	4.2.2 Site reconnaissance - Ground features and geomorphology	4.2.2 Sequence of strata / ground profile	4.2.5 Description of soils and rocks	4.2.6 Description of discontinuities in soils and rocks	7 Physical properties	7 Chemical properties	8 Material strengths	9 Material stiffnesses	10 Dynamic properties	11 Groundwater conditions	11 Ground permeability	12 Thermal properties	Presence of voids (natural or man-made)	Properties for materials re-use	Contaminated ground	Chemical properties/ aggressive ground

Appropriate methods of investigations (H = most and most relevant information)		~~Ground information needed~~ → (EN 1997-2 clauses) (content to act as linkage between Parts 2 and 3)	4.2.1 Desk study - History of site	4.2.2 Site reconnaissance - Ground features and geomorphology	4.2.2 Sequence of strata / ground profile	4.2.5 Description of soils and rocks	4.2.6 Description of discontinuities in soils and rocks	7 Physical properties	7 Chemical properties	8 Material strengths	9 Material stiffnesses	10 Dynamic properties	11 Groundwater conditions	11 Ground permeability	12 Thermal properties	Presence of voids (natural or man-made)	Properties for materials re-use	Contaminated ground	Chemical properties/ aggressive ground
	EN 1997-2 Clauses (content to end up in Part 2 clauses)																		
		4.2.1 Desk study		H	H	H	H	M	M	H	H	H	H	H	H	H	H	H	H
		4.2.2 Site reconnaissance	H		H	H	H	L	L	M	M	L	H	M	M	H	M	H	M
		4.2.2 Surface mapping (geomorphological, geological)	H		H	H	H	L	L	L	L	L	H	M	L	H	M	H	L
		4.2.2 Ground mapping - geophysical survey	M	H	H	N/A	N/A	L	L	M	M	H	H	M	L	H	N/A	H	L
		6 Boreholes without samples	M	H	H	N/A	N/A	M	L	H	H	H	H	H	M	H	N/A	L	N/A
		6 Trial pits	H	H	H	H	H	H	M	H	H	M	H	H	M	H	M	M	L
		6.1 Boreholes with sampling	H	H	H	H	H	H	H	H	M	M	L	M	M	H	M	H	L
		6.2 Probes (DP, CPT)	N/A	N/A	H	N/A	N/A	H	L	H	H	M	M	M	L	H	L	L	N/A
		6.2 Field testing	N/A	N/A	N	N/A	N/A	H	M	H	H	H	H	H	H	H	M	H	M
		6.2 Geophysics	L	L	H	N/A	N/A	H	L	H	H	H	M	M	M	H	L	M	L
		6.2 Laboratory testing	N/A	N/A	N/A	M	M	H	H	H	H	H	N	M	H	N/A	H	L	H
		6.3 Instrumentation and monitoring	N/A	L	N/A	N/A	N/A	L	L	M	M	M	H	H	M	N/A	N/A	N/A	L

Annex B

(informative)

Desk study and site inspection

B.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in Clause 4 regarding desk studies and site inspection.

NOTE 118. 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 desk studies and site inspection.

B.3 Desk studies

(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 detrimentally 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/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 (e.g. tunnels, pipes, cables) and mine workings;

    - the potential for contamination 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.

(4) <PER> Interpretation of the desk study may include, as appropriate:

- 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.
- Ground investigation:
    - 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.

B.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, and geological maps and remotely sensed;
- 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> 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, potential construction and access and environmental constraints for ground investigation.

(3) <RCM> Items to inspect during the site inspection should include:

- 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 indications of ground or gas contamination;
- ecological conditions;
- access routes 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 actions and observations as follows:

- traverse the whole area, preferably on foot;
- set out the proposed location of work on plans, where appropriate;
- observe and record differences and omissions on plans and maps; for example, boundaries, buildings, roads and transmission lines;
- inspect and record details of existing structures;
- observe and record obstructions;

NOTE 119. Obstructions can include transmission lines, ancient monuments, trees subject to preservation orders, manhole covers, gas and water pipes, electricity cables, sewers

- 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;
- observe and record mine or quarry workings, old workings, old structures, and any other features that might be relevant;
- observe and record any obvious immediate hazards to public health and safety (including to trespassers) or the environment;
- observe and record any areas of discoloured soil, polluted water, distressed vegetation or significant odours;
- observe and record any evidence of gas production or underground combustion;
- tree types and locations if site underlain by fine soils;
- 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; for example, till and glacial gravel. Similarly, hollows and depressions, locally water-filled, could also indicate former glacial conditions;
  - 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) <POS> The inspection can be usefully enhanced considerably by suitably referenced photographs.

(6) <POS> Inspection of the site for ground investigation purposes can 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;
- areas for depot, offices, sample storage, field laboratories;
- ownership of working sites, where appropriate;
- 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, as appropriate;
- particulars of local telephone including mobile phone reception, employment, transport and other services;
- surface conditions at each exploratory hole and the particular reinstatement requirements (e.g. breaking out of pavement and replacement);
- details of post investigation access to instrumentation and any requirements to protect the instrument (e.g. fencing).

Annex C

(informative)

Information to be obtained from ground investigation

C.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in Clause 6 regarding the information to be obtained from ground investigation.

NOTE 120. 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

(2) This Informative Annex covers the information to be obtained from ground investigation.

C.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 construction:

- the suitability of the site with respect to the proposed construction and the level of acceptable risks;
- the deformation of the ground caused by the structure or resulting from construction works, its spatial distribution and behaviour over time;
- the safety with respect to limit states, 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 construction:
    - the foundation construction methods;
    - the sequence of foundation works;
    - the effects of the structure and its use on the surroundings;
    - 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.

(2) <RCM> The information obtained from investigation for materials to be used in construction 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 soil and rock;
- the workability of soil and rock during construction and possible changes in their properties during transport, placement and further treatment;
- the effects of construction traffic and heavy loads on the ground;
- the prospective methods of dewatering and/or excavation, effects of precipitation, resistance to weathering, and susceptibility to shrinkage, swelling and disintegration.

(3) <RCM>the information obtained from investigations of groundwater conditions should include the following:

- the depth, thickness, extent and permeability of water-bearing strata 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 pore water pressure distribution;
- the chemical composition and temperature of groundwater.

(4) <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 (e.g. risk of hydraulic failure, excessive seepage pressure or erosion);
- any measures necessary to protect the structure (e.g. waterproofing, drainage and measures against aggressive water);
- the effects of groundwater
- 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.

Annex D

(informative)

Qualifications and professional experience

<Drafting note: This Annex was originally prepared by a group within ISSMGE and a revised included in the October draft of EN 1997-1 by SC7.PT2. Annex G in 1997-1 is here extended into the investigation personnel; coordination is required with the documents in preparations by 22475.>

D.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in EN 1997-1, 1.3 regarding the information to be obtained from ground investigation.

(2) This Informative Annex establishes one possible way for verifying the assumption that the design, collection and evaluation of information is performed by competent persons.

NOTE 121. 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 provides guidelines on requirements for competence of persons responsible for either geotechnical design or any aspect of the design, execution or evaluation of the ground investigation.

(2) This Informative Annex is intended to be used in conjunction with other national legislation that gives complementary requirements on competence.

D.3 Guidelines

(1) <RCM> The persons responsible for Ground Investigation and Geotechnical Design should have appropriate qualifications and experience within their respective field that includes:

- a university diploma demonstrating successful completion of tertiary studies in a relevant field;
- professional experience in ground engineering;
- Continuous Professional Development (CPD) in ground engineering;
- membership of a relevant Professional Register, if available or required in individual countries.

(2) <RCM> The persons carrying out or supervising the carrying out of sampling not in a borehole, data collection, measurement and testing (including field or laboratory technicians) should have appropriate training and experience for the work that they are carrying out that could typically include:

- school or college diploma in a relevant field;
- training in the work tasks being carried out;
- ongoing professional development and training in their area of work.

(3) <RCM> The persons carrying out sampling not in a borehole, data collection, measurement and testing (including field or laboratory technicians) under supervision should be suitably trained for the work that they are carrying out.

(4) <RCM> The persons carrying out sampling and measurement in boreholes should meet the training and experience requirements given in EN ISO 22475-2.

NOTE 122. The minimum requirements for qualifications and experience are given in Table B.1 (NDP) unless the National Annex gives different requirements. Examples of current and proposed requirements for countries can be found in JRP X. (This is a planned paper that will give more guideline. JRP X is expected to be largely based on a publication by Buggy et al (2018) "Registration of Ground Engineering Professionals – a European Perspective", 13th IAEG Congress, San Francisco, Ca. USA 15 – 23 September 2018.)

NOTE 123. European Commission Directive 2005/36/EC on mutual recognition of professional qualifications acknowledges that engineers are organised differently in various EU member states.

NOTE 124. The minimum requirements in Table B.1 (NDP) are applicable for geotechnical structures that fall within Geotechnical Category 2.

NOTE 125. The National Annex can stipulate which professional titles or levels of registration meet the minimum requirements in Table B.1 (NDP).

Table B.1 (NDP) – Minimum requirements for qualifications and professional experience to fulfil the assumptions of EN 1997-1, 1.3, for Geotechnical Category 2 structures

Educational qualification (ECTS credit points)[a]	**Professional experience** [b]	**Continuous Professional Development (CPD)** [c]	**Professional competence** [a,e]	**Remarks, Registration, professional qualifications, and application**
For those responsible for Ground Investigation and Geotechnical Design…				
B Sc / B Eng (180 – 240) Dipl. Ing. / M Sc / M Eng (300)	B Sc / B Eng 5 years – GC 2 Dipl. Ing. / M Sc / M Eng 3 years – GC 2 and demonstrated appropriate competence	≥ 20 hours/year	General requirements are defined in Note 5.	National requirements for registration may be enforced by private or public law. Applications for professional registration should be documented, subject to independent assessment and include a statement of professional competency and curriculum vitae.
For those carrying out or supervising the carrying out of sampling not in a borehole, data collection, measurement and testing…				
EQF Level 4 National Vocational Qualification Certificate or	Documented training for and competence in the work being carried out	≥ 10 hours/year	General requirements are defined in Note 5.	

Educational qualification (ECTS credit points)[a]	**Professional experience [b]**	**Continuous Professional Development (CPD) [c]**	**Professional competence [a,e]**	**Remarks, Registration, professional qualifications, and application**
Diploma preferred (up to 60)				
For those carrying out sampling not in a borehole, data collection, measurement and testing under supervision…				
Suitable training for the work being carried out				
For those carrying out sampling and measurement in boreholes…				
See EN ISO 22475-2				
This table is an NDP and the NSB can clarify the following for its application. - Additional requirements for Geotechnical Category 3 structures - Additional acceptable academic qualification and associated professional experience - Specification of criteria for CPD - Additional general requirements on professional competence - Specific requirements on professional competence for different technical areas				

[a]Core subjects such as soil / rock mechanics, foundation engineering and engineering geology are required as part of university studies.
[b]The professional experience is measured in number of years demonstrating appropriate competence in the application of the relevant clauses of EN 1997.
[c]The criteria for valid CPD hours vary nationally. Learned Societies or Professional bodies can give input to the specification.
[d]The required professional competence, including level of competence, depends on which clauses of EN 1997 a person will apply. Specific requirements for different technical areas can vary. Examples of relevant technical areas include planning of field and laboratory investigation, evaluation of ground investigation results, pile design, ground reinforcement, numerical methods. The professional competence also includes general professional competence related to documentation, project management, risk management, and communication.
[e]The general requirements are defined in the following statement. "*Competence is the ability to carry out a task to an effective standard. To achieve competence requires the right level of knowledge, understanding and skill, and a professional attitude. Competence is developed by a combination of formal and informal learning, and training and experience, generally known as initial professional development. However, these elements are not necessarily separate or sequential and they may not always be formally structured. There are five generic areas of competence and commitment for all ground engineering professionals, broadly covering: A) Knowledge and understanding; B) Design and development of ground engineering processes, systems, services and products; C) Responsibility, management or leadership; D) Communication and inter-personal skills; E) Professional commitment*

Annex E

(informative)

Spacing and depth of investigation points

E.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in Clause 6 for the spacing and depth of ground investigation points.

NOTE 126. 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 assessment of the spacing and depth of ground investigation points.

E.3 Spacing and depth on investigation points

(1) <RCM> The maximum spacing, minimum number, and minimum depth of investigation points should be chosen according to the type of structure.

NOTE 127. The maximum spacing, minimum number, and minimum depth of investigation points for different structures are given in Table E.1 (NDP), unless the National Annex gives different values.

Table E.1 (NDP) – Maximum spacing, minimum number, and minimum depth of investigation points

Structures		**Maximum spacing/minimum number of investigation points**		**Minimum depth of investigation**
		Preliminary investigations	**Design and construction investigation**	
Small buildings		At least 1 No per 50 m² and 3 No	At least 1 No per 50 m² and at least 3 No	2x width of foundation and at least 3 m
Medium buildings		Spacing <25 m At least 1 No per 300 m² and 3 No	Spacing <20 m At least 1 No per 200 m² and 3 No *At least 1 per 100 m²*	2x width of foundation and at least 3 m
Large buildings	< 10,000 m²	Spacing <40 m At least 1 No per 600 m² and 3 No	Spacing <30 m At least 1 No per 400 m² and 3 No	2 m into the compressible horizon and at least 1.5 times width of foundation
	>10,000 m²	At least 1 No additional per 1200 m²	At least 1 No additional per 800 m²	2 m into the compressible horizon and at least 1.5 times width of foundation
Estate roads, parking areas and pavements		Case by case	At least 1 No per 1500 m² and 2 No	3 m below base of works and 2 m into natural ground
Power lines, wind turbines		1 No per pylon/ turbine	1 No per pylon/ turbine	5 m below deepest part of foundation
Linear routes	Buried services	Case by case	1 No per 100 m	1 m below excavation depth
	Tramway, light rail	Case by case	1 No per 100 m	5 m below finished level
	Roads, railways	Case by case	1 No per 100 m	5 m below finished level
	Quays, ports	Case by case	2 No per 100 m	At least 5 m into subgrade
	Retaining structures <3 m high	Case by case	1 No per 100 m	At least 5 m into subgrade
	Retaining structures >3 m high	Case by case	1 No per 50 m	At least 5 m into subgrade
Treatment works		At least 1 No per 500 m² and 1 No per structure	3 No per structure	5 m into the compressible horizon and at least 1.5 times width of building
Silos, reservoirs		At least 1 No per 250 m² and 2 No	At least 1 No per 150 m² and 3 No	5 m into the compressible horizon and at least 1.5 times width of building
Structures (bridges)		1 No per 2 piers/ bases	1 No per pier/ base	5 m below deepest part of foundation

Annex F

(informative)

Methods for evaluating physical and chemical properties

<Drafting note: this Annex is not currently needed>

Annex G

(informative)

Methods for evaluating strength properties

G.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in Clause 8 for evaluating the strength properties of soils and rock.

NOTE 128. 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:

- Mohr-Coulomb parameters;
- peak drained friction angle;
- peak undrained strength;

<DRAFTING NOTE: other relations may be added to those outlined below as drafting proceeds>

G.3 Peak drained friction angle

a. From plasticity

(1) <PER> Provided the conditions give in (2) are satisfied, the drained peak friction angle (φ_{peak}) of normally consolidated clays may be determined from Formula (G.1):

$$\varphi_{peak} = 43° - 10\log_{10} I_P\,, 5 \leq I_P \leq 200 \qquad \text{(G.1)}$$

where:

I_p (%) is the plasticity index of the soil (see EN ISO 17892-12);

φ_{peak} is the peak friction angle that would be obtained by triaxial shearing following EN ISO 17892-9.

(2) <RCM> Formula (A.1) should only be used if:

- the soil is classified as clay according to ISO 14688-2, and
- OCR ≤ 4, and
- effective cohesion is taken as zero.

NOTE 129. The standard error associated with Formula A.1 is 3.7°.

NOTE 130. The database supporting the relation is published in Sorensen and Okkels (2013).

(3) <PER> Provided the conditions give in (4) are satisfied, the drained peak friction angle (φ_{peak}) of normally consolidated clays may be determined from Formula (G.2):

$$\varphi'_{peak} = \begin{cases} 45° - 14\log_{10} I_P & 5 < I_P < 50 \\ 26° - 3\log_{10} I_P & 50 < I_P < 150 \end{cases} \quad \text{(G.2)}$$

where:

I_p (%) is the plasticity index of the soil (see EN ISO 17892-12);

φ_{peak} is the peak friction angle that would be obtained by shearing in triaxial compression following EN ISO 17892-9.

(4) <RCM> Formula (G.2) should only be used if:

- the soil is classified as clay according to ISO 14688-2; and
- $< I_p < 150$, and
- OCR > 4, and
- the cohesive strength component, c, in kPa, is taken as
  - 30 kPa for 5 % < $I_P \leq$ 30 %;
  - (48 – 0.6 I_P) kPa for 30 % < $I_P \leq$ 80 %; and
  - 0 for 80 % < I_P.

The standard error associated with Formula A.2 is 3°.

NOTE 131. The database supporting the relation is published in Sorensen and Okkels (2013).

b. From CPT results

(1) <PER> Provided the conditions given in (2) are satisfied, the expression given by Formula (G.3) may be used to relate CPT results and drained peak friction angle:

$$\varphi'_{peak} = min(11\log_{10} q_{t1} + 17.6°; 45°) \quad \text{(G.3)}$$

$$q_{t1} = \frac{q_t}{\sqrt{\sigma'_{v0}/p_a}}$$

where:

q_{t1} is the corrected cone resistance as per EN ISO 22476-1;

σ'_{v0} is the vertical effective stress at the measurement location.

(2) <RCM> Formula (G.3) should only be used if:

- the fine content of the soil is below 20%;
- the soil D_{50} should is below 40 mm;
- the soil mineralogy is consistent mostly of quartz;
- the vertical effective stress σ'_{v0} is below 1 MPa.

NOTE 132. The standard error associated with Formula A.3 is 3.2°

NOTE 133. The database supporting the relation is published in Ching et al. (2018).

c. From SPT results

(1) <PER> Provided the conditions given in (2) are satisfied, the expression given by Formula (G.4) may be used to relate SPT results and drained peak friction angle

$$\varphi'_{peak} = \min(22.3° + 3.5\sqrt{(N_1)_{60}}, 45°) \quad \text{(G.4)}$$

$$(N_1)_{60} = \frac{N_{60}}{\sqrt{\frac{\sigma'_{v0}}{p_a}}}$$

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 (G.4) should only be used if:

- the soil is classified as sand according to ISO 14688-2;
- the fine content of the sand is below 15%;
- the sand mineralogy is consistent mostly of quartz.

NOTE 134. The standard error associated with Formula A.4 is 2.3°

NOTE 135. The database supporting the relation is published in Hatanaka and Uchida (1996).

d. From DMT results

(1) <PER> Provided the conditions given in (2) are satisfied, the expression given by Formula (G.5) may be used to relate DMT results and drained peak friction angle:

$$\varphi'_{peak} = \min(28° + 14.6\log_{10} K_D - 2.1(\log_{10} K_D)^2, 45°) \quad \text{(G.5)}$$

where:

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

(2) <RCM> Formula (G.5) should only be used if the soil is classified as sand according to ISO 14688-2.

NOTE 1. The relation G.5 is believed to result in conservative estimates.

G.4 Peak undrained strength

a. From plasticity and pre-consolidation pressure

(1) <PER> Provided the conditions given in (2) are satisfied, Formula (G.6) may be used to determine the peak undrained shear strength $c_{u,peak}$ of a clay:

$$\frac{c_{u,peak}}{\sigma'_p} = 0.11 + 0.0037 I_P \tag{G.6}$$

where:

σ'_p pre-consolidation pressure;

I_P is the plasticity index of the clay.

(2) <RCM> Formula (A.6) should only be used if:

- the soil is classified as clay according to ISO 14688-2;
- the soil is not silt-dominated or formed by diatomites;
- the clay organic matter content is below 2%.

NOTE 136. The average ratio of measurement/prediction (bias) for relation G.6 is 0.97.

NOTE 137. The coefficient of variation of the ratio of measurement / prediction relation A.6 is 0.35

NOTE 138. Background to the relation is published in D'Ignazio et al. (2016).

b. From CPT results

(1) <PER> Provided the conditions given in (2) are satisfied, Formula (G.7) may be used to determine the peak undrained shear strength $c_{u,peak}$ of a clay:

$$c_{u,peak} = \frac{q_n}{N_{kt}} = \frac{q_n}{10.5 - 4.6 \log_e \left(\frac{\Delta u_2}{q_n} + 0.1\right)} \tag{G.7}$$

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 pressure measured at the gap between cone tip and friction sleeve as per EN ISO 22476-1.

(2) <RCM> Formula (G.7) should only be used if:

- the soil is classified as clay according to ISO 14688-2;
- the clay is saturated when the CPT is performed;
- the clay is of low sensitivity according to ISO 14688-2;
- the clay has OCR < 2.5.

NOTE 139. The average ratio of measurement/prediction (bias) for relation A.7 is 1.09

NOTE 140. The standard deviation of the ratio of measurement / prediction for relation A.7 is 0.28

NOTE 141. The original database supporting the relation is described in Mayne and Peuchen (2018).

c. From SPT results

(3) <PER> Provided the conditions given in (2) are satisfied, Formula (G.8) may be used to determine the peak undrained shear strength $c_{u,peak}$ of a clay from SPT results

$$c_{u,UU}(kPa) = 7.57 \cdot N_{60} \tag{G.8}$$

where

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

$c_{u,UU}$ is the peak undrained strength that would be obtained from UU triaxial tests as per EN ISO 17892-8.

(4) <RCM> Formula (G.8) should only be used if:

- the soil is classified as clay according to ISO 14688-2;
- the clay must be of low sensitivity according to ISO 14688-2;
- the Atteberg limits are such that 20% < w_L < 110% and 14% < w_p < 44%.

NOTE 142. The standard error associated with relation 8.9 is 36 kPa

NOTE 143. The original database supporting the relation is described in Sivrikaya and Toğrol (2006).

d. From PMT results

(1) <PER> Relations in the form given by Formula (G.9) may be calibrated to relate PMT results and undrained strength:

$$c_{u,X} = \frac{p_{LM} - p_1}{K_{PMT}} \tag{G.9}$$

where:

$c_{u,X}$ is a peak undrained strength directly measured by a specific test X (where X may be UCS, UU, TXCU, FVT….);

p_{LM} is the Ménard 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 (EN ISO 22476-4);

K_{PMT} is a calibration factor.

NOTE 144. Typically observed values of KPMT range from 2 to 20, with higher values corresponding to stiffer soils.

NOTE 145. The National Annex can specify a narrow down range of values for KPMT for specific geological formations.

Annex H

(informative)

Methods for evaluating stiffness and consolidation properties

H.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in Clause 9 for evaluating the stiffness and consolidation properties of soils and rock.

NOTE 146. 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.

H.2 Scope and field of application

(1) This Informative Annex covers:

- evaluation of sample disturbance;
- definitions of soil stiffness;
- parameters for empirical models;
- <others?>

H.3 Evaluation of sample disturbance

NOTE 147. 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 (visual) assessment of sample quality may be made by examination of sample X-rays or CT scans as described in ISO 19901-8:2014.

(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 sample 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 sample quality parameter $\Delta e/e_0$ should be computed from Formula H.1:

$$\Delta e/e_0 = \varepsilon_{vol} \cdot (1 + e_0)/e_0 \tag{H.1}$$

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 148. 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 149. The sample quality can be determined using Table H.1 (see reference [8]).

Table H.1 — Evaluation of intact sample quality for low to medium OCR clays

Quality	**$\Delta e/e_0$**		**Specimen MQC**
	OCR = 1-2	**OCR = 2-4**	
Very good	<0,04	<0,03	1 (small strain)
Good	0,04−0,07	0,03−0,05	1
Poor	0,07−0,14	0,05−0,10	2
Very poor	>0,14	>0,10	3

NOTE 150. The sample quality criteria in Table H.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.

H.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 151. Strain levels appropriate for different structures are shown in Figure H.1.

NOTE 152. Figure H.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.

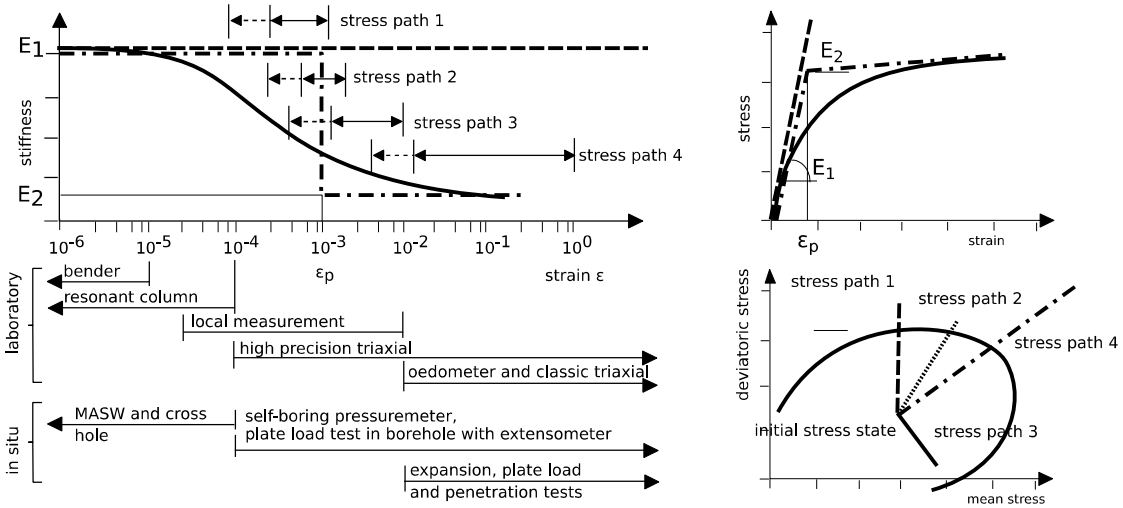

Figure H.1 – Extrapolation errors and preferred range for the determination of modulus (Atkinson and Sällfors, 1991, Tatsuoka et al., 1997)

NOTE 153. The secant moduli decrease with the amplitude of the (shear) strain. Therefore, to recommend or propose the measurement of a module, it is also necessary to specify the corresponding level of loading. The mean or confining pressure must also be defined. And then, it is necessary to indicate the direction of the stress in case of anisotropy, because the ratio of the horizontal on vertical modules can vary widely beyond the unit.

NOTE 154. In contrast, rocks are quite often elastic. Their modulus also varies sometimes in the other direction, by clamping effect and at the beginning of the loading (the module increases with the strain, by effect of compressibility, like a soil in the oedometer ring). And then there are materials with intermediate behavior.

(2) <RCM> The determination on the experimental curves should be adapted to the range of variation possible for these parameters.

NOTE 155. Figure H.1 compares elastic (dashed), elastoplastic (dashed with dots), and hyperbolic (continuous line) laws. Determining a unique secant modulus at any particular strain on the experimental curve depends on the assumption of purely elastic behaviour before reaching the plasticity.

(3) <RCM> The homogeneous deformability of the soil mass should be evaluated judiciously in order to be representative of the average behaviour or at least of that which one seeks to observe. It cannot be relevant for all observable behaviours from initial loading to failure.

NOTE 156. 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 adapt, thanks to the command, the obtaining of the experimental curves:

- at the proposed rheological model;
- at the range of possible variation of the parameters and to have a suitable data acquisition frequency.

NOTE 157. As shown on Figure H.1 for triaxial compression test, the use of an E_{sec} secant module 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} module becomes problematic because of the increasing resolution that this requires. An alternative to determining an initial or secant modulus by tests such as the resonant column is the determination of an E_{cyc} cyclic module for low amplitude unloading (i.e. loops). Often, the module obtained then is higher than the initial module 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.

NOTE 158. The “true” elastic modulus is the modulus obtained at unloading.

(5) <RCM> The realization of cycles during laboratory and field tests should be stated when planning the ground investigation.

NOTE 159. The same goes for the modules and their variation in power of the average pressure. It is preferable to have spread mean pressures to catch this non-linearity.

NOTE 160. For other more sophisticated and non-linear criteria, stakeholder shall give themselves the means needed to experimentally find the peculiarities of the model, to multiply the number of specimens and, again, to perform tests around, but also at a distance, of singularities (passage from the "back" of the yield surface of Coulomb envelop for example).

H.5 Parameters for empirical models

(1) <PER> Values of the parameters for use with Formula (9.3) may be taken from Table H.2.

(2) <PER> Values other than those given in Table H.2 may be used with Formula (9.3) provided the testing, reporting, and interpretation procedures conform to the general prescriptions given in Clause 9.

Table H.2 — Values of parameters for use with Formula (9.3)

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}	2
Clay and silt	0,0022 I_p	0,735 ±0,122	0	5

(3) <PER> Values of the parameters for use with Formula (9.4) may be taken from Table H.3.

(4) <PER> Values other than those given in Table H.3 may be used with Formula (9.4) provided the testing, reporting, and interpretation procedures conform to the general prescriptions given in Clause 9.

Table H.3 — Values of parameters for use with equation (9.4)

Soil type as per 14688-1	Parameters				Reference
	k_1	k_2	k_3	p_{ref}	
Fine grained soil	2100	0	0,6-0,8	1	4
Sand	370-5760	3	0,49-0,86	100	2
Clay and silt	20000 ±5000	2,4	0,5	1	5

Annex I

(informative)

Methods for evaluating mechanical response to dynamic loads and parameters for seismic design

I.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in Clause 10 for evaluating the mechanical response of soils and rocks to dynamic loads and parameters for seismic design.

NOTE 161. 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.

I.2 Scope and field of application

(1) This Informative Annex covers:

- indirect methods for the evaluation of normalised secant shear moduli and damping ratio curves;
- indirect methods for the evaluation of shear wave velocity (V_S).

I.3 Indirect methods for the evaluation of normalised secant shear moduli and damping ratio curves

a. Fine soils

(1) <PER> The normalised secant shear modulus for fine soils may be evaluated as a function of cyclic shear strain according to Formula (I.1):

$$\frac{G_{\sec}(\gamma)}{G_0} = \left[1 + \left(\frac{\gamma}{\gamma_{\text{ref}}}\right)^{\alpha}\right]^{-1} \tag{I.1a}$$

$$\gamma_{ref}(\%) = \left(\phi_1 + \phi_2 \times PI \times OCR^{\phi_3}\right) \times \left(\frac{\sigma_0'}{p_a}\right)^{\phi_4} \tag{I.1b}$$

where:

G_0	is the soil small-strain shear modulus;
γ	is the cyclic shear strain;
α	is a curvature coefficient, given in Table I.1 as ϕ_5
γ_{ref}	is a reference value of engineering shear strain (at which $G_{\sec}/G_0$ = 0.5)
PI	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 I.1.

NOTE 162. Formula (I.1) was originally proposed by Darendeli (2001).

(2) <PER> The shear damping ratio D of fine soils may be evaluated as a function of cyclic shear strain according to Formula (I.2):

$$D(\gamma) = D_0 + f\left(\frac{G(\gamma)}{G_0}\right) \tag{I.2a}$$

$$D_0 = \left(\phi_6 + \phi_7 \times PI \times OCR^{\phi_8}\right) \times \sigma_0'^{\phi_9} \times [1 + \phi_{10}\, ln(f)] \tag{I.2b}$$

$$f\left(\frac{G(\gamma)}{G_0}\right) = b \times D_M(\gamma) \times \left(\frac{G(\gamma)}{G_0}\right)^{0,1} \tag{I.2c}$$

where:

b is given by $b = \phi_{11} + \phi_{12}\, ln\, N$

D_0 is the small strain damping ratio;

$D_M(\gamma)$ is given by
$D_M(\gamma) = c_1\left(D_{M,\alpha=1}\right) + c_2\left(D_{M,\alpha=1}\right)^2 + c_3\left(D_{M,\alpha=1}\right)^3$

$D_{M,\alpha=1}(\gamma)$ is given by

$$D_{M,\alpha=1}(\gamma) = \frac{100}{\pi}\left[4\frac{\gamma - \gamma_r \ln\left(\frac{\gamma+\gamma_r}{\gamma_r}\right)}{\frac{\gamma^2}{\gamma+\gamma_r}} - 2\right]$$

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)

PI 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 I.1.

NOTE 163. Formula (I.2) was originally proposed by Darendeli (2001).

Table I.1 – Constants for the evaluation of normalised shear modulus and damping ratio of fine soils

Parameter	Value	Parameter	Value
ϕ_1	0.0352	ϕ_9	-0.2889
ϕ_2	0.0010	ϕ_{10}	0.2919
ϕ_3	0.3246	ϕ_{11}	0.6329
ϕ_4	0.3483	ϕ_{12}	-0.0057
ϕ_5	0.9190	ϕ_{13}	-4.23
ϕ_6	0.8005	ϕ_{14}	3.62
ϕ_7	0.0129	ϕ_{15}	-5.00
ϕ_8	-0.1069	ϕ_{16}	-0.25

(3) <PER> The variability of the normalised shear modulus may be estimated assuming a normal distribution and a value of the variance given by according to Formula (I.3):

$$\sigma_{NG} = e^{\phi_{13}} + \sqrt{\frac{0.25}{e^{\phi_{14}}} - \frac{\left(\left[\frac{G(\gamma)}{G_0}\right]_{mean} - 0.5\right)^2}{e^{\phi_{14}}}} \tag{I.3}$$

where:

$\left[\frac{G(\gamma)}{G_0}\right]_{mean}$ is given by Formula I.1; and

ϕ_{13}, ϕ_{14} are constants given in Table I.1.

NOTE 164. Formula (I.3) was originally proposed by Darendeli (2001).

(4) <PER> The variability of the shear damping ratio may be estimated assuming a normal distribution and a value of the variance σ_D given by according to Formula (I.4):

$$\sigma_D = e^{\phi_{15}} + e^{\phi_{16}}\sqrt{[D(\gamma)]_{mean}} \tag{I.4}$$

where:

$[D(\gamma)]_{mean}$ is given by Formula I.2;

ϕ_{15}, ϕ_{16} are constants given in Table I.1.

I.4 Coarse soils

(1) <PER> Provided the conditions given in (2) are satisfied, the normalised secant shear modulus for coarse soils may be evaluated as a function of cyclic shear strain according to Formula (I.5):

$$\frac{G_{sec}(\gamma)}{G_0} = \left[1 + \left(\frac{\gamma}{\gamma_{ref}}\right)^{\alpha}\right]^{-1} \tag{I.5a}$$

$$\gamma_{ref}(\%) = 0.12 c_u^{-0.6} \times \left(\frac{\sigma_0'}{p_a}\right)^{0.5 c_u^{-0.15}} \tag{I.5b}$$

where:

G_0 is the soil small-strain shear modulus;

γ is the cyclic shear strain;

α is the curvature coefficient given by $\alpha = 0.86 + 0.1 \log\left(\frac{\sigma_0'}{p_a}\right)$;

γ_{ref} is a reference value of engineering shear strain (at which G_{sec}/G_{max} = 0.5);

c_u is the uniformity coefficient;

p_a is the atmospheric pressure;

σ_0' is the mean effective stress;

NOTE 165. Formula (I.5) was originally proposed by Menq (2003).

(2) <PER> The shear damping ratio D of coarse soils can be evaluated as a function of cyclic shear strain according to Formula (I.6) proposed by Menq (2003):

$$D(\gamma) = D_0 + f\left(\frac{G(\gamma)}{G_0}\right) \tag{I.6a}$$

$$D_0 = 0.55 c_u^{0.1} \times D_{50}^{-0.3} \times \left(\frac{\sigma_0'}{p_a}\right)^{-0.05} \tag{I.6b}$$

$$f\left(\frac{G(\gamma)}{G_0}\right) = b \times D_M(\gamma) \times \left(\frac{G(\gamma)}{G_0}\right)^{0,1} \tag{I.6c}$$

where:

D_0 is the small strain damping;

c_u is the uniformity coefficient;

D_{50} Is the median grain size

p_a is the atmospheric pressure;

σ'_0 is the mean effective stress;

b is given by: $b = 0.6329 - 0.0057\, ln\, N$

$D_M(\gamma)$ is given by: $D_M(\gamma) = c_1\left(D_{M,\alpha=1}\right) + c_2\left(D_{M,\alpha=1}\right)^2 + c_3\left(D_{M,\alpha=1}\right)^3$

$D_{M,\alpha=1}(\gamma)$ is given by: $D_{M,\alpha=1}(\gamma) = \frac{100}{\pi}\left[4\frac{\gamma-\gamma_r \ln\left(\frac{\gamma+\gamma_r}{\gamma_r}\right)}{\frac{\gamma^2}{\gamma+\gamma_r}} - 2\right]$

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 number of cycles (default value 10),

NOTE 166. The model is reliable for dry soils and shear strains ranging between 0.0001% and 0.6%.

I.5 Indirect methods for the evaluation of shear wave velocity

a. From Standard Penetration Tests

(1) <PER> The shear wave velocity V_S of sands may be evaluated from the results of Standard Penetration Tests using Formula (I.7):

$$V_S = k_{vs} N_{60}^{0.23} \sigma_v'^{0.25} \tag{I.7}$$

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 167. Formula (I.7) was originally proposed by Wair et al. (2012).

b. From Cone Penetration Tests

(1) <PER> The shear wave velocity V_S of Pleistocene sands may be evaluated from the results of Cone Penetration Tests using Formula (I.8):

$$V_S = \left(10^{(0.55 I_C + 1.68)} \frac{q_t - \sigma'_v}{p_a}\right)^{0.5} \tag{I.8}$$

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 (I.8) was originally proposed by Robertson (2009).

c. From Flat Dilatometer Tests

(1) <PER> The small-strain shear modulus G_0 may be evaluated from the results of Flat Dilatometer Tests using Formula (I.9):

$$G_0 = k_1 K_D^{-k_2} M_{DMT} \tag{I.9}$$

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 (I.9) was originally proposed Monaco et al. (2009).

I.6 References

<Drafting NOTE: these references will be moved to the Bibliography in the final version>

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Annex J

(informative)

Methods for evaluating groundwater and hydraulic conductivity

<Drafting note: this Annex is not currently needed>

Annex K

(informative)

Methods for evaluating thermal properties

<Drafting note: this Annex is not currently needed>

Annex L

(informative)

Ground Investigation Report

L.1 Use of this Informative Annex

(2) This Informative Annex provides additional guidance to that given in Clause 13 for preparing the Ground Investigation Report.

NOTE 2. 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.

L.2 Scope and field of application

(3) This Informative Annex covers:

- contents of the Ground Investigation Report;

L.3 Contents of Ground Investigation Report

(1) <RCM> The contents of the Ground Investigation Report should include the following information:

- General information
  - Project name
  - Planned structure stage of execution relevant for GIR, scope of investigation
  - Normative references
  - List of available information needed for planning
  - Geotechnical Category (selection for the ground investigation purposes), if applicable
  - Site overview (topography, existing structures, vegetation)
  - Location (coordinates)
  - Current iteration of the Ground Model
  - Desk study information
  - Site reconnaissance
- Field Investigation
  - List of performed investigations and locations
  - Dates of fieldwork
  - Names of field personnel
  - Type of Equipment
  - Documentation of calibration and certification documents
  - Handling of samples
  - Environmental conditions during testing
  - Main site observations during the investigation
- Field testing
  - List of tests performed, locations and depths
  - Dates of tests
  - Names of field personnel
  - Documentation of calibration and certification documents

    - Observations during testing (quality, down hole conditions, environmental site conditions)
    - Results of tests and evaluation
  - Laboratory testing
    - List of performed investigations, on which samples
    - Dates tests performed
    - Names of laboratory personnel
    - Documentation of calibration and certification documents
    - Main observations during testing (quality, sample content)
    - Results of tests and evaluation
  - Groundwater investigations
    - List of performed investigations and locations (short and long term)
    - Dates of investigations
    - Names of field personnel
    - Documentation of calibration and certification documents
    - Handling of samples
    - Main site observations during the investigation
  - Derived values
    - Physical and chemical properties
    - Strength properties
    - Stiffness properties
    - Mechanical response to dynamic loads
    - Groundwater
    - Thermal properties
  - Review of investigation, results and evaluation
    - Known limitations of the results
    - Important observations from the field and laboratory investigations
    - Identification of gaps in knowledge about the site
- Reports and measurements:
  - Current iteration of the ground model
  - Desk studies
  - Site inspection
  - Field reports
  - Field test reports
  - Laboratory reports
  - Groundwater test reports and measurements
  - Evaluation of results with derived values
  - Graphical presentation of derived values
  - Estimates of coefficients of variation of the ground properties.
- Drawings:
  - plans
  - cross sections
- Data for inclusion in the digital Model (such as BIM)

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Clause 6

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