



BSI Standards Publication

Eurocode 8 — Design of structures for earthquake resistance

Part 5: Geotechnical aspects, foundations, retaining and underground structures

National foreword

This British Standard is the UK implementation of EN 1998-5:2024. It supersedes BS EN 1998-5:2004, which will be withdrawn on 30 March 2028.

The UK participation in its preparation was entrusted to Technical Committee B/525/8, Structures in seismic regions.

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

National choice is allowed in this standard where explicitly stated within notes. The National Annex to this standard contains the national choices to be used for buildings and civil engineering works constructed in the UK.

The first generation of EN Eurocodes was published between 2002 and 2007, with conflicting British Standards withdrawn in 2010. This document forms part of the second generation of EN Eurocodes.

The second generation of EN Eurocodes is expected to be published between 2023 and 2026. These documents are being published as soon as they are available. This is being done to enable users to prepare for the transition from the first generation to second generation of EN Eurocodes.

UK adoptions of the first generation of EN Eurocodes will be withdrawn by BSI on 30 March 2028. Until that date, the first generation documents should be considered as the applicable standards for buildings and civil engineering works constructed in the UK unless otherwise specified by the relevant authority or in the specification for a particular project.

This standard is intended to be used with its National Annex and other referenced documents, including other second generation Eurocodes, as an interdependent suite of documents.

While the use of provisions in this standard in conjunction with first generation Eurocodes is not precluded, it should be undertaken with care and should only be done when users are satisfied that it will not result in a lower level of reliability than the minimum level set in the first generation Eurocodes and associated UK National Annexes.

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English Version

**Eurocode 8 - Design of structures for earthquake
resistance - Part 5: Geotechnical aspects, foundations,
retaining and underground structures**

Eurocode 8 - Calcul des structures pour leur résistance
au séisme - Partie 5 : Aspects géotechniques,
fondations, soutènements et structures souterraines

Eurocode 8 - Auslegung von Bauwerken gegen
Erdbeben - Teil 5: Geotechnische Aspekte,
Gründungen, Stütz- und Untertagebauwerke

This European Standard was approved by CEN on 7 July 2024.

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European foreword

This document (EN 1998-5:2024) 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 European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by September 2027, and conflicting national standards shall be withdrawn at the latest by March 2028.

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. CEN shall not be held responsible for identifying any or all such patent rights.

This document supersedes EN 1998-5:2004.

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 main changes compared to the previous edition are listed below:

- improved consistency with EN 1997;
- distinction between geotechnical systems and geotechnical structures;
- alignment of “basis of design” to the new concepts in EN 1998-1-1;
- derivation of seismic action for geotechnical systems/structures;
- water levels according to EN 1990;
- rules for capacity design of foundations;
- development of the displacement-based approach for all topics;
- development of models for soil-structure interaction;
- rules for analysis and verification of piles;
- new clauses on seismic action effects on underground structures (tunnels, large spaces, pipelines, etc.).

The Eurocodes recognize 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.

Any feedback and questions on this document should be directed to the users’ national standards body. A complete listing of these bodies can be found on the CEN website.

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

0 Introduction

0.1 Introduction to the Eurocodes

The Structural Eurocodes comprise the following standards generally consisting of several 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 are under development, e.g. Eurocode for design of structural glass

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

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

0.2 Introduction to EN 1998 (all parts)

EN 1998 (all parts) defines the rules for the seismic design of new buildings and other structures, as well as temporary ones, including geotechnical aspects.

EN 1998 (all parts) also defines the rules for the seismic assessment and retrofit of existing buildings and other structures.

EN 1998 (all parts) additionally covers the verification of structures in the seismic design situation during construction, when required.

For the design of structures in seismic regions, the provisions of EN 1998 (all parts) are to be applied in conjunction with the relevant provisions of EN 1990 to EN 1997 and EN 1999.

EN 1998 (all parts) applies to structures of consequence classes CC1, CC2 and CC3, as defined in EN 1990. The provisions in the Eurocodes do not entirely cover design rules needed for structures classified as CC4. For these structures, additional provisions to those given in the Eurocodes can be needed.

Given that seismic hazard is characterized by a significant uncertainty, a null seismic risk is not achievable in practice. Therefore, in Eurocode 8, the seismic action is represented in a conventional form, proportional in amplitude to earthquake ground motions likely to occur at a given location and representative of their frequency content. This representation is not the prediction of a particular seismic movement, and such a movement could give rise to more severe effects than those of the seismic action considered, inflicting damage greater than the one described by the Limit States contemplated in EN 1998 (all parts).

In addition, engineering methods are associated with assumptions that cannot be verified when considering the effects of the seismic action, under which structures are assumed to respond in the nonlinear regime. Such uncertainties are taken into account according to the general framework of EN 1990, with a residual risk of underestimation of their effects.

EN 1998 is subdivided in various parts:

- EN 1998-1-1 Eurocode 8 — Design of structures for earthquake resistance — Part 1-1: General rules and seismic action
- EN 1998-1-2 Eurocode 8 — Design of structures for earthquake resistance — Part 1-2: Buildings
- EN 1998-2 Eurocode 8 — Design of structures for earthquake resistance — Part 2: Bridges
- EN 1998-3 Eurocode 8 — Design of structures for earthquake resistance — Part 3: Assessment and retrofitting of buildings and bridges
- EN 1998-4 Eurocode 8 — Design of structures for earthquake resistance — Part 4: Silos, tanks, pipelines, towers, masts and chimneys
- EN 1998-5 Eurocode 8 — Design of structures for earthquake resistance — Part 5: Geotechnical aspects, foundations, retaining and underground structures

0.3 Introduction to EN 1998-5

This document contains general requirements for the earthquake resistant design of geotechnical structures and geotechnical systems, including the definition of the seismic action, of the ground characteristics, general requirements for siting and foundations soils, design of foundation systems, retaining structures and underground structures, as well as rules for consideration of soil-structure interaction.

This document also contains provisions for the assessment of existing geotechnical structures and geotechnical systems.

0.4 Verbal forms used in the Eurocodes

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

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

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

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

0.5 National annex for EN 1998-5

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

The national standard implementing EN 1998-5 can have a National Annex containing all national choices to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

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

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

National choice is allowed in EN 1998-5 through notes to the following clauses:

4.2(3)	4.2(6)	4.3(3)	6.5(2)
6.5(3)	7.3.1(2)	9.4.2.1.3(7)	

National choice is allowed in EN 1998-5 on the application of the following informative annexes:

Annex A	Annex B	Annex C	Annex D
Annex E	Annex F	Annex G	Annex H
Annex I			

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 1998-5

(1) This document establishes general principles for the design and assessment of geotechnical systems in seismic regions. It gives general rules relevant to all families of geotechnical structures, to the design of foundations, retaining structures and underground structures and complements EN 1997-3 for the seismic design situation.

(2) This document contains the basic performance requirements and compliance criteria applicable to geotechnical structures and geotechnical systems in seismic regions.

(3) This document refers to the rules for the representation of seismic actions and the description of the seismic design situations defined in EN 1998-1-1 and provides specific definition of the seismic action applicable to geotechnical structures.

1.2 Assumptions

(1) The assumptions of EN 1990 apply to this document.

2 Normative references

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

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

EN 1990:2023¹, *Eurocode — Basis of structural and geotechnical design*

EN 1997-1:2024, *Eurocode 7 — Geotechnical design — Part 1: General rules*

EN 1997-2:2024, *Eurocode 7 — Geotechnical design — Part 2: Ground properties*

EN 1997-3:—², *Eurocode 7 — Geotechnical design — Part 3: Geotechnical structures*

EN 1998-1-1:2024, *Eurocode 8 — Design of structures for earthquake resistance — Part 1-1: General rules and seismic action*

EN 1998-3:—³, *Eurocode 8 — Design of structures for earthquake resistance — Part 3: Assessment and retrofitting of buildings and bridges*

ISO 80000, *Quantities and units*

¹ As impacted by EN 1990:2023/prA1:2024.

² Under preparation. Stage at the time of publication: FprEN 1997-3:2024.

³ Under preparation. Stage at the time of publication: prEN 1998-3:2023.

3 Terms, definitions and symbols

3.1 Terms and definitions

For the purposes of this document, the terms and definitions given in EN 1990 and in EN 1998-1-1:2024, 3.1, apply.

The terms and definitions in EN 1997-1, EN 1997-2 and EN 1997-3 apply, while the definitions of other geotechnical terms specifically related to earthquakes are given in this standard.

Additional terms in 3.1.1 to 3.1.28 are used in this document with the corresponding definitions.

3.1.1

apparent wave velocity

horizontal surface velocity of the wave field under an incident angle within the ground

Note 1 to entry: It can be closer to the velocity of seismic waves through deep rocks.

3.1.2

clay fraction

percent in dry weight of soil with particle size smaller than 2 μm

3.1.3

coarse-grained soil

soil where particle sizes between 63 μm and 63 mm predominate

3.1.4

critical layer

soil layer within a soil profile identified based on its depth, relative density, and thickness, as the most likely to liquefy and be responsible for the greatest amount of damage

3.1.5

critical zone (of piles)

zone around the section of a pile where the action effect reaches the elastic limit expressed in terms of stresses or deformations

3.1.6

critical seismic coefficient

minimum value of the seismic coefficient that leads to pseudo-static failure

3.1.7

cyclic undrained shear strength

shear strength of a ground material under cyclic undrained loading produced by the considered earthquake, also referred to as the cyclic resistance of a material

3.1.8

cyclic stress ratio

CSR

cyclic shear stress normalised by the initial vertical effective stress for a given depth, described also as the cyclic loading on the soil

3.1.9

cyclic resistance ratio

CRR

cyclic undrained shear strength normalised by the initial vertical effective stress for a given depth, described also as the resistance of a soil to liquefaction triggering

3.1.10

displacing retaining structure

retaining structure that is able to undergo permanent seismic displacements

3.1.11

fine-grained soil

soil where particle sizes smaller than 63 μm predominate

3.1.12

finer content

percent in dry weight of material smaller than 63 μm

3.1.13

foundation element

structural member of a foundation system

EXAMPLE Footings, foundation beams, rafts, pile caps, tie-beams

3.1.14

geotechnical structure

structure that includes ground or a structural member that relies on the ground for resistance

3.1.15

geotechnical system

complex system where one geotechnical structure interacts with other structures or geotechnical structures

EXAMPLE Retaining walls with a supported structure at the crest, slopes with a structure at the crest or toe

3.1.16

inertial effect

action effect induced in the seismic design situation by the inertia forces

3.1.17

kinematic effect

action effect induced in the seismic design situation by the seismic ground displacements

3.1.18

liquefaction susceptibility

potential for a soil deposit to trigger liquefaction in the seismic design situation

3.1.19

material damping

energy dissipated by the material in cyclic loading

3.1.20

non-displacing retaining structure

retaining structure that is not able to undergo permanent seismic displacements

3.1.21**p-y curve**

relationship between the resultant of the horizontal contact stresses per unit length of a vertical pile and the corresponding horizontal displacement

3.1.22**permanent seismic displacement**

seismically-induced displacement that remains after an earthquake

3.1.23**pile group effect**

modification of the pile group response, with respect to that of a single isolated pile, due to the pile-soil-pile interaction

3.1.24**precarious slope**

slope with a low margin of safety in a static situation as calculated according to EN 1997-3 or slopes classified with geotechnical complexity GCC3 according to EN 1997-1:2024, 4.1.2.3

3.1.25**radiation damping**

energy dissipated in the ground by waves travelling away from the foundation

3.1.26**residual strength**

shear strength of a liquefied material, or lower limit of the shear strength of a fine-grained soil reached after extensive shearing and particle reorientation as defined in EN 1997-2:2024, 3.1.5.6

3.1.27**t-z curve**

relationship between the resultant of the vertical shear contact stresses per unit length of a vertical pile and the corresponding vertical displacement

3.1.28**yielding (non-yielding) pile**

pile that undergoes (does not undergo) inelastic deformation in the seismic design situation

3.2 Symbols and abbreviations

For the purposes of this document, the symbols and abbreviations given in EN 1990:2023, 3.2, and in EN 1998-1-1:2024, 3.2, apply.

For the symbols related to materials, as well as for symbols not specifically related to the seismic design situation, the provisions of the relevant Eurocodes apply.

In addition, further symbols and abbreviations, used in connection with the seismic design situation, are defined in the present standard where they occur, for ease of use. However, the most frequently occurring symbols used in EN 1998-5 are listed and defined in 3.2.1 and additional abbreviations are given in 3.2.2.

3.2.1 Symbols

3.2.1.1 Upper case Latin symbols

A_b	Area of the base of the footing in contact with the ground
A_T	Area of the cross section of the tunnel
A_{Sw}	Amplitude of the shear wave
B	Foundation width
B_b	Building width
CAV_{dp}	Filtered cumulative absolute velocity
C_E	Hammer energy correction factor in SPTs
C_N	Overburden stress correction factor for normalisation to atmospheric pressure
C_R	Flexibility ratio for tunnels
C_{xx}	Radiation dashpot coefficient in the horizontal X direction
C_{yy}	Radiation dashpot coefficient in the horizontal Y direction
C_{zz}	Radiation dashpot coefficient in the vertical Z direction
C_{rx}	Radiation dashpot coefficient around the horizontal X direction
C_{ry}	Radiation dashpot coefficient around the horizontal Y direction
C_{rz}	Radiation dashpot coefficient around the vertical Z direction
CRR	Cyclic resistance ratio in liquefaction assessment
CSR	Cyclic stress ratio in liquefaction assessment
C_α	Dashpot coefficient for degree of freedom α corresponding to radiation damping
$C_{\alpha\tau}$	Dashpot coefficient for degree of freedom α corresponding to foundation damping (material + radiation)
D_e	Depth from the ground surface to the base of the foundation
D_H	Lateral displacement due to lateral spreading
D_R	Duration of the strong part of a ground motion
D_S	Settlement under building
E_{di}	Design value of the action effect on the zone or element i in the seismic design situation
E_{Ei}	Elastic strain energy in element i
E_{Fd}	Design value of the action effect in the seismic situation expressed as generalised stresses (force-based approach) or as calculated displacements (displacement-based approach)
$E_{Fd,G}$	Design value of the action effect in non-seismic situations
$E_{Fd,E}$	Action effect of the design seismic situation calculated from the seismic analysis
E_L	Young's modulus of the tunnel lining
E_P	Young's modulus of a pile
ER	Energy ratio, in percent, specific to the testing equipment, in SPTs

$E_R I_R$	Bending stiffness, per unit length, of the retaining wall: product of the Young's modulus times the bending inertia per unit length
E_S	Young's modulus of soil compatible with a representative in space and time shear strain
E_{str}	Young's modulus of a box structure
F	Normalised friction ratio in CPTs
$\bar{F}$	Dimensionless soil inertia force
F_A	Ratio of S_a to the zero-period spectral acceleration
F_R	Flexibility ratio for tunnels
F_{Rd}	Design value of the sliding resistance of a footing
FS	Factor of safety against liquefaction
G	Secant shear modulus compatible with a representative in space and time shear strain
G_0	Shear modulus at very small strain
H	Total height of a slope, or height of the retaining structure in contact with the ground
H_{cov}	Minimum soil cover between the rock formation and the foundation elevation to allow construction in the vicinity of an active fault
H_{heav}	Heaviside function
H_L	Thickness of liquefiable layer
H_R	Total height of the retaining system
H_{st}	Total height of the underground structure from roof to the invert slab elevation
H_w	Total height of water in contact with the retaining structure
I_{bx}, I_{by}, I_{bz}	Moments of inertia (about the x, y and z axes, respectively) of the base of the footing in contact with the ground
I_c	Soil behaviour type index for CPTs
I_D	Sand relative density in percent
I_e	Effective moment of inertia per unit length of the lining of a circular segmented tunnel
I_j	Moment of inertia per unit length of the joint of a segmental lining of a circular tunnel
I_l	Moment of inertia per unit length of the invert slab of the structure
I_L	Moment of inertia per unit length of the tunnel lining cross section
I_P	Plasticity index of the soil
I_R	Moment of inertia per unit length of the roof slab of the structure
I_T	Moment of inertia of the tunnel lining's cross section
I_w	Moment of inertia per unit length of the side walls of the structure
K_a	Soil spring stiffness in the longitudinal horizontal direction
K_{AE}	Active earth pressure coefficient in the seismic design situation
K_h	Soil spring stiffness in the transverse horizontal direction
$K_{ii,D}$	Foundation impedance for embedded foundation
K_{ii}	Foundation impedance for surface foundation

K_{xx}	Zero-frequency value of the impedance in the horizontal X direction
K_{yy}	Zero-frequency value of the impedance in the horizontal Y direction
K_{zz}	Zero-frequency value of the impedance in the vertical Z direction
K_{rx}	Zero-frequency value of the rocking impedance around the horizontal X direction
K_{ry}	Zero-frequency value of the rocking impedance around the horizontal Y direction
K_{rz}	Zero-frequency value of the torsional impedance around the vertical Z direction
K_{PE}	Passive earth pressure coefficient in the seismic design situation
K_{α}	Spring coefficient for degree of freedom α
K_{σ}	Overburden stress factor
L	Largest in-plane foundation dimension
L_e	Distance of anchors from wall in seismic conditions
L_P	Pile length
L_V	Distance from the centre of the pile top plastic hinge to the pile point of contraflexure
L_s	Distance of anchors from wall under static conditions
LSN	Liquefaction severity number
LPI	Liquefaction potential index
M_{Ed}	Design value of the action in terms of moments
M_{ff}	Bending moment in the lining in absence of SSI
MSF	Magnitude scaling factor
M_{ssi}	Bending moment in the lining in presence of SSI
M_w	Moment magnitude
M_{wT}	Moment magnitude threshold for liquefaction assessment
N	SPT blow count
$\bar{N}, \bar{V}, \bar{M}$	Dimensionless vertical force, shear force and overturning moment acting on the foundation
$(N_1)_{60}$	Normalised SPT blow count for overburden and energy ratio
$(N_1)_{60,cs}$	Normalised SPT blow count for overburden, energy ratio and fines content
N_c	Bearing capacity factor
N_{Ed}	Mean value of the design axial forces of the connected vertical elements in the seismic design situation
N_{ff}	Normal force in the lining in absence of SSI
N_{max}	Ultimate bearing capacity of the foundation under a vertical centred load
$N_{pl,Rd}$	Design value of the yield resistance in tension of the gross cross section of a member in accordance with EN 1993-1-1
N_{SSI}	Normal force in the lining in presence of SSI
PGA_e	Design peak value of horizontal ground acceleration defined in EN 1998-1-1:2024, 5.2.2.4 (equal to a_H)

PGA	Horizontal peak ground acceleration at depth
PGV	Horizontal peak ground velocity at depth
PGD	Horizontal peak ground displacement at depth
PGD_e	Design peak value of the horizontal ground displacement
Q	Normalised point resistance in CPTs
Q_L	Foundation contact pressure
R	Horizontal distance from site to seismic source
R_c	Ovaling ratio
R_{di}	Design value of the resistance of the zone or element i
R_d	Design value of the resistance (force-based approach) or allowable displacement (displacement-based approach)
R_r	Racking ratio
S_α	Maximum response spectral acceleration (5 % damping) corresponding to the constant acceleration range of the horizontal elastic response spectrum
S_1	Spectral ordinate at 1 s of the horizontal acceleration elastic response spectrum (5 % damping)
S_{ff}	Free-field settlement
S_{Ru}	Required force to cause a unit racking deflection of the underground structure
T_0	Fundamental period of the soil deposit
T_m	Average period of the seismic action
T_B, T_C	Lower- and upper-corner periods of the constant spectral acceleration range
$T_{LS,CC}$	Return period associated to a given limit state LS and a given consequence class CC
T_{SSI}	Predominant period of the soil-structure system in the considered direction
U	Resultant of pore water pressure acting on the foundation
V_{app}	Apparent wave velocity
V_{Ed}	Design value of the horizontal shear force acting at the lower face of the foundation
V_{ff}	Shear force in the lining in absence of SSI
$V_{pl,Rd}$	Design value of the shear resistance of a member in accordance with EN 1993-1-1
$V_{Rd,1}$	Design value of the shear resistance between the horizontal base of a foundation and the ground
$V_{Rd,2}$	Design value of the shear resistance between the vertical sides of a cast-in place foundation and the ground
$V_{Rd,3}$	Design value of the horizontal component of passive earth pressure on the sides of a foundation
V_{SSI}	Shear force in the lining in presence of SSI
W	Weighting factor
W_t	Width of the transverse section of the underground structure
W_T	Weight of tunnel

3.2.1.2 Lower case Latin symbols

$a(t)$	Time-history of acceleration
a_1	Ratio (in %) of height of free-face to the distance from the free-face to the point of interest
a_2	Cumulative thickness of liquefiable layers
a_3	Average fines content in liquefiable layers
a_4	Mean grain size of granular material in the liquefiable layers
a_5	Ground slope (in %)
$\bar{a}_{eq}$	Average value of the equivalent acceleration, considering variation with depth and time
a_H	Conventional horizontal ground acceleration
c'	Cohesion of soil in terms of effective stress
$c'_{rx}, c'_{ry}, c'_{rz}$	Frequency dependent coefficients for radiation damping rocking dashpots
c_u	Undrained shear strength of soil
d	Pile diameter
d_b	Diameter of the pile longitudinal reinforcement
d_e	Foundation thickness
d_t	Tunnel diameter
f	Wall flexibility coefficient
f_s	Measured sleeve resistance in CPTs
f_{yk}	Characteristic value of yield strength of the longitudinal reinforcement in a pile
g	Acceleration of gravity
l_{pl}	Length of the critical section for detailing in yielding piles
m	Stress exponent for overburden correction in SPTs and CPTs
m_a	Added mass of water
n	Stress exponent for overburden correction in CPTs
n_s	Number of segments of a segmented tunnel
p_a	Atmospheric pressure (=100 kN/m ²)
p_w	Hydrodynamic water pressure acting on a retaining structure
q_c	Measured cone tip resistance as per EN ISO 22476-1; for sand q_c can be taken equal to q_b , the cone resistance corrected for water pressure as per EN ISO 22476-1
q_{c1N}	Tip resistance in CPTs, normalised and corrected for overburden
$q_{c1N,cs}$	Tip resistance in CPTs, normalised and corrected for overburden and fines content
q_u	Unconfined compressive strength
r_d	Reduction factor for the amplitude of the maximum cyclic shear stress with depth
r_{dt}	Depth-dependent stress reduction function for underground structures
r_L	Radius of the tunnel lining

s	Coefficient expressing the decrease of PGA_e with depth for underground structures
t_L	Thickness of the tunnel lining
u	Pore water pressure
$\dot{u}_s$	Peak particle velocity associated with shear waves
$\dot{u}_p$	Peak particle velocity associated with dilatational waves
$\dot{u}_{rp}$	Peak particle velocity associated with the dilatational component of Rayleigh waves
$\dot{u}_{rs}$	Peak particle velocity associated with the shear component of Rayleigh waves
v_p	Dilatational wave velocity
v_r	Rayleigh wave velocity
v_s	Shear wave velocity
v_{sc}	Shear wave velocity compatible with the average strain amplitude
x_L	Normalisation distance for calculation of relative displacement between two sites
z	Depth below ground surface
z_w	Depth below water table

3.2.1.3 Upper case Greek symbols

Δ	Unit deflection of the underground structure
Ω_d	Particular value of $\Omega_{d,i}$ among all $\Omega_{d,i}$ applied in the design of specific parts of a structure with $\Omega_{d,i}$ ratio of the design resistance R_{di} to the design action effect E_{di} in member i or in storey i
$\mathcal{G}_{ii}$	Correction factors for surface foundation impedances

3.2.1.4 Lower case Greek symbols

α_c	Critical seismic coefficient
α_{Ca}	Critical seismic coefficient of a mechanism that activates the resistance of anchors
α_H	Horizontal seismic coefficient
α_p	Peak acceleration of dilatational waves
α_s	Peak acceleration associated with shear waves
α_{rp}	Peak acceleration associated with the dilatational component of Rayleigh waves
α_{rs}	Peak acceleration associated with the shear component of Rayleigh waves
α_v	Vertical seismic coefficient
β_H	Coefficient reflecting the spatial variation with depth of horizontal ground motion
β_{ID}	Coefficient function of I_D for calculation of volumetric shear strain
β_{sl}	Inclination of the ground surface
γ	Weight density of the ground
$\gamma_{LS,CC}$	Performance factor associated to a given limit state LS and a given consequence class CC

γ_R	Global resistance factor
γ_{cyc}	Earthquake induced effective cyclic shear strain
γ_M	Partial factor for material strength
γ_{max}	Maximum free-field shear strain
γ_{Rd}	Overstrength factor
γ_{ref}	Reference shear strain
γ_s, γ_{sm}	Shear strain, respectively maximum shear strain in tunnels
γ_w	Weight density of water
δ	Coefficient that specifies S_δ depending on the consequence class
δ_f	Friction angle between the ground and the footing or retaining wall
δ_{ff}	Free-field racking displacement at the burial depth of the structure
δ_{str}	Structural horizontal deflection of the lining
ε_c	Extreme fibre concrete compressive strain
$\varepsilon_l, \varepsilon_{lm}$	Normal strain in the longitudinal direction and its maximum value, respectively, in tunnels and in the absence of soil-structure interaction effects
$\varepsilon_n, \varepsilon_{nm}$	Normal strain in the circumferential direction and its maximum value, respectively, in tunnels and in the absence of soil-structure interaction effects
ε_{sd}	Longitudinal reinforcement tensile strain
ε_{shear}	Liquefaction-induced free-field shear strain
ε_{ss}	Steel shell extreme fibre strain in piles
ε_{vi}	Earthquake induced volumetric strain in layer i
ς, ς_m	Curvature and maximum curvature, respectively
θ	Interstorey drift sensitivity coefficient
θ_{eq}	Apparent inclination of the gravity field in the seismic design situation
κ	Parameter reflecting the non-simultaneity of the design action effects in the ground and on the foundation
λ	Wavelength
ν	Soil Poisson's ratio
ν_L	Poisson's ratio of the tunnel lining
ξ	Damping ratio
ξ_i	Damping ratio in element i
ρ	Mass density
ρ_s	Effective volumetric ratio of confining steel = (volume of confining steel in one loop) / (volume of concrete core for a length equal to the confining steel spacing along the pile length)
σ_a	Total contact stress normal to a vertical wall for an active limit state
σ_p	Total contact stress normal to a vertical wall for a passive limit state

σ_v	Total overburden pressure, same as total vertical stress
$\sigma_{v,H}$	Total vertical stress at depth H
σ'_v	Effective overburden pressure, same as effective vertical stress
$\tau_{cy,u}$	Cyclic undrained shear strength of soil of coarse-grained soils
τ_{max}	Maximum cyclic shear stress
ϕ'	Angle of shearing resistance in terms of effective stress
χ_H	Coefficient related to the amplitude of acceptable permanent displacement induced by the ground motion
ψ	Angle between the direction of wave propagation and the tunnel axis

3.2.2 Abbreviations

CPT	Cone penetration test
EFA	Effect factor approach
MFA	Material factor approach
RFA	Resistance factor approach
SPT	Standard penetration test

3.3 S.I. Units

For the purposes of this document, the S.I. Units given in ISO 80000 apply.

For calculations, the following units are used when applicable:

— mass:	kg, t, kt
— mass density:	kg/m ³ , t/m ³ , kt/m ³
— forces and loads:	kN, kN/m, MN/m
— weight density:	N/m ³ , kN/m ³ , MN/m ³
— stresses and strengths:	Pa (=N/m ²), kPa (kN/m ²), MPa (= MN/m ²)
— moments (bending, etc.):	Nm, kNm, MNm
— time:	s
— displacement and length:	m
— velocity:	m/s
— acceleration:	m/s ²

4 Basis of design

4.1 Performance requirements

(1) For geotechnical systems, the fundamental requirements shall refer to the state of damage in the structure, herein described through the Limit States (LS) defined in EN 1998-1-1:2024, 4.3(1).

(2) For geotechnical structures, the requirements of this standard under the prescribed design seismic actions shall be applied.

4.2 Consequence classes

(1) The consequence classes of geotechnical systems should be the same as the consequence classes of the structure, as defined in the corresponding part of Eurocode 8 for the considered type of structure.

(2) The consequence of failure of geotechnical systems shall be classified according to EN 1990:2023, 4.3.

(3) Geotechnical structures shall be classified in consequence classes CC1, CC2 and CC3.

NOTE Table 4.1 (NDP) gives a classification of geotechnical structures in consequence classes CC1 to CC3, unless the National Annex gives a different classification for use in a country.

Table 4.1 (NDP) — Consequence classes for geotechnical structures

Consequence Class	Description of consequence	Examples
CC1	Lower	Retaining walls and foundations supporting buildings which are considered CC1 according to EN 1990:2023, A1.2, (e.g. an agricultural building). Man-made slopes and cuts, in areas where a failure will have low impact on the society. Minor road embankments and/or bridge foundations which, according to EN 1998-2, are not critical for communications. Underground constructions which have a low exposure and are not critical for society (e.g. small animal culvert).
CC2	Normal	All geotechnical structures not classified as CC1 or CC3.
CC3	Higher	Retaining walls and foundations supporting buildings which are considered CC3 according to EN 1990:2023, A1.2, (e.g. a hospital). Man-made slopes and cuts, retaining structures with high exposure to society or infrastructure. Major road/railway embankments, and/or bridge foundations, which, according to EN 1998-2, are of critical importance for maintaining communications, especially in the immediate post-earthquake period. Underground constructions which have a high exposure to society and have critical importance (e.g. major railway or motorway tunnel, major river culvert).

(4) The seismic action classes are those defined in EN 1998-1-1:2024, 4.1(4), depending on the seismic action index coefficient δ also defined in EN 1998-1-1:2024, 4.1(4).

(5) For geotechnical systems, the values of δ are equal to those of the structure and are given in the relevant parts of Eurocode 8.

(6) For geotechnical structures, δ should be determined.

NOTE The values of δ applicable to geotechnical structures are given in Table 4.2 (NDP), unless the National Annex gives different values for use in a country.

Table 4.2 (NDP) — δ values for geotechnical structures

	Consequence class		
	CC1	CC2	CC3
δ	0,6	1,0	1,5

4.3 Limit states and associated seismic action

(1) The state of damage should be referred to three of the limit states defined in EN 1998-1-1:2024, 4.3(1).

NOTE 1 For the geotechnical systems, the limit states are similar to those defined for the structures.

NOTE 2 The fully operational (OP) limit state, defined in EN 1998-1-1:2024, 4.3(1), is not used in this standard.

(2) For geotechnical systems, the seismic actions associated to each specified limit state should be the same as the seismic action of the structure.

NOTE The seismic action is defined numerically by a return period $T_{LS,CC}$ or by a performance factor $\gamma_{LS,CC}$.

(3) For geotechnical structures, EN 1998-1-1:2024, 4.3(3) to (5), should be applied.

NOTE 1 The values of $T_{LS,CC}$ according to EN 1998-1-1:2024, 4.3(3), are those given in Table 4.3 (NDP), unless the National Annex gives different values for use in a country.

NOTE 2 When performance factors are used according to EN 1998-1-1:2024, 4.3(5), the values of $\gamma_{LS,CC}$ are those given in Table 4.4 (NDP), unless the National Annex gives different values for use in a country.

Table 4.3 (NDP) — Return periods of seismic actions, in years, for geotechnical structures

Limit State	Consequence class		
	CC1	CC2	CC3
NC	600	1600	2500
SD	275	475	600
DL	100	115	125

Table 4.4 (NDP) — Performance factors

Limit State	Consequence class		
	CC1	CC2	CC3
NC	1,10	1,50	1,75
SD	0,80	1,00	1,10
DL	0,60	0,60	0,65

4.4 Compliance criteria

- (1) For new geotechnical systems, the relevant requirements of EN 1998-1-1:2024, 4.5, should be applied.
- (2) For new geotechnical structures, to satisfy the performance requirements, non-exceedance of the SD limit state should be verified according to the relevant clauses of the present standard.
- (3) For existing geotechnical systems, the relevant clauses of EN 1998-3 should be applied.
- (4) For existing geotechnical structures, to satisfy the performance requirements, non-exceedance of the NC limit state should be verified according to the relevant clauses of the present standard.

4.5 Methods of analysis

- (1) The seismic action effects should be calculated using either a) or b):
 - a) a force-based approach;
 - b) a displacement-based approach.
- (2) In the force-based approach, verifications should be in terms of generalized stresses.
- (3) In the displacement-based approach, compliance should be checked by comparison of calculated permanent displacements to acceptable ones.

NOTE 1 The values of the acceptable displacements can be agreed for a specific project by the relevant parties.

NOTE 2 The permanent displacement and partial factor values depend on the model and analysis method.

4.6 Verification of seismic performance

- (1) Geotechnical structures and geotechnical systems shall be verified according to Formula (4.1).

$$E_{Fd} \leq R_d \quad (4.1)$$

where

E_{Fd} is the design value of the effect of actions expressed as generalised stresses in the force-based approach, or expressed as calculated displacement in the displacement-based approach;

R_d is the design value of the resistance in the force-based approach, or the allowable displacement in the displacement-based approach.

- (2) Structural members of geotechnical structures and geotechnical systems shall be verified according to Formula (4.1), where

E_{Fd} is the design value of the effect of actions (generalised stresses in the structural member) in the force-based approach; in the displacement-based approach, it is either a generalised stress in brittle members or the deformations of the structural member in ductile ones;

R_d is the design value of the resistance of the structural member, in the same terms used for the action effect (internal force or deformations of the structural member).

- (3) Anchors employed in geotechnical structures or geotechnical systems should comply with EN 1997-3:—², Clause 8.
- (4) Members of geotechnical structures or geotechnical systems made with reinforced fill, soil nails, ground improvement, rock bolts and surface support should be verified in accordance with EN 1997-3:—², Clauses 9, 10, 11 and 13.

(5) The post-seismic static stability of geotechnical structures and geotechnical systems should be verified.

(6) The post-seismic stability should be verified using the material parameters applicable to the post-earthquake condition.

(7) Verifications in (1) and (2) that use the force-based approach may be carried out either with the material factor approach (MFA) or with the resistance factor approach (RFA), unless a specific partial factor approach is indicated in this standard.

NOTE Since in the combination of actions for the seismic design situation, given in EN 1990:2023, 8.3.4.4(1) and 8.4.3.5(1), permanent and variable actions are not factored, the effect factor approach (EFA) in EN 1997-1:2024, 8.2(9), does not apply in the seismic design situation.

(8) Verifications in (1) and (2) that use the displacement-based approach should be carried out with the material factor approach (MFA).

5 Seismic action

5.1 Definition of the seismic action

- (1) The seismic action should be defined according to EN 1998-1-1:2024, 5.2.2, depending on the type of analysis used.
- (2) Simplifications in the definition of the seismic action for geotechnical systems and geotechnical structures may be assumed as given in 5.2.

5.2 Seismic action for geotechnical systems and geotechnical structures

- (1) For geotechnical systems and geotechnical structures, the horizontal ground acceleration should be defined as given in Formula (5.1).

$$a_H = \frac{\beta_H}{\chi_H} \frac{S_\alpha}{F_A} = \frac{\beta_H}{\chi_H} PGA_e \quad (5.1)$$

where

S_α defined in EN 1998-1-1:2024, 5.2.2.2, is the maximum response spectral acceleration (for 5 % damping) corresponding to the constant acceleration range of the elastic response spectrum;

F_A defined in EN 1998-1-1:2024, 5.2.2.2, is the ratio of S_α to the zero-period spectral acceleration;

β_H is a coefficient reflecting the spatial variation with depth of the horizontal ground motion within the ground mass under consideration;

χ_H is a coefficient reflecting the amplitude of accepted permanent displacements of the soil-structure system induced by the horizontal ground motion for the considered consequence class and limit state;

PGA_e is the design peak value of horizontal ground acceleration.

NOTE 1 In force-based or displacement-based approaches, β_H varies with the model and the method of analysis and can take values $0 < \beta_H \leq 1$. It is specified accordingly in the relevant clauses of this standard.

NOTE 2 χ_H is specified in the relevant clauses of this standard. It accounts for nonlinear behaviour and depends on the type of soil-structure system and on the considered LS. In displacement-based approaches $\chi_H = 1,0$.

- (2) Alternatively, when given in this standard (e.g. for retaining structures or slopes), the horizontal ground acceleration may be expressed as a horizontal seismic coefficient α_H given by Formula (5.2).

$$\alpha_H = \frac{a_H}{g} \quad (5.2)$$

where g is the acceleration of gravity and a_H is given in Formula (5.1).

- (3) In a displacement-based approach involving calculation of permanent ground deformations, the representation of the seismic action should consist of recorded accelerograms selected according to EN 1998-1-1:2024, 5.2.3.1, or evaluated from site-specific ground response studies, in accordance with 7.5, where appropriate.

NOTE Although EN 1998-1-1 allows the use of artificial or spectrally matched accelerograms, determination of ground permanent deformations or displacements are better estimated with natural accelerograms recorded in real earthquakes, provided spurious baseline displacement trends are removed from input records for the calculation of absolute displacements.

6 Ground properties

6.1 Ground investigations

- (1) The ground investigation related to seismic design shall be performed according to EN 1997 (all parts).
- (2) Derived and representative values of ground parameters used for seismic design should be given respectively in the ground model and the geotechnical design model as defined in EN 1997 (all parts).
- (3) Site-specific investigations related to seismic response of the ground may be omitted for low seismic action classes as defined in EN 1998-1-1:2024, 4.1(4).
- (4) For moderate seismic action class as defined in EN 1998-1-1:2024, 4.1(4), data from adjacent sites likely to have similar geotechnical properties may be used as documented by geotechnical investigations.
- (5) For application of (4), existing seismic microzonation studies and maps may be considered, provided they are supported by specific geotechnical investigations at the construction site as defined in EN 1997-2.
- (6) The profile of the shear wave velocity v_s in the ground should be regarded as the most reliable indicator of the stiffness of the ground layers for seismic design.
- (7) In analyses requiring the shear wave velocity, v_s , or the small-strain shear modulus G_0 , of the ground, a direct measurement of the v_s profile should be used for moderate and high seismic action classes as defined in EN 1998-1-1:2024, 4.1(4).
- (8) For all other cases, the v_s profile may be estimated by empirical correlations using the *in situ* penetration resistance or other geotechnical properties, accounting for the scatter of such correlations. In that case, the coefficient of variation taken for v_s should not be smaller than 0,3.
- (9) In addition to the shear wave velocity, v_s , additional investigations in a) or b) should be considered for the purpose of this standard:
 - a) Standard penetration tests (SPTs) and/or static cone penetration tests (CPTs), in accordance with EN 1997-2;
 - b) Cyclic and dynamic laboratory tests, in accordance with EN 1997-2.

6.2 Water levels

- (1) The water level to be considered in the seismic design situation should be equal to its quasi-permanent value, as defined in EN 1990:2023, 6.1.2.3(1) and 6.1.3.2(4).

6.3 Strength parameters

- (1) In the seismic design situation, saturated soils should be considered to behave under undrained conditions.
- (2) Only pervious materials with coefficient of permeability larger than 5×10^{-3} m/s may be considered to behave in drained conditions.
- (3) To comply with (1), the soil undrained behaviour may be studied in terms of total stresses, or in terms of effective stresses with due account of the pore water pressure.

(4) In terms of total stresses, the value of the soil strength parameters applicable under static undrained conditions may be used, considering a) or b):

- a) For fine-grained soils, the appropriate strength parameter should be the undrained shear strength c_u ; its value should account for cyclic degradation effects under long duration earthquake actions;
- b) For coarse-grained soil, the appropriate strength parameter should be the cyclic undrained shear strength $\tau_{cy,u}$.

NOTE With reference to Table 5.6 of EN 1998-1-1:2024, long duration earthquake actions are earthquakes with $D_R \geq 12$ s.

(5) The potential increase in c_u due to the rapid rate of loading may be considered based on laboratory data.

(6) Where potentially sensitive clays are present, the reduction in c_u due to large strain cycles may be estimated based on literature data. For high seismic action class, it should be based on laboratory data.

(7) As an alternative to (4), effective strength parameters with appropriate pore water pressure generated during cyclic loading may be used.

(8) For rocks, strength properties may be defined based on the unconfined compressive strength, q_u .

6.4 Stiffness and energy dissipation properties

(1) Due to its influence on the design values of the seismic actions, the main stiffness parameter of the ground under seismic actions should be the secant shear modulus G . The small strain stiffness value, G_0 , should be taken from Formula (6.1) (see also EN 1997-2:2024, 10.4.1(1)).

$$G_0 = \rho v_s^2 \quad (6.1)$$

where

ρ is the mass density;

v_s is the small strain shear wave propagation velocity of the ground.

(2) In calculations involving dynamic ground properties, the difference between the small-strain values G_0 , such as those derived from measured v_s , and the secant shear stiffness values, G , compatible with the strain amplitudes induced by the seismic design motion should be considered.

NOTE See EN 1997-2:2024, Annex F, F.2 and F.3.

(3) When appropriate, energy dissipation caused by inelastic behaviour under cyclic loading should be considered as an additional ground property.

NOTE Energy dissipation caused by inelastic behaviour is often modelled as linear visco-elastic with frequency-independent hysteretic damping ratio ξ .

(4) The variation of the damping ratio ξ with the strain amplitudes induced by the seismic design motion should be taken into account in analyses involving dynamic ground properties.

NOTE See EN 1997-2:2024, Annex F, F.2 and F.3.

(5) In the absence of specific data, the values given in Table 6.1, for materials with plasticity index $I_p < 50$, may be used in order to comply with (2) and (4).

NOTE The values in Table 6.1 are indicative. More accurate values can be obtained from published data or specific analyses.

Table 6.1 — Damping ratios and average reduction factors ($\pm$ one standard deviation) of the normalized secant shear modulus G/G_0 within 20 m depth

Seismicity level	$150 \leq v_s < 250$ m/s		$250 \leq v_s < 400$ m/s		$400 \leq v_s < 800$ m/s		800 m/s $\leq v_s$	
	G/G_0	ξ	G/G_0	ξ	G/G_0	ξ	G/G_0	ξ
Very low	0,70 ($\pm 0,08$)	0,04	0,80 ($\pm 0,09$)	0,03	1,00	0,03	1,00	0,02
Low	0,50 ($\pm 0,14$)	0,07	0,65 ($\pm 0,16$)	0,05	0,80 ($\pm 0,10$)	0,03	1,00	0,02
Moderate	0,30 ($\pm 0,10$)	0,10	0,50 ($\pm 0,20$)	0,07	0,70 ($\pm 0,10$)	0,05	1,00	0,02
High	0,20 ($\pm 0,10$)	0,20	0,40 ($\pm 0,20$)	0,12	0,60 ($\pm 0,20$)	0,10	0,90 ($\pm 0,10$)	0,02
NOTE 1 The seismicity level is defined in Table 5.2 of EN 1998-1-1:2024.								
NOTE 2 G_0 is the best estimate value at small strains ($<10^{-5}$), see also EN 1997-2:2024, 9.1.4 and Annex F.								

(6) Except for moderate and high seismicity levels, the values in Table 6.1 listed for $150 \text{ m/s} \leq v_s < 250 \text{ m/s}$ may also be adopted for the case of very soft soils with $100 \text{ m/s} < v_s \leq 150 \text{ m/s}$.

(7) In application of (5), uncertainty should be accounted for by considering the plus or minus one standard deviation ranges (given in parenthesis in Table 6.1), depending on such factors as stiffness and layering of the ground profile, whichever is the most unfavourable.

NOTE Uncertainty on the damping ratio values is larger than on G/G_0 ; the values are given in Table 6.1 are safe-sided.

6.5 Partial factors and design cases

(1) In the verifications expressed by Formula (4.1) that use the material factor approach (MFA), partial factors γ_M should be applied to the ground strength parameters.

(2) The partial factors γ_M which should be applied on the ground strength properties for c_u , $\tau_{cy,u}$, q_u , c' , $\tan\phi'$ and $\tan\delta_f$ are denoted by γ_{cu} , $\gamma_{\tau_{cy,u}}$, γ_{qu} , $\gamma_{c'}$, $\gamma_{\tan\phi'}$ and $\gamma_{\tan\delta}$ respectively.

NOTE The values assigned to γ_{cu} , $\gamma_{\tau_{cy,u}}$, γ_{qu} , $\gamma_{c'}$, $\gamma_{\tan\phi'}$, and $\gamma_{\tan\delta}$ in the seismic design situation for the SD and NC limit states are $\gamma_{cu} = 1,0$, $\gamma_{\tau_{cy,u}} = 1,25$, $\gamma_{qu} = 1,0$, $\gamma_{c'} = 1,0$, $\gamma_{\tan\phi'} = 1,0$ and $\gamma_{\tan\delta} = 1,0$, unless different values for use in a country are specified in the National Annex.

(3) In the verifications expressed by Formula (4.1) that use the resistance factor approach (RFA), partial factors γ_R should be applied to the resistance according to EN 1990:2023, 8.3.5.3, Formula (8.20).

NOTE The values assigned to γ_R are equal to 1,0, unless different values are given in this standard or different values for use in a country are specified in the National Annex.

(4) In the force-based approach, the design cases to use for the verifications should be those given in EN 1997-3, except when specifically indicated in this standard.

7 Evaluation of the seismic response of the construction site

7.1 Siting

7.1.1 General

(1) An assessment of the stability of the construction site shall be carried out with respect to the hazards related to fault rupture, slope instability, liquefaction, lateral spreading, and excessive soil settlement in the event of an earthquake.

NOTE 7.1 to 7.4 give guidance on how structures can be analysed against these seismic hazards.

7.1.2 Potentially active seismic faults

(1) If the conditions given in EN 1998-1-1:2024, 5.1.1(5)a) and 5.1.1(5)c), are met, structures of consequence classes CC2 and CC3 may be constructed in the vicinity of potentially active faults, only if a) and b) are satisfied:

- a) a continuous stiff foundation is provided;
- b) the soil cover exceeds a certain thickness H_{cov} .

NOTE A seismically active fault is determined either from an official map, or from a specific tectonic investigation certified by a recognized National Authority.

(2) The depth H_{cov} defined in (1) may be taken from Figure 7.1.

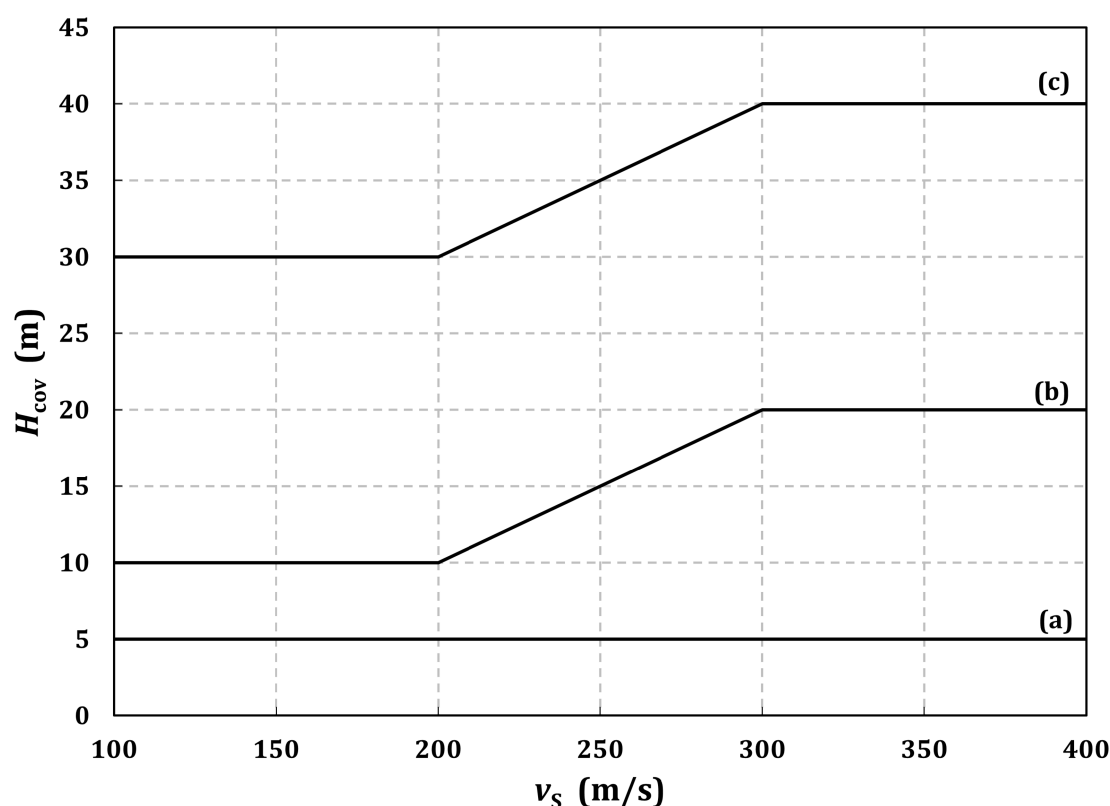


Figure 7.1 — Thickness H_{cov} of minimum allowed soil cover versus average soil shear wave velocity v_s within the depth of influence of the foundation: (a) low seismic action class; (b) moderate seismic action class; (c) high seismic action class

NOTE The average shear wave velocity, v_s , can be calculated according to EN 1998-1-1:2024, 5.1.2(7), with H equal to the depth of influence of the foundation.

(3) The foundations of structures of consequence classes CC2 and CC3 satisfying (2), constructed in the vicinity of tectonic faults, should be designed to minimize the effect of faulting on the performance of the structure.

NOTE 1 The vicinity of a potential fault means a relative broad region (of the order of a few hundred meters) from a hypothesized (or even previously activated) fault line.

NOTE 2 To satisfy (3), design of appropriate foundations can include interventions in the soil under and/or surrounding the structure.

(4) Bearing piles should not be designed to cross the potential fault plane, and their tip should be located at least 10 diameters above this plane.

(5) For extended structures, like bridges, underground structures and tunnels, in the vicinity of potentially active faults, special provisions should be taken to design adequately stiff foundations and/or to accommodate differential displacements.

NOTE For tunnels, provisions are given in Clause 11.

(6) The simultaneous effects of fault rupture with structural vibrations due to ground shaking may be neglected.

7.2 Slope stability

7.2.1 General

(1) A verification of slope stability should be carried out for structures to be constructed on or near natural or man-made slopes, in order to ensure the safety and/or serviceability of the structure in the seismic design situation. Verification should also be carried out in situations where slope instability itself results in an unsafe situation, for instance in the case of levees.

NOTE See also EN 1997-3:—², Clause 4.

(2) When slope instability can affect an adjacent structure, the consequence class and the limit states to be considered for the slope should be taken the same as those of the structure.

(3) The limit states for slopes should be associated to acceptable permanent ground displacements, compatible with the limit states considered for the structure.

(4) Modelling the mechanical behaviour of the ground under the combined static and seismic actions should be done in accordance with 6.3.

7.2.2 Methods of analysis

7.2.2.1 General

(1) The analysis of a slope may be carried out with a force-based approach, in accordance with 7.2.2.2, whereby the seismic action is applied using the pseudo-static method.

(2) If an evaluation of permanent displacements is required, a displacement-based approach should be used in accordance with 7.2.2.3.

(3) Except for high seismic action class, the vertical component of the seismic action may be neglected.

7.2.2.2 Force-based approach

(1) A force-based approach should not be carried out when there is significant reduction in soil strength, as defined in 6.3(4) to (7), or potential for liquefaction.

(2) The seismic demand for the slope should be expressed by a horizontal seismic coefficient α_H defined in 5.2.

(3) Coefficient β_H in Formula (5.1) should be related to the slope height and predominant wavelength of the seismic motion.

NOTE Annex A provides guidance for the calculation of β_H .

(4) Coefficient $\chi_H \geq 1,0$ in Formula (5.1) should be related to the accepted permanent ground displacements and may take different values depending on the considered limit state (DL, SD or NC).

(5) In a force-based approach, the design value of the action effects should be calculated for the relevant limit state using Formula (5.1) with the values for χ_H given in Table 7.1, which indicatively correspond to the given range of permanent displacements.

Table 7.1 — Values of χ_H for slope stability analyses

χ_H	1,5	2,0	2,5
Range of permanent displacements (mm)	30–50	60–100	120–200
NOTE Values of χ_H in Table 7.1 are calibrated for the recommended values of material factors and global resistance factors. Values of χ_H for other values of the material factors or global resistance factors are not provided in this standard.			

(6) For high seismic action class, the equivalent vertical static seismic forces using a vertical seismic coefficient as given in Formula (7.1) should be used.

$$\alpha_V = \pm 0,5 \alpha_H \quad (7.1)$$

(7) The safety verification of the ground slope should be performed with the material factor approach (MFA), using the partial factors defined in 6.5 and should consider the least safe potential slip surface.

(8) The seismic resistance of the slope should be expressed by its critical seismic coefficient α_c , defined as the minimum value of the horizontal seismic coefficient that leads to pseudo-static failure.

7.2.2.3 Displacement-based approach

(1) Displacement-based approaches should be used when an evaluation of displacements is needed.

(2) The performance of a slope should be evaluated based on acceptable permanent displacements for the considered limit states.

NOTE Acceptable permanent displacements depend on the environment of the slope like the presence of adjacent structures.

(3) The permanent displacement of a slope due to the seismic action may be calculated either by means of established rigid block models or by nonlinear dynamic analysis with modelling of the ground domain.

NOTE 1 For nonlinear dynamic analyses, a representative assessment of the initial state of stresses is essential.

NOTE 2 A rigid-block model cannot be used where there is significant reduction in soil strength unless the residual soil shear strength is used and the block is stable under gravity loads.

(4) In rigid-block analyses, the effects of ground deformability and damping may be considered with simplified uncoupled analyses.

NOTE Uncoupled analyses are analyses in which the applied seismic action is not affected by the system displacements.

(5) Acceleration time-histories used in a displacement-based approach should be selected according to 5.2(3).

(6) The seismic demand of a slope should be expressed as its permanent displacement produced by the seismic action, calculated as in (3).

(7) The seismic capacity of a slope should be expressed as the maximum acceptable permanent displacement as defined in (2).

7.3 Potentially liquefiable soils

7.3.1 General

(1) Liquefaction assessment should be performed for the free-field site conditions (ground surface elevation, ground water level) prevailing during the design service life of the structure.

NOTE Liquefaction is hereinafter referred to as a substantial decrease in the shear strength of the soil and/or as a transient softening of the soil caused by the increase in pore water pressures in saturated materials during the propagation of shear waves, such as to give rise to significant permanent deformations or even to a condition of near-zero effective stress in the soil.

(2) Liquefaction assessment may be neglected for magnitudes smaller than M_{WT} .

NOTE 1 The value assigned to the threshold magnitude is $M_{WT} = 5,0$, unless a different value for use in a country is specified in the National Annex.

NOTE 2 In absence of site-specific hazard studies, Table 5.6 in EN 1998-1-1:2024 provides conventional magnitude values.

(3) The relevant data for liquefaction assessment should be the cyclic undrained shear strength as defined in 6.3(4) and soil classification as determined by grain size distribution or other state properties (see EN 1997-2:2024, Clause 7).

(4) The cyclic undrained shear strength in (3) may be preferably obtained by correlations with *in situ* standard penetration tests (SPTs) or cone penetration tests (CPTs). Laboratory tests (EN 1997-2:2024, Clause 10) may also be used.

NOTE *In situ*-based methods are preferred over laboratory-based methods for liquefaction assessment because of the difficulty and high cost associated with securing high-quality, undisturbed samples for laboratory testing that satisfactorily represent *in situ* conditions.

7.3.2 Consideration of site conditions

(1) For structures on foundations other than piles, in low seismic action classes, the consequences of liquefaction may be ignored if liquefaction is found at depths greater than 15 m below the foundation base.

(2) Soils that should be evaluated for liquefaction susceptibility include gravels, sands, gravelly sands, silts, mine tailings, and fine-grained soils with plasticity index not greater than 15. In soil layers with a clay fraction greater than 15 %, evaluation of liquefaction susceptibility may be omitted regardless of the plasticity index.