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Eurocode 8: Design of structures for earthquake resistance - Part 5: Foundations, retaining structures and geotechnical aspects

Eurocode 8: Calcul des structures pour leur résistance aux séismes - Partie 5: Fondations, ouvrages de soutènement et aspects géotechniques

Eurocode 8 : Auslegung von Bauwerken gegen Erdbeben - Teil 5: Gründungen, Stützbauwerke und geotechnische Aspekte

This draft European Standard is submitted to CEN members for formal vote. It has been drawn up by the Technical Committee CEN/TC 250.

If this draft becomes a European Standard, CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration.

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Foreword

This document (EN 1998–5:2003) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held by BSI.

This European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by MM 200Y, and conflicting national standards shall be withdrawn at the latest by MM 20YY.

This document supersedes ENV 1998–5:1994.

CEN/TC 250 is responsible for all Structural Eurocodes.

Background of the Eurocode programme

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

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

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

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

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

EN 1990	Eurocode :	Basis of Structural Design
EN 1991	Eurocode 1:	Actions on structures
EN 1992	Eurocode 2:	Design of concrete structures
EN 1993	Eurocode 3:	Design of steel structures

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

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

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

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes:

– as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire ;

– as a basis for specifying contracts for construction works and related engineering services ;

– as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

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

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

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

[3] According to Art. 12 of the CPD the interpretative documents shall :

a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;

b) indicate methods of correlating these classes or levels of requirement with the technical specifications, *e.g.* methods of calculation and of proof, technical rules for project design, etc. ;

c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

National Standards implementing Eurocodes

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

The National annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e. :

– values and/or classes where alternatives are given in the Eurocode,

– values to be used where a symbol only is given in the Eurocode,

– country specific data (geographical, climatic, etc.), *e.g.* snow map,

– the procedure to be used where alternative procedures are given in the Eurocode.

It may also contain

– decisions on the application of informative annexes,

– references to non-contradictory complementary information to assist the user to apply the Eurocode.

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

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

Additional information specific to EN 1998-5

The scope of Eurocode 8 is defined in EN 1998-1:2004, **1.1.1** and the scope of this Part of Eurocode 8 is defined in **1.1**. Additional Parts of Eurocode 8 are listed in EN 1998-1:2004, **1.1.3**.

EN 1998-5:2004 is intended for use by:

- clients (e.g. for the formulation of their specific requirements on reliability levels and durability) ;
- designers and constructors ;
- relevant authorities.

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

For the design of structures in seismic regions the provisions of this European Standard are to be applied in addition to the provisions of the other relevant parts of Eurocode 8 and the other relevant Eurocodes. In particular, the provisions of this European Standard complement those of EN 1997-1:2004, which do not cover the special requirements of seismic design.

Owing to the combination of uncertainties in seismic actions and ground material properties, Part 5 may not cover in detail every possible design situation and its proper use may require specialised engineering judgement and experience.

National annex for EN 1998-5

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1998-5 should have a National annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1998-5:2004 through clauses:

Reference	**Item**
1.1 (4)	Informative Annexes A, C, D and F
3.1 (3)	Partial factors for material properties
4.1.4 (11)	Upper stress limit for susceptibility to liquefaction
5.2 (2)c	Reduction of peak ground acceleration with depth from ground surface

1 GENERAL

1.1 Scope

(1)P This Part of Eurocode 8 establishes the requirements, criteria, and rules for the siting and foundation soil of structures for earthquake resistance. It covers the design of different foundation systems, the design of earth retaining structures and soil-structure interaction under seismic actions. As such it complements Eurocode 7 which does not cover the special requirements of seismic design.

(2)P The provisions of Part 5 apply to buildings (EN 1998-1), bridges (EN 1998-2), towers, masts and chimneys (EN 1998-6), silos, tanks and pipelines (EN 1998-4).

(3)P Specialised design requirements for the foundations of certain types of structures, when necessary, shall be found in the relevant Parts of Eurocode 8.

(4) Annex B of this Eurocode provides empirical charts for simplified evaluation of liquefaction potential, while Annex E gives a simplified procedure for seismic analysis of retaining structures.

NOTE 1 Informative Annex A provides information on topographic amplification factors.

NOTE 2 Informative Annex C provides information on the static stiffness of piles.

NOTE 3 Informative Annex D provides information on dynamic soil-structure interaction.

NOTE 4 Informative Annex F provides information on the seismic bearing capacity of shallow foundations.

1.2 Normative references

(1)P This European Standard incorporates by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies (including amendments).

1.2.1 General reference standards

EN 1990	Eurocode - Basis of structural design
EN 1997-1	Eurocode 7 - Geotechnical design – Part 1: General rules
EN 1997-2	Eurocode 7 - Geotechnical design – Part 2: Design assisted by laboratory and field testing
EN 1998-1	Eurocode 8 - Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings
EN 1998-2	Eurocode 8 - Design of structures for earthquake resistance – Part 2: Bridges

EN 1998-4 Eurocode 8 - Design of structures for earthquake resistance – Part 4: Silos, tanks and pipelines

EN 1998-6 Eurocode 8 - Design of structures for earthquake resistance – Part 6: Towers, masts and chimneys

1.3 Assumptions

(1)P The general assumptions of EN 1990:2002, **1.3** apply.

1.4 Distinction between principles and applications rules

(1)P The rules of EN 1990:2002, **1.4** apply.

1.5 Terms and definitions

1.5.1 Terms common to all Eurocodes

(1)P The terms and definitions given in EN 1990:2002, **1.5** apply.

(2)P EN 1998-1:2004, **1.5.1** applies for terms common to all Eurocodes.

1.5.2 Additional terms used in the present standard

(1)P The definition of ground found in EN 1997-1:2004, **1.5.2** applies while that of other geotechnical terms specifically related to earthquakes, such as liquefaction**,** are given in the text.

(2) For the purposes of this standard the terms defined in EN 1998-1:2004, **1.5.2** apply.

1.6 Symbols

(1) For the purposes of this European Standard the following symbols apply. All symbols used in Part 5 are defined in the text when they first occur, for ease of use. In addition, a list of the symbols is given below. Some symbols occurring only in the annexes are defined therein:

E_d Design action effect

E_{pd} Lateral resistance on the side of footing due to passive earth pressure

ER Energy ratio in Standard Penetration Test (SPT)

F_H Design seismic horizontal inertia force

F_V Design seismic vertical inertia force

F_{Rd} Design shear resistance between horizontal base of footing and the ground

G Shear modulus

G_{max} Average shear modulus at small strain

L_e Distance of anchors from wall under dynamic conditions

L_s Distance of anchors from wall under static conditions

M_{Ed} Design action in terms of moments

$N_1(60)$ SPT blowcount value normalised for overburden effects and for energy ratio

N_{Ed} Design normal force on the horizontal base

N_{SPT} Standard Penetration Test (SPT) blowcount value

PI Plasticity Index of soil

R_d Design resistance of the soil

S Soil factor defined in EN 1998-1:2004, **3.2.2.2**

S_T Topography amplification factor

V_{Ed} Design horizontal shear force

W Weight of sliding mass

a_g Design ground acceleration on type A ground ($a_g = \gamma_I\, a_{gR}$)

a_{gR} Reference peak ground acceleration on type A ground

a_{vg} Design ground acceleration in the vertical direction

c' Cohesion of soil in terms of effective stress

c_u Undrained shear strength of soil

d Pile diameter

d_r Displacement of retaining walls

g Acceleration of gravity

k_h Horizontal seismic coefficient

k_v Vertical seismic coefficient

q_u Unconfined compressive strength

r Factor for the calculation of the horizontal seismic coefficient (Table 7.1)

v_s Velocity of shear wave propagation

$v_{s,max}$ Average v_s value at small strain ($< 10^{-5}$)

α Ratio of the design ground acceleration on type A ground, a_g, to the acceleration of gravity g

γ Unit weight of soil

γ_d Dry unit weight of soil

γ_I Importance factor

γ_M Partial factor for material property

γ_{Rd} Model partial factor

γ_w Unit weight of water

δ Angle of shearing resistance between the ground and the footing or retaining wall

ϕ' Angle of shearing resistance in terms of effective stress

ρ Unit mass

σ_{vo} Total overburden pressure, same as total vertical stress

σ'_{vo} Effective overburden pressure, same as effective vertical stress

$\tau_{cy,u}$ Cyclic undrained shear strength of soil

τ_e Seismic shear stress

1.7 S.I. Units

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

(2) In addition the units recommended in EN 1998-1:2004, **1.7** apply.

NOTE For geotechnical calculations, reference should be made to EN 1997-1:2004, **1.6** (2).

2 SEISMIC ACTION

2.1 Definition of the seismic action

(1)P The seismic action shall be consistent with the basic concepts and definitions given in EN 1998-1:2004, **3.2** taking into account the provisions given in **4.2.2**.

(2)P Combinations of the seismic action with other actions shall be carried out according to EN 1990:2002, **6.4.3.4** and EN 1998-1:2004, **3.2.4.**

(3) Simplifications in the choice of the seismic action are introduced in this European Standard wherever appropriate.

2.2 Time-history representation

(1)P If time-domain analyses are performed, both artificial accelerograms and real strong motion recordings may be used. Their peak value and frequency content shall be as specified in EN 1998-1:2004, **3.2.3.1**.

(2) In verifications of dynamic stability involving calculations of permanent ground deformations the excitation should preferably consist of accelerograms recorded on soil sites in real earthquakes, as they possess realistic low frequency content and proper time correlation between horizontal and vertical components of motion. The strong motion duration should be selected in a manner consistent with EN 1998-1:2004, **3.2.3.1**.

3 GROUND PROPERTIES

3.1 Strength parameters

(1) The value of the soil strength parameters applicable under static undrained conditions may generally be used. For cohesive soils the appropriate strength parameter is the undrained shear strength c_u, adjusted for the rapid rate of loading and cyclic degradation effects under the earthquake loads when such an adjustment is needed and justified by adequate experimental evidence. For cohesionless soil the appropriate strength parameter is the cyclic undrained shear strength $\tau_{cy,u}$ which should take the possible pore pressure build-up into account.

(2) Alternatively, effective strength parameters with appropriate pore water pressure generated during cyclic loading may be used. For rocks the unconfined compressive strength, q_u , may be used.

(3) The partial factors (γ_M) for material properties c_u, $\tau_{cy,u}$ and q_u are denoted as γ_{cu}, $\gamma_{\tau cy}$ and γ_{qu}, and those for tan ϕ' are denoted as $\gamma_{\phi'}$.

NOTE The values ascribed to γ_{cu}, $\gamma_{\tau cy}$, γ_{qu}, and $\gamma_{\phi'}$ for use in a country may be found in its National Annex. The recommended values are γ_{cu} = 1,4, $\gamma_{\tau cy}$ = 1,25, γ_{qu} = 1,4, and $\gamma_{\phi'}$ = 1,25.

3.2 Stiffness and damping parameters

(1) Due to its influence on the design seismic actions, the main stiffness parameter of the ground under earthquake loading is the shear modulus G, given by

$$G = \rho v_s^2 \tag{3.1}$$

where ρ is the unit mass and v_s is the shear wave propagation velocity of the ground.

(2) Criteria for the determination of v_s, including its dependence on the soil strain level, are given in **4.2.2** and **4.2.3**.

(3) Damping should be considered as an additional ground property in the cases where the effects of soil-structure interaction are to be taken into account, specified in Section **6**.

(4) Internal damping, caused by inelastic soil behaviour under cyclic loading, and radiation damping, caused by seismic waves propagating away from the foundation, should be considered separately.

4 REQUIREMENTS FOR SITING AND FOR FOUNDATION SOILS

4.1 Siting

4.1.1 General

(1)P An assessment of the site of construction shall be carried out to determine the nature of the supporting ground to ensure that hazards of rupture, slope instability, liquefaction, and high densification susceptibility in the event of an earthquake are minimised.

(2)P The possibility of these adverse phenomena occurring shall be investigated as specified in the following subclauses.

4.1.2 Proximity to seismically active faults

(1)P Buildings of importance classes II, III, IV defined in EN 1998-1:2004, **4.2.5,** shall not be erected in the immediate vicinity of tectonic faults recognised as being seismically active in official documents issued by competent national authorities.

(2) An absence of movement in the Late Quaternary may be used to identify non active faults for most structures that are not critical for public safety.

(3)P Special geological investigations shall be carried out for urban planning purposes and for important structures to be erected near potentially active faults in areas of high seismicity, in order to determine the ensuing hazard in terms of ground rupture and the severity of ground shaking.

4.1.3 Slope stability

4.1.3.1 General requirements

(1)P A verification of ground stability shall be carried out for structures to be erected on or near natural or artificial slopes, in order to ensure that the safety and/or serviceability of the structures is preserved under the design earthquake.

(2)P Under earthquake loading conditions, the limit state for slopes is that beyond which unacceptably large permanent displacements of the ground mass take place within a depth that is significant both for the structural and functional effects on the structures.

(3) The verification of stability may be omitted for buildings of importance class I if it is known from comparable experience that the ground at the construction site is stable.

4.1.3.2 Seismic action

(l)P The design seismic action to be assumed for the verification of stability shall conform to the definitions given in **2.1**.

(2)P An increase in the design seismic action shall be introduced, through a topographic amplification factor, in the ground stability verifications for structures with importance factor γ_I greater than 1,0 on or near slopes.

NOTE Some guidelines for values of the topographic amplification factor are given in Informative Annex A.

(3) The seismic action may be simplified as specified in **4.1.3.3**.

4.1.3.3 Methods of analysis

(1)P The response of ground slopes to the design earthquake shall be calculated either by means of established methods of dynamic analysis, such as finite elements or rigid block models, or by simplified pseudo-static methods subject to the limitations of (3) and (8) of this subclause.

(2)P In modelling the mechanical behaviour of the soil media, the softening of the response with increasing strain level, and the possible effects of pore pressure increase under cyclic loading shall be taken into account.

(3) The stability verification may be carried out by means of simplified pseudo-static methods where the surface topography and soil stratigraphy do not present very abrupt irregularities.

(4) The pseudo-static methods of stability analysis are similar to those indicated in EN 1997-1:2004, **11.5**, except for the inclusion of horizontal and vertical inertia forces applied to every portion of the soil mass and to any gravity loads acting on top of the slope.

(5)P The design seismic inertia forces F_H and F_V acting on the ground mass, for the horizontal and vertical directions respectively, in pseudo-static analyses shall be taken as:

$$F_H = 0,5\alpha \cdot S \cdot W \tag{4.1}$$

$$F_V = \pm 0,5 F_H \text{ if the ratio } a_{vg}/a_g \text{ is greater than 0,6} \tag{4.2}$$

$$F_V = \pm 0,33 F_H \text{ if the ratio } a_{vg}/a_g \text{ is not greater than 0,6} \tag{4.3}$$

where

α is the ratio of the design ground acceleration on type A ground, a_g, to the acceleration of gravity g;

a_{vg} is the design ground acceleration in the vertical direction;

a_g is the design ground acceleration for type A ground;

S is the soil parameter of EN 1998-1:2004, **3.2.2.2**;

W is the weight of the sliding mass.

A topographic amplification factor for a_g shall be taken into account according to **4.1.3.2** (2).

(6)P A limit state condition shall then be checked for the least safe potential slip surface.

(7) The serviceability limit state condition may be checked by calculating the permanent displacement of the sliding mass by using a simplified dynamic model consisting of a rigid block sliding against a friction force on the slope. In this model the seismic action should be a time history representation in accordance with **2.2** and based on the design acceleration without reductions.

(8)P Simplified methods, such as the pseudo-static simplified methods mentioned in (3) to (6)P in this subclause, shall not be used for soils capable of developing high pore water pressures or significant degradation of stiffness under cyclic loading.

(9) The pore pressure increment should be evaluated using appropriate tests. In the absence of such tests, and for the purpose of preliminary design, it may be estimated through empirical correlations.

4.1.3.4 Safety verification for the pseudo-static method

(1)P For saturated soils in areas where $\alpha \cdot S > 0{,}15$, consideration shall be given to possible strength degradation and increases in pore pressure due to cyclic loading subject to the limitations stated in **4.1.3.3** (8).

(2) For quiescent slides where the chances of reactivation by earthquakes are higher, large strain values of the ground strength parameters should be used. In cohesionless materials susceptible to cyclic pore-pressure increase within the limits of **4.1.3.3**, the latter may be accounted for by decreasing the resisting frictional force through an appropriate pore pressure coefficient proportional to the maximum increment of pore pressure. Such an increment may be estimated as indicated in **4.1.3.3** (9).

(3) No reduction of the shear strength need be applied for strongly dilatant cohesionless soils, such as dense sands.

(4)P The safety verification of the ground slope shall be executed according to the principles of EN 1997-1:2004.

4.1.4 Potentially liquefiable soils

(1)P A decrease in the shear strength and/or stiffness caused by the increase in pore water pressures in saturated cohesionless materials during earthquake ground motion, such as to give rise to significant permanent deformations or even to a condition of near-zero effective stress in the soil, shall be hereinafter referred to as liquefaction.

(2)P An evaluation of the liquefaction susceptibility shall be made when the foundation soils include extended layers or thick lenses of loose sand, with or without silt/clay fines, beneath the water table level, and when the water table level is close to the ground surface. This evaluation shall be performed for the free-field site conditions (ground surface elevation, water table elevation) prevailing during the lifetime of the structure.

(3)P Investigations required for this purpose shall as a minimum include the execution of either in situ Standard Penetration Tests (SPT) or Cone Penetration Tests (CPT), as well as the determination of grain size distribution curves in the laboratory.

(4)P For the SPT, the measured values of the blowcount N_{SPT}, expressed in blows/30 cm, shall be normalised to a reference effective overburden pressure of 100 kPa and to a ratio of impact energy to theoretical free-fall energy of 0,6. For depths of less than 3 m, the measured N_{SPT} values should be reduced by 25%.

(5) Normalisation with respect to overburden effects may be performed by multiplying the measured N_{SPT} value by the factor $(100/\sigma'_{vo})^{1/2}$, where σ'_{vo} (kPa) is the effective overburden pressure acting at the depth where the SPT measurement has been made, and at the time of its execution. The normalisation factor $(100/\sigma'_{vo})^{1/2}$ should be taken as being not smaller than 0,5 and not greater than 2.

(6) Energy normalisation requires multiplying the blowcount value obtained in (5) of this subclause by the factor $ER/60$, where ER is one hundred times the energy ratio specific to the testing equipment.

(7) For buildings on shallow foundations, evaluation of the liquefaction susceptibility may be omitted when the saturated sandy soils are found at depths greater than 15 m from ground surface.

(8) The liquefaction hazard may be neglected when $\alpha \cdot S < 0{,}15$ and at least one of the following conditions is fulfilled:

- the sands have a clay content greater than 20% with plasticity index $PI > 10$;
- the sands have a silt content greater than 35% and, at the same time, the SPT blowcount value normalised for overburden effects and for the energy ratio $N_1(60) > 20$;
- the sands are clean, with the SPT blowcount value normalised for overburden effects and for the energy ratio $N_1(60) > 30$.

(9)P If the liquefaction hazard may not be neglected, it shall as a minimum be evaluated by well-established methods of geotechnical engineering, based on field correlations between in situ measurements and the critical cyclic shear stresses known to have caused liquefaction during past earthquakes.

(10) Empirical liquefaction charts illustrating the field correlation approach under level ground conditions applied to different types of in situ measurements are given in Annex B. In this approach, the seismic shear stress τ_e, may be estimated from the simplified expression

$$\tau_e = 0{,}65\ \alpha \cdot S \cdot \sigma_{vo} \tag{4.4}$$

where σ_{vo} is the total overburden pressure and the other variables are as in expressions (4.1) to (4.3). This expression may not be applied for depths larger than 20 m.

(11)P If the field correlation approach is used, a soil shall be considered susceptible to liquefaction under level ground conditions whenever the earthquake-induced shear stress exceeds a certain fraction λ of the critical stress known to have caused liquefaction in previous earthquakes.

NOTE The value ascribed to λ for use in a Country may be found in its National Annex. The recommended value is $\lambda = 0,8$, which implies a safety factor of 1,25.

(12)P If soils are found to be susceptible to liquefaction and the ensuing effects are deemed capable of affecting the load bearing capacity or the stability of the foundations, measures, such as ground improvement and piling (to transfer loads to layers not susceptible to liquefaction), shall be taken to ensure foundation stability.

(13) Ground improvement against liquefaction should either compact the soil to increase its penetration resistance beyond the dangerous range, or use drainage to reduce the excess pore-water pressure generated by ground shaking.

NOTE The feasibility of compaction is mainly governed by the fines content and depth of the soil.

(14) The use of pile foundations alone should be considered with caution due to the large forces induced in the piles by the loss of soil support in the liquefiable layer or layers, and to the inevitable uncertainties in determining the location and thickness of such a layer or layers.

4.1.5 Excessive settlements of soils under cyclic loads

(l)P The susceptibility of foundation soils to densification and to excessive settlements caused by earthquake-induced cyclic stresses shall be taken into account when extended layers or thick lenses of loose, unsaturated cohesionless materials exist at a shallow depth.

(2) Excessive settlements may also occur in very soft clays because of cyclic degradation of their shear strength under ground shaking of long duration.

(3) The densification and settlement potential of the previous soils should be evaluated by available methods of geotechnical engineering having recourse, if necessary, to appropriate static and cyclic laboratory tests on representative specimens of the investigated materials.

(4) If the settlements caused by densification or cyclic degradation appear capable of affecting the stability of the foundations, consideration should be given to ground improvement methods.

4.2 Ground investigation and studies

4.2.1 General criteria

(1)P The investigation and study of foundation materials in seismic areas shall follow the same criteria adopted in non-seismic areas, as defined in EN 1997-1:2004, Section **3**.

(2) With the exception of buildings of importance class I, cone penetration tests, possibly with pore pressure measurements, should be included whenever feasible in the

field investigations, since they provide a continuous record of the soil mechanical characteristics with depth.

(3)P Seismically-oriented, additional investigations may be required in the cases indicated in **4.1** and **4.2.2**.

4.2.2 Determination of the ground type for the definition of the seismic action

(1)P Geotechnical or geological data for the construction site shall be available in sufficient quantity to allow the determination of an average ground type and/or the associated response spectrum, as defined in EN 1998-1:2004, **3.1**, **3.2**.

(2) For this purpose, in situ data may be integrated with data from adjacent areas with similar geological characteristics.

(3) Existing seismic microzonation maps or criteria should be taken into account, provided that they conform with (1)P of this subclause and that they are supported by ground investigations at the construction site.

(4)P The profile of the shear wave velocity v_s in the ground shall be regarded as the most reliable predictor of the site-dependent characteristics of the seismic action at stable sites.

(5) In situ measurements of the v_s profile by in-hole geophysical methods should be used for important structures in high seismicity regions, especially in the presence of ground conditions of type D, S_1, or S_2.

(6) For all other cases, when the natural vibration periods of the soil need to be determined, the v_s profile may be estimated by empirical correlations using the in situ penetration resistance or other geotechnical properties, allowing for the scatter of such correlations.

(7) Internal soil damping should be measured by appropriate laboratory or field tests. In the case of a lack of direct measurements, and if the product $a_g \cdot S$ is less than 0,1 g (i.e. less than 0,98 m/s^2), a damping ratio of 0,03 should be used. Structured and cemented soils and soft rocks may require separate consideration.

4.2.3 Dependence of the soil stiffness and damping on the strain level

(1)P The difference between the small-strain values of v_s, such as those measured by in situ tests, and the values compatible with the strain levels induced by the design earthquake shall be taken into account in all calculations involving dynamic soil properties under stable conditions.

(2) For local ground conditions of type C or D with a shallow water table and no materials with plasticity index $PI > 40$, in the absence of specific data, this may be done using the reduction factors for v_s given in Table 4.1. For stiffer soil profiles and a deeper water table the amount of reduction should be proportionately smaller (and the range of variation should be reduced).

(3) If the product $a_g \cdot S$ is equal to or greater than 0,1 g, (i.e. equal to or greater than 0,98 m/s^2), the internal damping ratios given in Table 4.1 should be used, in the absence of specific measurements.

Table 4.1 — Average soil damping ratios and average reduction factors (± one standard deviation) for shear wave velocity v_s and shear modulus G within 20 m depth.

Ground acceleration ratio, $\alpha.S$	Damping ratio	$\frac{v_s}{v_{s,max}}$	$\frac{G}{G_{max}}$
0,10	0,03	0,90(±0,07)	0,80(±0,10)
0,20	0,06	0,70(±0,15)	0,50(±0,20)
0,30	0,10	0,60(±0,15)	0,36(±0,20)

$v_{s,\,max}$ is the average v_s value at small strain (< 10^{-5}), not exceeding 360 m/s.

G_{max} is the average shear modulus at small strain.

NOTE Through the ± one standard deviation ranges the designer can introduce different amounts of conservatism, depending on such factors as stiffness and layering of the soil profile. Values of $v_s/v_{s,max}$ and G/G_{max} above the average could, for example, be used for stiffer profiles, and values of $v_s/v_{s,max}$ and G/G_{max} below the average could be used for softer profiles.

5 FOUNDATION SYSTEM

5.1 General requirements

(1)P In addition to the general rules of EN 1997-1:2004 the foundation of a structure in a seismic area shall conform to the following requirements.

a) The relevant forces from the superstructure shall be transferred to the ground without substantial permanent deformations according to the criteria of **5.3.2**.

b) The seismically-induced ground deformations are compatible with the essential functional requirements of the structure.

c) The foundation shall be conceived, designed and built following the rules of **5.2** and the minimum measures of **5.4** in an effort to limit the risks associated with the uncertainty of the seismic response.

(2)P Due account shall be taken of the strain dependence of the dynamic properties of soils (see **4.2.3**) and of effects related to the cyclic nature of seismic loading. The properties of in-situ improved or even substituted soil shall be taken into account if the improvement or substitution of the original soil is made necessary by its susceptibility to liquefaction or densification.

(3) Where appropriate (or needed), ground material or resistance factors other than those mentioned in **3.1** (2) may be used, provided that they correspond to the same level of safety.

NOTE Examples are resistance factors applied to the results of pile load tests.

5.2 Rules for conceptual design

(1)P In the case of structures other than bridges and pipelines, mixed foundation types, eg. piles with shallow foundations, shall only be used if a specific study demonstrates the adequacy of such a solution. Mixed foundation types may be used in dynamically independent units of the same structure.

(2)P In selecting the type of foundation, the following points shall be considered.

a) The foundation shall be stiff enough to uniformly transmit the localised actions received from the superstructure to the ground.

b) The effects of horizontal relative displacements between vertical elements shall be taken into account when selecting the stiffness of the foundation within its horizontal plane.

c) If a decrease in the amplitude of seismic motion with depth is assumed, this shall be justified by an appropriate study, and in no case may it correspond to a peak acceleration ratio lower than a certain fraction p of the product $\alpha \cdot S$ at the ground surface.

NOTE The value ascribed to p for use in a Country may be found in its National Annex. The recommended value is $p = 0{,}65$.

5.3 Design action effects

5.3.1 Dependence on structural design

(1)P *Dissipative structures*. The action effects for the foundations of dissipative structures shall be based on capacity design considerations accounting for the development of possible overstrength. The evaluation of such effects shall be in accordance with the appropriate clauses of the relevant parts of Eurocode 8. For buildings in particular the limiting provision of EN 1998-1:2004, **4.4.2.6** (2)P shall apply.

(2)P *Non-dissipative structures*. The action effects for the foundations of non-dissipative structures shall be obtained from the analysis in the seismic design situation without capacity design considerations. See also EN 1998-1:2004, **4.4.2.6** (3).

5.3.2 Transfer of action effects to the ground

(1)P To enable the foundation system to conform to **5.1**(1)P(a), the following criteria shall be adopted for transferring the horizontal force and the normal force/bending moment to the ground. For piles and piers the additional criteria specified in **5.4.2** shall be taken into account.

(2)P *Horizontal force*. The design horizontal shear force V_{Ed} shall be transferred by the following mechanisms:

a) by means of a design shear resistance F_{Rd} between the horizontal base of a footing or of a foundation-slab and the ground, as described in **5.4.1.1**;

b) by means of a design shear resistance between the vertical sides of the foundation and the ground;

c) by means of design resisting earth pressures on the side of the foundation, under the limitations and conditions described in **5.4.1.1**, **5.4.1.3** and **5.4.2**.

(3)P A combination of the shear resistance with up to 30% of the resistance arising from fully-mobilised passive earth pressures shall be allowed.

(4)P *Normal force and bending moment*. An appropriately calculated design normal force N_{Ed} and bending moment M_{Ed} shall be transferred to the ground by means of one or a combination of the following mechanisms:

a) by the design value of resisting vertical forces acting on the base of the foundation;

b) by the design value of bending moments developed by the design horizontal shear resistance between the sides of deep foundation elements (boxes, piles, caissons) and the ground, under the limitations and conditions described in **5.4.1.3** and **5.4.2**;

c) by the design value of vertical shear resistance between the sides of embedded and deep foundation elements (boxes, piles, piers and caissons) and the ground.

5.4 Verifications and dimensioning criteria

5.4.1 Shallow or embedded foundations

(1)P The following verifications and dimensioning criteria shall apply for shallow or embedded foundations bearing directly onto the underlying ground.

5.4.1.1 Footings (ultimate limit state design)

(1)P In accordance with the ultimate limit state design criteria, footings shall be checked against failure by sliding and against bearing capacity failure.

(2)P *Failure by sliding*. In the case of foundations having their base above the water table, this type of failure shall be resisted through friction and, under the conditions specified in (5) of this subclause, through lateral earth pressure.

(3) In the absence of more specific studies, the design friction resistance for footings above the water table, F_{Rd}, may be calculated from the following expression:

$$F_{\mathrm{Rd}} = N_{\mathrm{Ed}} \frac{\tan \delta}{\gamma_{\mathrm{M}}} \tag{5.1}$$

where

N_{Ed} is the design normal force on the horizontal base;

δ is the structure-ground interface friction angle on the base of the footing, which may be evaluated according to EN 1997-1:2004, **6.5.3**;

γ_{M} is the partial factor for material property, taken with the same value as that to be applied to tan ϕ' (see **3.1** (3)).

(4)P In the case of foundations below the water table, the design shearing resistance shall be evaluated on the basis of undrained strength, in accordance with EN 1997-1:2004, **6.5.3**.

(5) The design lateral resistance E_{pd} arising from earth pressure on the side of the footing may be taken into account as specified in **5.3.2**, provided appropriate measures are taken on site, such as compacting of backfill against the sides of the footing, driving a foundation vertical wall into the soil, or pouring a concrete footing directly against a clean, vertical soil face.

(6)P To ensure that there is no failure by sliding on a horizontal base, the following expression shall be satisfied.

$$V_{\mathrm{Ed}} \leq F_{\mathrm{Rd}} + E_{\mathrm{pd}} \tag{5.2}$$

(7) In the case of foundations above the water table, and provided that both of the following conditions are fulfilled:

- the soil properties remain unaltered during the earthquake;
- sliding does not adversely affect the performance of any lifelines (eg water, gas, access or telecommunication lines) connected to the structure;

a limited amount of sliding may be tolerated. The magnitude of sliding should be reasonable when the overall behaviour of the structure is considered.

(8)P *Bearing capacity failure*. To satisfy the requirement of **5.1** (1)P a), the bearing capacity of the foundation shall be verified under a combination of applied action effects N_{Ed}, V_{Ed}, and M_{Ed}.

NOTE To verify the seismic bearing capacity of the foundation, the general expression and criteria provided in Informative Annex F may be used, which allow the load inclination and eccentricity arising from the inertia forces in the structure as well as the possible effects of the inertia forces in the supporting soil itself to be taken into account.

(9) Attention is drawn to the fact that some sensitive clays might suffer a shear strength degradation, and that cohesionless materials are susceptible to dynamic pore pressure build-up under cyclic loading as well as to the upwards dissipation of the pore pressure from underlying layers after an earthquake.

(10) The evaluation of the bearing capacity of soil under seismic loading should take into account possible strength and stiffness degradation mechanisms which might start even at relatively low strain levels. If these phenomena are taken into account, reduced values for the partial factors for material properties may be used. Otherwise, the values referred to in **3.1** (3) should be used.

(11) The rise of pore water pressure under cyclic loading should be taken into account, either by considering its effect on undrained strength (in total stress analysis) or on pore pressure (in effective stress analysis). For structures with importance factor γ_I greater than 1,0, non-linear soil behaviour should be taken into account in determining possible permanent deformation during earthquakes.

5.4.1.2 Foundation horizontal connections

(1)P Consistent with **5.2** the additional action effects induced in the structure by horizontal relative displacements at the foundation shall be evaluated and appropriate measures to adapt the design taken.

(2) For buildings, the requirement specified in (1)P of this subclause is deemed to be satisfied if the foundations are arranged on the same horizontal plane and tie-beams or an adequate foundation slab are provided at the level of footings or pile caps. These measures are not necessary in the following cases: a) for ground type A, and b) in low seismicity cases for ground type B.

(3) The beams of the lower floor of a building may be considered as tie-beams provided that they are located within 1,0 m from the bottom face of the footings or pile caps. A foundation slab may possibly replace the tie-beams, provided that it too is located within 1,0 m from the bottom face of the footings or pile caps.

(4) The necessary tensile strength of these connecting elements may be estimated by simplified methods.

(5)P If more precise rules or methods are not available, the foundation connections shall be considered adequate when all the rules given in (6) and (7) of this subclause are met.

(6) *Tie-beams*

The following measures should be taken:

a) the tie-beams should be designed to withstand an axial force, considered both in tension and compression, equal to:

$\pm$ 0,3 $\alpha \cdot S \cdot N_{Ed}$ for ground type B

$\pm$ 0,4 $\alpha \cdot S \cdot N_{Ed}$ for ground type C

$\pm$ 0,6 $\alpha \cdot S \cdot N_{Ed}$ for ground type D

where N_{Ed} is the mean value of the design axial forces of the connected vertical elements in the seismic design situation;

b) longitudinal steel should be fully anchored into the body of the footing or into the other tie-beams framing into it.

(7) *Foundation slab*

The following measures should be taken:

a) Tie-zones should be designed to withstand axial forces equal to those given in (6) a) of this subclause.

b) The longitudinal steel of tie-zones should be fully anchored into the body of the footings or into the continuing slab.

5.4.1.3 Raft foundations

(1) All the provisions of **5.4.1.1** may also be applied to raft foundations, but with the following qualifications:

a) The global frictional resistance may be taken into account in the case of a single foundation slab. For simple grids of foundation beams, an equivalent footing area may be considered at each crossing.

b) Foundation beams and/or slabs may be considered as being the connecting ties; the rule for their dimensioning is applicable to an effective width corresponding to the width of the foundation beam or to a slab width equal to ten times its thickness.

(2) A raft foundation may also need to be checked as a diaphragm within its own plane, under its own lateral inertial loads and the horizontal forces induced by the superstructure.

5.4.1.4 Box-type foundations

(1) All the provisions of **5.4.1.3** may also be applied to box-type foundations. In addition, lateral soil resistance as specified in **5.3.2** (2) and **5.4.1.1** (5), may be taken into account in all soil categories, under the prescribed limitations.

5.4.2 Piles and piers

(1)P Piles and piers shall be designed to resist the following two types of action effects.

a) *Inertia forces* from the superstructure. Such forces, combined with the static loads, give the design values N_{Ed}, V_{Ed}, M_{Ed} specified in **5.3.2**.

b) *Kinematic forces* arising from the deformation of the surrounding soil due to the passage of seismic waves.

(2)P The ultimate transverse load resistance of piles shall be verified in accordance with the principles of EN 1997-1:2004, **7.7**.

(3)P Analyses to determine the internal forces along the pile, as well as the deflection and rotation at the pile head, shall be based on discrete or continuum models that can realistically (even if approximately) reproduce:

- the flexural stiffness of the pile;
- the soil reactions along the pile, with due consideration to the effects of cyclic loading and the magnitude of strains in the soil;
- the pile-to-pile dynamic interaction effects (also called dynamic "pile-group" effects);
- the degree of freedom of the rotation at/of the pile cap, or of the connection between the pile and the structure.

NOTE To compute the pile stiffnesses the expressions given in Informative Annex C may be used as a guide.

(4)P The side resistance of soil layers that are susceptible to liquefaction or to substantial strength degradation shall be ignored.

(5) If inclined piles are used, they should be designed to safely carry axial loads as well as bending loads.

NOTE Inclined piles are not recommended for transmitting lateral loads to the soil.

(6)P Bending moments developing due to kinematic interaction shall be computed only when all of the following conditions occur simultaneously:

- the ground profile is of type D, S_1 or S_2, and contains consecutive layers of sharply differing stiffness;
- the zone is of moderate or high seismicity, i.e. the product $a_g \cdot S$ exceeds 0,10 g , (i.e. exceeds 0,98 m/s^2), and the supported structure is of importance class III or IV.

(7) Piles should in principle be designed to remain elastic, but may under certain conditions be allowed to develop a plastic hinge at their heads. The regions of potential plastic hinging should be designed according to EN 1998-1:2004, **5.8.4**.

6 SOIL-STRUCTURE INTERACTION

(l)P The effects of dynamic soil-structure interaction shall be taken into account in:

a) structures where P-δ (2nd order) effects play a significant role;

b) structures with massive or deep-seated foundations, such as bridge piers, offshore caissons, and silos;

c) slender tall structures, such as towers and chimneys, covered in EN 1998-6:2004;

d) structures supported on very soft soils, with average shear wave velocity $v_{s,max}$ (as defined in Table 4.1) less than 100 m/s, such as those soils in ground type S_1.

NOTE Information on the general effects and significance of dynamic soil-structure interaction is given in Informative Annex D.

(2)P The effects of soil-structure interaction on piles shall be assessed according to **5.4.2** for all structures.

7 EARTH RETAINING STRUCTURES

7.1 General requirements

(l)P Earth retaining structures shall be designed to fulfil their function during and after an earthquake, without suffering significant structural damage.

(2) Permanent displacements, in the form of combined sliding and tilting, the latter due to irreversible deformations of the foundation soil, may be acceptable if it is shown that they are compatible with functional and/or aesthetic requirements.

7.2 Selection and general design considerations

(l)P The choice of the structural type shall be based on normal service conditions, following the general principles of EN 1997-1:2004, Section **9.**

(2)P Proper attention shall be given to the fact that conformity to the additional seismic requirements may lead to adjustment and, occasionally, to a more appropriate choice of structural type.

(3)P The backfill material behind the structure shall be carefully graded and compacted in situ, so as to achieve as much continuity as possible with the existing soil mass.

(4)P Drainage systems behind the structure shall be capable of absorbing transient and permanent movements without impairment of their functions.

(5)P Particularly in the case of cohesionless soils containing water, the drainage shall be effective to well below the potential failure surface behind the structures.

(6)P It shall be ensured that the supported soil has an enhanced safety margin against liquefaction under the design earthquake.

7.3 Methods of analysis

7.3.1 General methods

(l)P Any established method based on the procedures of structural and soil dynamics, and supported by experience and observations, is in principle acceptable for assessing the safety of an earth-retaining structure.

(2) The following aspects should be accounted for:

a) the generally non-linear behaviour of the soil in the course of its dynamic interaction with the retaining structure;

b) the inertial effects associated with the masses of the soil, of the structure, and of all other gravity loads which might participate in the interaction process;

c) the hydrodynamic effects generated by the presence of water in the soil behind the wall and/or by the water on the outer face of the wall;

d) the compatibility between the deformations of the soil, the wall, and the tiebacks, when present.

7.3.2 Simplified methods: pseudo-static analysis

7.3.2.1 Basic models

(l)P The basic model for pseudo-static analysis shall consist of the retaining structure and its foundation, of a soil wedge behind the structure supposed to be in a state of active limit equilibrium (if the structure is flexible enough), of any surcharge loading acting on the soil wedge, and, possibly, of a soil mass at the foot of the wall, supposed to be in a state of passive equilibrium.

(2) To produce an active soil state, a sufficient amount of wall movement is necessary to occur during the design earthquake which can be made possible for a flexible structure by bending, and for gravity structures by sliding or rotation. For the wall movement needed for development of an active limit state, see EN 1997-1:2004, **9.5.3**.

(3) For rigid structures, such as basement walls or gravity walls founded on rock or piles, greater than active pressures develop, and it is more appropriate to assume an at rest soil state, as shown in **E.9**. This should also be assumed for anchored retaining walls if no movement is permitted.

7.3.2.2 Seismic action

(l)P For the purpose of the pseudo-static analysis, the seismic action shall be represented by a set of horizontal and vertical static forces equal to the product of the gravity forces and a seismic coefficient.

(2)P The vertical seismic action shall be considered as acting upward or downward so as to produce the most unfavourable effect.

(3) The intensity of such equivalent seismic forces depends, for a given seismic zone, on the amount of permanent displacement which is both acceptable and actually permitted by the adopted structural solution.

(4)P In the absence of specific studies, the horizontal (k_h) and vertical (k_v) seismic coefficients affecting all the masses shall be taken as:

$$k_h = \alpha \frac{S}{r} \qquad (7.1)$$

$$k_v = \pm 0{,}5 k_h \quad \text{if } a_{vg}/a_g \text{ is larger than 0,6} \qquad (7.2)$$

$$k_v = \pm 0{,}33 k_h \quad \text{otherwise} \qquad (7.3)$$

where the factor r takes the values listed in Table 7.1 depending on the type of retaining structure. For walls not higher than 10 m, the seismic coefficient shall be taken as being constant along the height.

Table 7.1 — Values of factor *r* for the calculation of the horizontal seismic coefficient

Type of retaining structure	*r*
Free gravity walls that can accept a displacement up to d_r = 300 $\alpha \cdot S$ (mm)	2
Free gravity walls that can accept a displacement up to d_r = 200 $\alpha \cdot S$ (mm)	1,5
Flexural reinforced concrete walls, anchored or braced walls, reinforced concrete walls founded on vertical piles, restrained basement walls and bridge abutments	1

(5) In the presence of saturated cohesionless soils susceptible to the development of high pore pressure:

a) the *r* factor of Table 7.1 should not be taken as being larger than 1,0;

b) the safety factor against liquefaction should not be less than 2.

NOTE The value of 2 of the safety factor results from the application of clause **7.2**(6)P within the framework of the simplified method of clause **7.3.2**.

(6) For retaining structures more than 10m high and for additional information on the factor *r*, see **E.2**.

(7) For non-gravity walls, the effects of vertical acceleration may be neglected for the retaining structure.

7.3.2.3 Design earth and water pressure

(l)P The total design force acting on the wall under seismic conditions shall be calculated by considering the condition of limit equilibrium of the model described in **7.3.2.l**.

(2) This force may be evaluated according to Annex E.

(3) The design force referred to in (1)P of this subclause should be considered to be the resultant force of the static and the dynamic earth pressures.

(4)P The point of application of the force due to the dynamic earth pressures shall be taken to lie at mid-height of the wall, in the absence of a more detailed study taking into account the relative stiffness, the type of movements and the relative mass of the retaining structure.

(5) For walls which are free to rotate about their toe the dynamic force may be taken to act at the same point as the static force.

(6)P The pressure distributions on the wall due to the static and the dynamic action shall be taken to act with an inclination with respect to a direction normal to the wall not greater than $(2/3)\phi'$ for the active state and equal to zero for the passive state.

(7)P For the soil under the water table, a distinction shall be made between dynamically pervious conditions in which the internal water is free to move with respect to the solid skeleton, and dynamically impervious ones in which essentially no drainage can occur under the seismic action.

(8) For most common situations and for soils with a coefficient of permeability of less than $5x10^{-4}$ m/s, the pore water is not free to move with respect to the solid skeleton, the seismic action occurs in an essentially undrained condition and the soil may be treated as a single-phase medium.

(9)P For the dynamically impervious condition, all the previous provisions shall apply, provided that the unit weight of the soil and the horizontal seismic coefficient are appropriately modified.

(10) Modifications for the dynamically impervious condition may be made in accordance with **E.6** and **E.7**.

(11)P For the dynamically pervious backfill, the effects induced by the seismic action in the soil and in the water shall be assumed to be uncoupled effects.

(12) Therefore, a hydrodynamic water pressure should be added to the hydrostatic water pressure in accordance with **E.7**. The point of application of the force due to the hydrodynamic water pressure may be taken at a depth below the top of the saturated layer equal to 60% of the height of such a layer.

7.3.2.4 Hydrodynamic pressure on the outer face of the wall

(l)P The maximum (positive or negative) pressure fluctuation with respect to the existing hydrostatic pressure, due to the oscillation of the water on the exposed side of the wall, shall be taken into account.

(2) This pressure may be evaluated in accordance with **E.8**.

7.4 Stability and strength verifications

7.4.1 Stability of foundation soil

(l)P The following verifications are required:

- overall stability;
- local soil failure.

(2)P The verification of overall stability shall be carried out in accordance with the rules of **4.1.3.4**.

(3)P The ultimate capacity of the foundation shall be checked for failure by sliding and for bearing capacity failure (see **5.4.1.1**).

7.4.2 Anchorage

(l)P The anchorages (including free tendons, anchorage devices, anchor heads and the restraints) shall have enough resistance and length to assure equilibrium of the critical soil wedge under seismic conditions (see **7.3.2.1**), as well as a sufficient capacity to adapt to the seismic deformations of the ground.

(2)P The resistance of the anchorage shall be derived according to the rules of EN 1997-1:2004 for persistent and transient design situations at ultimate limit states.

(3)P It shall be ensured that the anchoring soil maintains the strength required for the anchor function during the design earthquake and, in particular, has an enhanced safety margin against liquefaction .

(4)P The distance L_e between the anchor and the wall shall exceed the distance L_s, required for non-seismic loads.

(5) The distance L_e, for anchors embedded in a soil deposit with similar characteristics to those of the soil behind the wall and for level ground conditions, may be evaluated in accordance with the following expression:

$$L_e = L_s(1 + 1{,}5\alpha \cdot S) \tag{7.4}$$

7.4.3 Structural strength

(l)P It shall be demonstrated that, under the combination of the seismic action with other possible loads, equilibrium is achieved without exceeding the design strengths of the wall and the supporting structural elements.

(2)P For that purpose, the pertinent limit state modes for structural failure in EN 1997-1:2004, **8.5** shall be considered.

(3)P All structural elements shall be checked to ensure that they satisfy the condition

$$R_d > E_d \tag{7.5}$$

where

R_d is the design value of the resistance of the element, evaluated in the same way as for the non seismic situation;

E_d is the design value of the action effect, as obtained from the analysis described in **7.3**.

Annex A (Informative)

Topographic amplification factors

A.l This annex gives some simplified amplification factors for the seismic action used in the verification of the stability of ground slopes. Such factors, denoted S_T, are to a first approximation considered independent of the fundamental period of vibration and, hence, multiply as a constant scaling factor the ordinates of the elastic design response spectrum given in EN 1998-1:2004. These amplification factors should in preference be applied when the slopes belong to two-dimensional topographic irregularities, such as long ridges and cliffs of height greater than about 30 m.

A.2 For average slope angles of less than about 15° the topography effects may be neglected, while a specific study is recommended in the case of strongly irregular local topography. For greater angles the following guidelines are applicable.

a) *Isolated cliffs and slopes.* A value $S_T \geq 1,2$ should be used for sites near the top edge;

b) *Ridges with crest width significantly less than the base width.* A value $S_T \geq 1,4$ should be used near the top of the slopes for average slope angles greater then 30° and a value $S_T \geq 1,2$ should be used for smaller slope angles;

c) *Presence of a loose surface layer*. In the presence of a loose surface layer, the smallest S_T value given in a) and b) should be increased by at least 20%;

d) *Spatial variation of amplification factor*. The value of S_T may be assumed to decrease as a linear function of the height above the base of the cliff or ridge, and to be unity at the base.

A.3 In general, seismic amplification also decreases rapidly with depth within the ridge. Therefore, topographic effects to be reckoned with in stability analyses are largest and mostly superficial along ridge crests, and much smaller on deep seated landslides where the failure surface passes near to the base. In the latter case, if the pseudo-static method of analysis is used, the topographic effects may be neglected.

Annex B (Normative)

Empirical charts for simplified liquefaction analysis

B.l *General*. The empirical charts for simplified liquefaction analysis represent field correlations between in situ measurements and cyclic shear stresses known to have caused liquefaction during past earthquakes. On the horizontal axis of such charts is a soil property measured in situ, such as normalised penetration resistance or shear wave propagation velocity v_s, while on the vertical axis is the earthquake-induced cyclic shear stress (τ_e), usually normalised by the effective overburden pressure (σ'_{vo}). Displayed on all charts is a limiting curve of cyclic resistance, separating the region of no liquefaction (to the right) from that where liquefaction is possible (to the left and above the curve). More than one curve is sometimes given, e.g. corresponding to soils with different fines contents or to different earthquake magnitudes.

Except for those using CPT resistance, it is preferable not to apply the empirical liquefaction criteria when the potentially liquefiable soils occur in layers or seams no more than a few tens of cm thick.

When a substantial gravel content is present, the susceptibility to liquefaction cannot be ruled out, but the observational data are as yet insufficient for construction of a reliable liquefaction chart.

B.2 *Charts based on the SPT blowcount*. Among the most widely used are the charts illustrated in Figure B.l for clean sands and silty sands. The SPT blowcount value normalised for overburden effects and for energy ratio $N_1(60)$ is obtained as described in **4.1.4**.

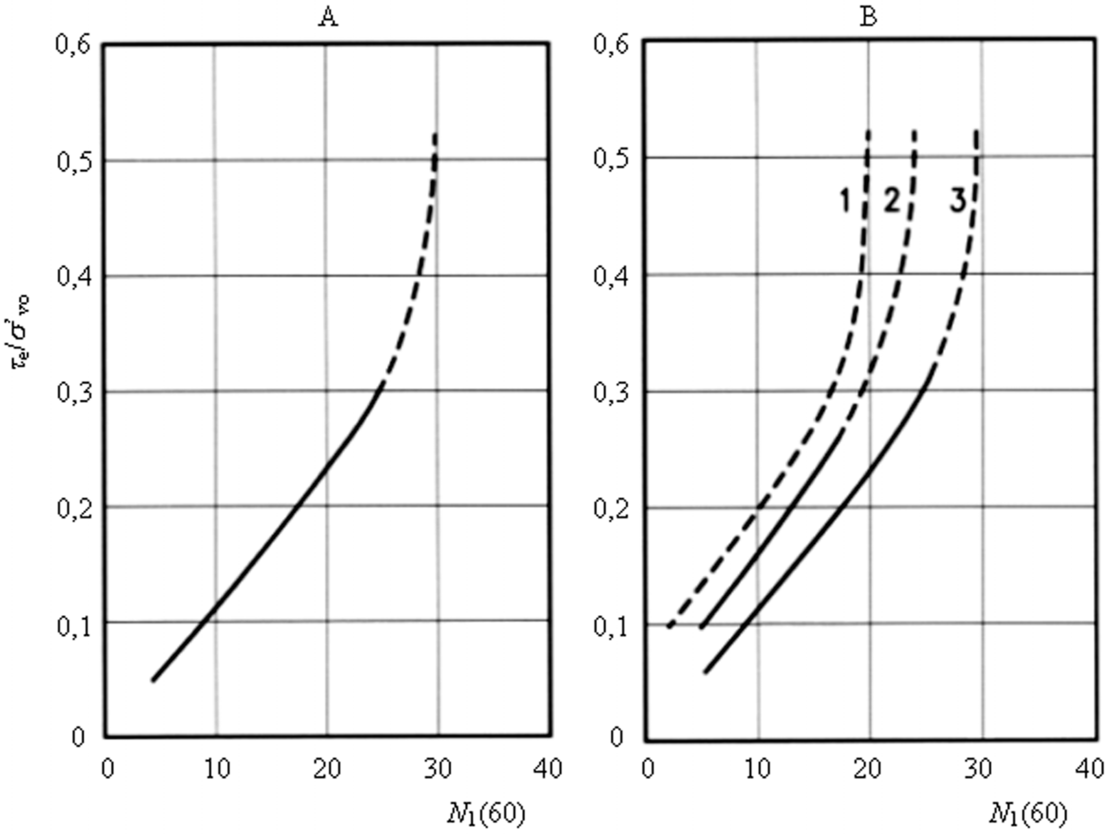
Liquefaction is not likely to occur below a certain threshold of τ_e, because the soil behaves elastically and no pore-pressure accumulation takes place. Therefore, the limiting curve is not extrapolated back to the origin. To apply the present criterion to earthquake magnitudes different from M_S = 7,5, where M_S is the surface-wave magnitude, the ordinates of the curves in Figure B.l should be multiplied by a factor CM indicated in Table B.1.

Table B.1 — Values of factor CM

M_S	CM
5,5	2,86
6,0	2,20
6,5	1,69
7,0	1,30
8,0	0,67

B.3 *Charts based on the CPT resistance*. Based on numerous studies on the correlation between CPT cone resistance and soil resistance to liquefaction, charts similar to Figure B.1 have been established. Such direct correlations shall be preferred to indirect correlations using a relationship between the SPT blowcount and the CPT cone resistance.

B.4 *Charts based on the shear wave velocity v_s*. This property has strong promise as a field index in the evaluation of liquefaction susceptibility in soils that are hard to sample (such as silts and sands) or penetrate (gravels). Also, significant advances have been made over the last few years in measuring v_s in the field. However, correlations between v_s and the soil resistance to liquefaction are still under development and should not be used without the assistance of a specialist.



Key

τ_e/σ'_{vo} – cyclic stress ratio

A – clean sands;

B – silty sands

curve 1: 35 % fines

curve 2: 15% fines

curve 3: < 5% fines

Figure B.1 — Relationship between stress ratios causing liquefaction and N_1 (60) values for clean and silty sands for M_S=7,5 earthquakes.

Annex C (Informative)

Pile-head static stiffnesses

C.l The pile stiffness is defined as the force (moment) to be applied to the pile head to produce a unit displacement (rotation) along the same direction (the displacements/rotations along the other directions being zero), and is denoted by K_{HH} (horizontal stiffness), K_{MM} (flexural stiffness) and $K_{\mathrm{HM}} = K_{\mathrm{MH}}$ (cross stiffness).

The following notations are used in Table C.l below:

E is Young's modulus of the soil model, equal to 3G;

E_{p} is Young's modulus of the pile material;

E_{s} is Young's modulus of the soil at a depth equal to the pile diameter;

d is the pile diameter;

z is the pile depth.

Table C.l — Expressions for static stiffness of flexible piles embedded in three soil models

Soil model	$\dfrac{K_{\mathrm{HH}}}{dE_{\mathrm{s}}}$	$\dfrac{K_{\mathrm{MM}}}{d^3E_{\mathrm{s}}}$	$\dfrac{K_{\mathrm{HM}}}{d^2E_{\mathrm{s}}}$
$E = E_{\mathrm{s}} \cdot z/d$	$0{,}60\left(\dfrac{E_{\mathrm{p}}}{E_{\mathrm{s}}}\right)^{0,35}$	$0{,}14\left(\dfrac{E_{\mathrm{p}}}{E_{\mathrm{s}}}\right)^{0,80}$	$-0{,}17\left(\dfrac{E_{\mathrm{p}}}{E_{\mathrm{s}}}\right)^{0,60}$
$E = E_{\mathrm{s}}\sqrt{z/d}$	$0{,}79\left(\dfrac{E_{\mathrm{p}}}{E_{\mathrm{s}}}\right)^{0,28}$	$0{,}15\left(\dfrac{E_{\mathrm{p}}}{E_{\mathrm{s}}}\right)^{0,77}$	$-0{,}24\left(\dfrac{E_{\mathrm{p}}}{E_{\mathrm{s}}}\right)^{0,53}$
$E = E_{\mathrm{s}}$	$1{,}08\left(\dfrac{E_{\mathrm{p}}}{E_{\mathrm{s}}}\right)^{0,21}$	$0{,}16\left(\dfrac{E_{\mathrm{p}}}{E_{\mathrm{s}}}\right)^{0,75}$	$-0{,}22\left(\dfrac{E_{\mathrm{p}}}{E_{\mathrm{s}}}\right)^{0,50}$

Annex D (Informative)

Dynamic soil-structure interaction (SSI). General effects and significance

D.l As a result of dynamic SSI, the seismic response of a flexibly-supported structure, i.e. a structure founded on deformable ground, will differ in several ways from that of the same structure founded on rigid ground (fixed base) and subjected to an identical free-field excitation, for the following reasons:

a) the foundation motion of the flexibly-supported structure will differ from the free-field motion and may include an important rocking component of the fixed-base structure;

b) the fundamental period of vibration of the flexibly-supported structure will be longer than that of the fixed-base structure;

c) the natural periods, mode shapes and modal participation factors of the flexibly-supported structure will be different from those of the fixed-base structure;

d) the overall damping of the flexibly-supported structure will include both the radiation and the internal damping generated at the soil-foundation interface, in addition to the damping associated with the superstructure.

D.2 For the majority of common building structures, the effects of SSI tend to be beneficial, since they reduce the bending moments and shear forces in the various members of the superstructure. For the structures listed in Section **6** the SSI effects might be detrimental.

Annex E (Normative)

Simplified analysis for retaining structures

E.l Conceptually, the factor r is defined as the ratio between the acceleration value producing the maximum permanent displacement compatible with the existing constraints, and the value corresponding to the state of limit equilibrium (onset of displacements). Hence, r is greater for walls that can tolerate larger displacements.

E.2 For retaining structures more than 10 m high, a free-field one-dimensional analysis of vertically propagating waves may be carried out and a more refined estimate of α, for use in expression (7.1), may be obtained by taking an average value of the peak horizontal soil accelerations along the height of the structure.

E.3 The total design force acting on the retaining structure from the land-ward side, E_d is given by

$$E_d = \frac{1}{2}\gamma^* (1 \pm k_v)\, K \cdot H^2 + E_{ws} + E_{wd} \qquad \text{(E.1)}$$

where

H is the wall height;

E_{ws} is the static water force;

E_{wd} is the hydrodynamic water force (defined below);

γ^* is the soil unit weight (defined below in E.5 to E.7);

K is the earth pressure coefficient (static + dynamic);

k_v is the vertical seismic coefficient (see expressions (7.2) and (7.3)).

E.4 The earth pressure coefficient may be computed from the Mononobe and Okabe formula.

For active states:

if $\beta \leq \phi'_d - \theta$

$$K = \frac{\sin^2(\psi + \phi'_d - \theta)}{\cos\theta \sin^2\psi \sin(\psi - \theta - \delta_d)\left[1 + \sqrt{\dfrac{\sin(\phi'_d + \delta_d)\sin(\phi'_d - \beta - \theta)}{\sin(\psi - \theta - \delta_d)\sin(\psi + \beta)}}\right]^2} \qquad \text{(E.2)}$$

if $\beta > \phi'_d - \theta$

$$K = \frac{\sin^2(\psi + \phi - \theta)}{\cos\theta \sin^2\psi \sin(\psi - \theta - \delta_d)} \qquad \text{(E.3)}$$

For passive states (no shearing resistance between the soil and the wall):

$$K = \frac{\sin^2(\psi + \phi'_d - \theta)}{\cos\theta \sin^2\psi \sin(\psi + \theta)\left[1 - \sqrt{\frac{\sin\phi'_d \sin(\phi'_d + \beta - \theta)}{\sin(\psi + \beta)\sin(\psi + \theta)}}\right]^2}. \tag{E.4}$$

In the preceeding expressions the following notations are used:

ϕ'_d is the design value of the angle of shearing resistance of soil i.e. $\phi'_d = \tan^{-1}\left(\frac{\tan\phi'}{\gamma_{\phi'}}\right)$;

ψ and β are the inclination angles of the back of the wall and backfill surface from the horizontal line, as shown in Figure E.l;

δ_d is the design value of the angle of shearing resistance between the soil and the wall i.e. $\delta_d = \tan^{-1}\left(\frac{\tan\delta}{\gamma_{\phi'}}\right)$;

θ is the angle defined below in **E.5** to **E.7**.

The passive states expression should preferably be used for a vertical wall face (ψ = 90°).

E.5 *Water table below retaining wall - Earth pressure coefficient.*

The following parameters apply:

γ* is the γ unit weight of soil (E.5)

$$\tan\theta = \frac{k_h}{1 \mp k_v} \tag{E.6}$$

$$E_{wd} = 0 \tag{E.7}$$

where

k_h is the horizontal seismic coefficient (see expression (7.1)).

Alternatively, use may be made of tables and graphs applicable for the static condition (gravity loads only) with the following modifications:

denoting

$$\tan\theta_A = \frac{k_h}{1 + k_v} \tag{E.8}$$

and

$$\tan\theta_B = \frac{k_h}{1 - k_v} \tag{E.9}$$

the entire soil-wall system is rotated appropriately by the additional angle θ_A or θ_B. The acceleration of gravity is replaced by the following value:

$$g_A = \frac{g(1+k_v)}{\cos\theta_A} \tag{E.10}$$

or

$$g_B = \frac{g(1-k_v)}{\cos\theta_B} \tag{E.11}$$

E.6 *Dynamically impervious soil below the water table - Earth pressure coefficient.*

The following parameters apply:

$$\gamma^* = \gamma - \gamma_w \tag{E.12}$$

$$\tan\theta = \frac{\gamma}{\gamma - \gamma_w} \frac{k_h}{1 \mp k_v} \tag{E.13}$$

$$E_{wd} = 0 \tag{E.14}$$

where:

γ is the saturated (bulk) unit weight of soil;

γ_w is the unit weight of water.

E.7 *Dynamically (highly) pervious soil below the water table - Earth pressure coefficient.*

The following parameters apply:

$$\gamma^* = \gamma - \gamma_w \tag{E.15}$$

$$\tan\theta = \frac{\gamma_d}{\gamma - \gamma_w} \frac{k_h}{1 \mp k_v} \tag{E.16}$$

$$E_{wd} = \frac{7}{12} k_h \cdot \gamma_w \cdot H'^2 \tag{E.17}$$

where:

γ_d is the dry unit weight of the soil;

H' is the height of the water table from the base of the wall.

E.8 *Hydrodynamic pressure on the outer face of the wall.*

This pressure, *q(z),* may be evaluated as:

$$q(z) = \pm\frac{7}{8}\ k_h \cdot \gamma_w \cdot \sqrt{h \cdot z} \qquad \text{(E.18)}$$

where

k_h is the horizontal seismic coefficient with r = 1 (see expression (7.1));

h is the free water height;

z is the vertical downward coordinate with the origin at the surface of water.

E.9 *Force due to earth pressure for rigid structures*

For rigid structures which are completely restrained, so that an active state cannot develop in the soil, and for a vertical wall and horizontal backfill the dynamic force due to earth pressure increment may be taken as being equal to

$$\Delta P_d = \alpha \cdot S \cdot \gamma \cdot H^2 \qquad \text{(E.19)}$$

where

H is the wall height.

The point of application may be taken at mid-height.

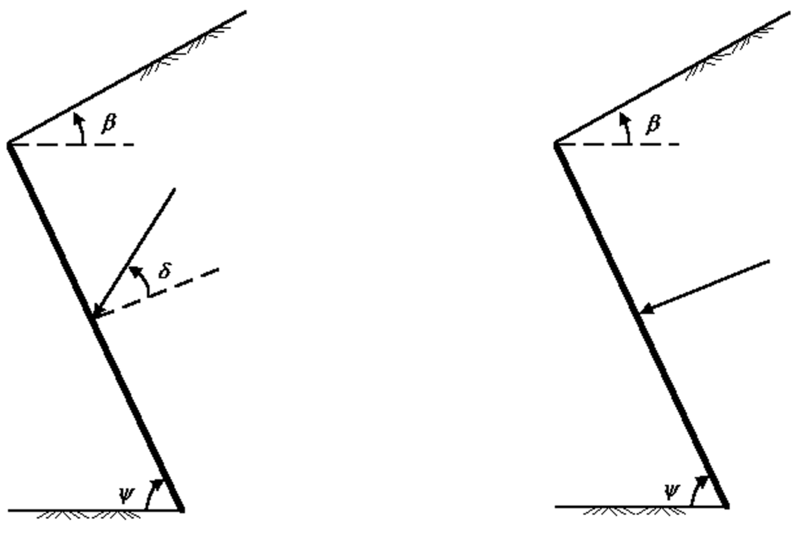


Figure E.1 — Convention for angles in formulae for calculating the earth pressure coefficient

Annex F (Informative)

Seismic bearing capacity of shallow foundations

F.1 *General expression.* The stability against seismic bearing capacity failure of a shallow strip footing resting on the surface of homogeneous soil, may be checked with the following expression relating the soil strength, the design action effects (N_{Ed}, V_{Ed}, M_{Ed}) at the foundation level, and the inertia forces in the soil

$$\frac{\left(1-e\overline{F}\right)^{c_T}\left(\beta\overline{V}\right)^{c_T}}{\left(\overline{N}\right)^{a}\left[\left(1-m\overline{F}^{k}\right)^{k'}-\overline{N}\right]^{b}}+\frac{\left(1-f\overline{F}\right)^{c'_M}\left(\gamma\overline{M}\right)^{c_M}}{\left(\overline{N}\right)^{c}\left[\left(1-m\overline{F}^{k}\right)^{k'}-\overline{N}\right]^{d}}-1\leq 0 \qquad \text{(F.1)}$$

where:

$$\overline{N}=\frac{\gamma_{Rd}N_{Ed}}{N_{max}}\quad,\quad \overline{V}=\frac{\gamma_{Rd}V_{Ed}}{N_{max}}\quad,\quad \overline{M}=\frac{\gamma_{Rd}M_{Ed}}{B\,N_{max}} \qquad \text{(F.2)}$$

N_{max} is the ultimate bearing capacity of the foundation under a vertical centered load, defined in **F.2** and **F.3**;

B is the foundation width;

$\overline{F}$ is the dimensionless soil inertia force defined in **F.2** and **F.3**;

γ_{Rd} is the model partial factor (values for this parameter are given in **F.6**).

a, b, c, d, e, f, m, k, k', c_T, c_M, c'_M, β, γ are numerical parameters depending on the type of soil, defined in **F.4**.

F.2 *Purely cohesive soil.* For purely cohesive soils or saturated cohesionless soils the ultimate bearing capacity under a vertical concentric load N_{max} is given by

$$N_{max}=\left(\pi+2\right)\frac{\overline{c}}{\gamma_M}B \qquad \text{(F.3)}$$

where

$\overline{c}$ is the undrained shear strength of soil, c_u, for cohesive soil, or the cyclic undrained shear strength, $\tau_{cy,u}$, for cohesionless soils;

γ_M is the partial factor for material properties (see **3.1** (3)).

The dimensionless soil inertia force $\overline{F}$ is given by

$$\overline{F}=\frac{\rho\cdot a_g\cdot S\cdot B}{\overline{c}} \qquad \text{(F.4)}$$

where

ρ is the unit mass of the soil;

a_g is the design ground acceleration on type A ground ($a_g = \gamma_I\ a_{gR}$);

a_{gR} is the reference peak ground acceleration on type A ground;

γ_I is the importance factor;

S is the soil factor defined in EN 1998-1:2004, **3.2.2.2**.

The following constraints apply to the general bearing capacity expression

$$0 < \overline{N} \leq 1 \quad , \quad \left|\overline{V}\right| \leq 1 \tag{F.5}$$

F.3 *Purely cohesionless soil*. For purely dry cohesionless soils or for saturated cohesionless soils without significant pore pressure building the ultimate bearing capacity of the foundation under a vertical centered load N_{max} is given by

$$N_{max} = \frac{1}{2} \rho\ g \left(1 \pm \frac{a_v}{g}\right) B^2\ N_\gamma \tag{F.6}$$

where

g is the acceleration of gravity;

a_v is the vertical ground acceleration, that may be taken as being equal to $0{,}5 a_g \cdot S$ and

N_γ is the bearing capacity factor, a function of the design angle of the shearing resistance of soil ϕ'_d (which includes the partial factor for material property γ_M of **3.1**(3), see **E.4**).

The dimensionless soil inertia force $\overline{F}$ is given by:

$$\overline{F} = \frac{a_g}{g\ \tan\phi'_d} \tag{F.7}$$

The following constraint applies to the general expression

$$0 < \overline{N} \leq \left(1 - m\overline{F}\right)^{k'} \tag{F.8}$$

F4 *Numerical parameters*. The values of the numerical parameters in the general bearing capacity expression, depending on the types of soil identified in **F.2** and **F.3**, are given in Table F.1.

Table F.1 — Values of numerical parameters used in expression (F.1)

	Purely cohesive soil	Purely cohesionless soil
a	0,70	0,92
b	1,29	1,25
c	2,14	0,92
d	1,81	1,25
e	0,21	0,41
f	0,44	0,32
m	0,21	0,96
k	1,22	1,00
k'	1,00	0,39
c_T	2,00	1,14
c_M	2,00	1,01
c'_M	1,00	1,01
β	2,57	2,90
γ	1,85	2,80

F.5 In most common situations $\overline{F}$ may be taken as being equal to 0 for cohesive soils. For cohesionless soils $\overline{F}$ may be neglected if $a_g \cdot S < 0,1\ g$ (i.e., if $a_g \cdot S < 0,98$ m/s^2).

F.6 The model partial factor γ_{Rd} takes the values indicated in Table F.2.

Table F.2 — Values of the model partial factor γ_{Rd}

Medium-dense to dense sand	Loose dry sand	Loose saturated sand	Non sensitive clay	Sensitive clay
1,00	1,15	1,50	1,00	1,15